Town of Oro Valley Drainage Criteria Manual

Town of Oro Valley

Department of Public Works

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List of Abbreviations & Acronyms

ADOT	=	Arizona Department of Transportation
ADWR	=	Arizona Department of Water Resources
AEP	=	Annual Exceedance Probability
ARF	=	Areal Reduction Factor
BFE	=	Base Flood Elevation
BMPs	=	Best Management Practices
C1FHP	=	Category 1 Flood Hydrology Procedure
C2FHP	=	Category 2 Flood Hydrology Procedure
CAF	=	Contributing Area Factor
CC&Rs	=	Covenants, Conditions, and Restrictions
CDO	=	Cañada Del Oro
cfs	=	cubic feet per second
CLOMR	=	Conditional Letter of Map Revision
CMP	=	Corrugated Metal Pipe
CN	=	Curve Number
Corps	=	United States Army Corps of Engineers
COT	=	City of Tucson
COTDSM	=	City of Tucson Drainage Standards Manual
DCM	=	Drainage Criteria Manual
EGL	=	Energy Grade Line
FEMA	=	Federal Emergency Management Agency
FFE	=	Finished-Floor Elevation
FHWA	=	Federal Highway Administration
FIRM	=	Flood Insurance Rate Map
ft	=	feet (foot)
HDPP	=	High-Density Polyethylene Pipe
HEC	=	Hydrologic Engineering Center

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HEC-1	=	Hydrologic Engineering Center Flood Hydrograph Package Computer Program
HEC-2	=	Hydrologic Engineering Center Water-Surface Profiles Computer Program
HEC-6	=	Hydrologic Engineering Center Scour and Deposition in Rivers and Reservoirs Computer Program
HEC-HMS	=	Hydrologic Engineering Center Hydrologic Modeling System Computer Program
HEC-RAS	=	Hydrologic Engineering Center River Analysis System Computer Program
HGL	=	Hydraulic Grade Line
hr(s)	=	hour(s)
HSG	=	Hydrologic Soil Group
in	=	inch(es)
KWM	=	Kinematic Wave Method
LOMR	=	Letter of Map Revision
LTD	=	Long-Term Degradation
min	=	minute(s)
MPM	=	Modified-Puls Method
NEH	=	National Engineering Handbook
NOAA	=	National Oceanic and Atmospheric Administration
NPDES	=	National Pollutant Discharge Elimination System
NRCS	=	Natural Resources Conservation Service
PAAL	=	Parking Area Access Lane
PAG	=	Pima Association of Governments
PCDOT	=	Pima Country Department of Transportation
PCRFCD	=	Pima County Regional Flood Control District
PC-ROUTE	=	Pima County Reservoir Routing Computer Program
PCRWRD	=	Pima County Regional Wastewater Reclamation Department
RCAC	=	Reinforced Concrete Arch Culvert

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RCBC	=	Reinforced Concrete Box Culvert
RCC	=	Reinforced Concrete Culvert
RCP	=	Reinforced Concrete Pipe
RFE	=	Regulatory Flood Elevation
ROW	=	Right-Of-Way
RREs	=	Regional Regression Equations
SCS	=	Soil Conservation Service
SFH	=	Synthetic Flood Hydrograph
SLA	=	Simons, Li & Associates, Inc.
SRP	=	Spiral-Ribbed Pipe
TSS	=	Time to Achieve a Stable Slope
USACE	=	United States Army Corps of Engineers
USBR	=	United States Bureau of Reclamation
USGS	=	United States Geological Survey
WSEL	=	Water-Surface Elevation
WUS	=	Waters of the United States

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List of Definitions

ALLUVIAL CHANNEL is a water-carrying CHANNEL made up of loose SEDIMENTS, called alluvium. An ALLUVIAL CHANNEL is subject to bed scour and bank EROSION/LATERAL MIGRATION during the occurrence of significant flows.

ALL-WEATHER ACCESS is a safe vehicular route which either ordinary or emergency vehicles require for the purpose of unimpeded access to a specific location. This standard applies to public or private streets, or to a designated route connecting a street and the DEVELOPMENT or building in question. Stormwater runoff flowing either across or in the direction of an ALL-WEATHER ACCESS route shall not have a depth of flow, y, in feet, plus VELOCITY HEAD, $V^2/2g$, in feet, that exceeds a numerical value of 1.30 for a duration in excess of thirty minutes during a 1% AEP FLOOD.

ALTERNATE DEPTHS are the two depths of flow possible—one depth which is lower than CRITICAL DEPTH, and one depth which is higher than CRITICAL DEPTH—for a given rate of flow and a given SPECIFIC HEAD. Refer to the definitions of CRITICAL DEPTH and SPECIFIC HEAD.

ANNUAL EXCEEDANCE PROBABILITY (AEP) is the probability of a specific FLOOD being equaled or exceeded in any given year. For example, a 1% AEP FLOOD is a FLOOD with one chance in one-hundred of being equaled or exceeded in any given year (i.e., a so-called 100-year FLOOD).

AREAL REDUCTION FACTOR (ARF) is a factor that is to be applied to point-precipitation values found using online *NOAA Atlas 14 Point Precipitation Frequency Estimates* in order to produce reduced precipitation values which can be applied to WATERSHEDS larger than 1 square mile in size, thus yielding the “areal” precipitation value for the same duration and return period of the corresponding point precipitation.

ARIZONA POLLUTANT DISCHARGE ELIMINATION SYSTEM (AZPDES) PERMIT is a permit for municipal, domestic, and non-domestic (industrial) discharges of pollutants to a surface water that fits the definition of “WATERS OF THE U.S.,” as described in the CLEAN WATER ACT. The permit is valid for five years. A renewal permit can be continued past five years if a renewal application is received 180 days prior to the expiration date of the permit.

ARTERIAL STREET is a street that carries high levels of traffic, typically serving over 12,000 vehicles per day.

AT-GRADE CROSSING is a depression or vertical SAG in the roadway designed to allow drainage to cross "at-grade" without using CULVERTS (also see the definition of DIP).

BACKWATER is the increase in upstream water-surface levels caused by downstream TAILWATER impacts. BACKWATER can also refer to the calculations that are performed to compute water-surface profiles in an open CHANNEL.

BALANCED BASIN/WATERSHED means a DRAINAGE BASIN which contains floodwater CHANNELS, natural or manmade, and/or flood-control STRUCTURES that are just adequate to contain existing runoff from a BASE FLOOD produced by the basin; but in which any added runoff cannot be safely contained by the existing drainage CHANNELS or hydraulic STRUCTURES.

BANK PROTECTION is a form of CHANNEL LINING where only the banks of a WATERCOURSE are protected from bank EROSION/LATERAL MIGRATION of a CHANNEL due to flowing waters.

BANKING is the lowering or tilting of the inside of the floor around a bend in a totally lined drainage CHANNEL in order to counteract the effects of superelevation of the water surface along the outer bank.

BASE FLOOD is a FLOOD stage or FLOOD height that, statistically, has a 1% AEP. The BASE FLOOD is often referred to as the ONE-HUNDRED-YEAR (100-YEAR) FLOOD.

BASE FLOOD ELEVATION (BFE) is the water-surface elevation of a BASE FLOOD at a specific location along a REGULATORY WATERCOURSE. The BFE is located 1 foot below the REGULATORY FLOOD ELEVATION established by the state of Arizona for Floodplain Management purposes.

BASIN MANAGEMENT STUDY is a study that incorporates a geographically-based approach of protecting and restoring water quality and quantity within a basin or watershed.

BED LOAD describes those sediments in stormwater runoff that are transported along the streambed of a watercourse. BED LOAD complements SUSPENDED LOAD and WASH LOAD. BED LOAD moves by rolling, sliding, and/or saltating (hopping).

BED-MATERIAL SEDIMENTS refers to those sediments contained in stormwater runoff that are comprised of BED LOAD and SUSPENDED LOAD, but excluding WASH LOAD.

BEST MANAGEMENT PRACTICES (BMPs) are methods that have been determined to be the most effective and practical means of preventing or reducing non-point-source pollution to help achieve water-quality goals.

CATCH BASIN refers to an appurtenance to STORM-DRAIN inlets which is used primarily to capture runoff and secondarily to trap solid, waterborne debris.

CATEGORY 1 FLOOD HYDROLOGY PROCEDURE (C1FHP) is a procedure that shall be used based upon the size and the natural hydrologic/hydraulic characteristics of the contributing watershed. That is, the CIFHP shall be used for natural watersheds and for uncontrolled urban watersheds (e.g., those with no significant stormwater DETENTION facilities) when contributing drainage areas do not exceed 1.0 square mile (640 acres) in size.

CATEGORY 2 FLOOD-HYDROLOGY PROCEDURE (C2FHP) is a procedure that can be used for any size watershed, but shall be used primarily for (1) natural watersheds larger than 1.0 square mile in size and (2) for controlled urban watersheds (e.g., those with significant stormwater DETENTION facilities) of any size. A hydrologic model such as HEC-HMS is to be used in conjunction with the C2FHP.

CHANNEL refers to a DRAINAGEWAY which has been created or extensively modified by man for the purpose of conveying floodwaters, and is no longer a WASH or a WATERCOURSE which exits in its natural condition.

CHANNEL LINING is erosion-resistant armoring or protection that is placed along the bottom and/or sides of drainage CHANNELS in order to prevent bed and bank scour as well as LATERAL MIGRATION.

CHECKLIST is a “stand-alone” document that shall be included as Appendix A in the back portion of any DRAINAGE REPORT or HYDROLOGY STUDY, and shall list those elements in each Chapter of the DCM that were utilized in the preparation of the Tentative DEVELOPMENT Plan. This CHECKLIST shall be included with all DRAINAGE REPORT and Drainage Plan submittals. The CHECKLIST shall also prominently include the name(s) of the Arizona Registered Professional Civil Engineer(s) who prepared the CHECKLIST, along with the name(s) of the Registrant(s) validating that the CHECKLIST is complete, accurate, and who is/are certifying to the accuracy and validity of the information provided therein.

CLEAN WATER ACT (CWA) is a federal act that establishes the basic structure for regulating discharges of pollutants into the “WATERS OF THE U.S.,” and regulating quality standards for surface waters.

CLOSED CONDUIT means any closed natural or artificial duct, such as a pipe or box culvert, for conveying fluids.

COLLECTOR CHANNEL is a drainage CHANNEL normally designed to capture DISPERSED SURFACE FLOW (SHEETFLOW) so that it can be concentrated for conveyance to a desired point using a CONVEYOR CHANNEL.

COMBINATION INLET is a pavement inlet consisting of a combined GUTTER inlet and CURB inlet.

CONDITIONAL LETTER OF MAP REVISION (CLOMR) is FEMA's comment on a proposed project that would, upon construction, affect the hydrologic or hydraulic characteristics of a flooding source and thus result in the modification of the existing REGULATORY FLOODWAY, the effective BFEs, or the SFHA. The letter does not revise an effective NFIP map, it indicates whether the project, if built as proposed, would be recognized by FEMA. Building permits cannot be issued based on a CLOMR, because a CLOMR does not change the NFIP map.

CONVEYOR CHANNEL is a drainage CHANNEL which generally receives flow from an upstream COLLECTOR CHANNEL, or CHANNELS, for conveyance to some downstream location.

COVENANTS, CONDITIONS, AND RESTRICTIONS (CC&Rs) are limits and rules placed on a group of homes or condominium complex by a builder, developer, neighborhood association, or homeowners association.

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CRITICAL BASIN/WATERSHED means a DRAINAGE BASIN which contains floodwater CHANNELS, natural or man-made, and/or flood-control STRUCTURES within its basin that cannot contain existing runoff produced by a BASE FLOOD; and which has a documented history of severe flooding hazards.

CRITICAL DEPTH is that particular depth of flow within an open CHANNEL which, for a given discharge, has its SPECIFIC ENERGY at a minimum. The given discharge may flow at an ALTERNATE DEPTH above or below CRITICAL DEPTH in a given CHANNEL, but the SPECIFIC ENERGY of the flow at either ALTERNATE DEPTH will be greater than that for flow at CRITICAL DEPTH. The FROUDE NUMBER of flow at CRITICAL DEPTH equals 1.0.

CRITICAL FLOW occurs when flow is at CRITICAL DEPTH, and it is the state at which the SPECIFIC ENERGY of flow is at a minimum for a given unit discharge. This state occurs when the inertial and gravitational forces are balanced.

CROSS-SLOPE is generally defined as the transverse slope of the pavement of a street, measured at a 90° angle to the direction of traffic flow.

CULVERT is a short, CLOSED CONDUIT employed for the purpose of passing surface runoff under an embankment. A rectangular or square concrete structure for passing such runoff is referred to as a box CULVERT.

CURB is a concrete barrier, usually six to eight inches in height, typically placed parallel to and at the edge of street pavement.

CURB INLET is a STORM-DRAIN inlet consisting of an opening in a vertical CURB, in combination with an underground CATCH BASIN, which allows the entrance of stormwater runoff into the STORM-DRAIN SYSTEM.

CURVE NUMBER (CN) is an empirical hydrologic parameter used for predicting direct runoff and infiltration from rainfall excess. The runoff CN is based on the area's HSG, land use(s), treatment, and hydrologic condition (e.g., percent of vegetative cover).

DETENTION BASIN is a type of flood-control system that employs a reservoir as a means of delaying the downstream progress of floodwaters in a controlled manner. This is generally accomplished through the combined use of temporary storage areas and a metered outlet device (such as a WEIR or orifice) that reduces downstream FLOOD peaks, and thereby causes a lengthening of the duration of flow.

DEVELOPMENT means any man-made change to improved or unimproved real estate, including but not limited to buildings or other STRUCTURES, mining, dredging, filling, grading, paving, excavation, or drilling.

DIP is a depression or vertical SAG in the roadway designed to allow drainage to cross "at-grade" without using CULVERTS (also see AT-GRADE CROSSING).

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DISPERSED SURFACE FLOW is characterized by wide, shallow, "SHEETFLOW" runoff conditions, usually found in areas where no CHANNELS or well-defined DRAINAGEWAYS exist to serve as the primary runoff-conveyance systems.

DRAINAGE BASIN means any contributing WATERSHED or runoff catchment area.

DRAINAGE CRITERIA MANUAL (DCM) is this document, containing the hydrologic and hydraulic guidance and criteria that is to be used for analysis and design of drainage systems located within the Town of Oro Valley.

DRAINAGE REPORT is a report that is required for any project site which is greater than one acre in size, or for any project site that is subject to stormwater DETENTION requirements. The DRAINAGE REPORT shall contain all elements of a HYDROLOGY STUDY, as well as the appropriate components to adequately describe the hydrologic and hydraulic elements of any required stormwater DETENTION facility design. In addition, a DRAINAGE REPORT shall be required for any site where extensive structural improvements for mitigating drainage impacts are required.

DRAINAGE STATEMENT is a document containing a brief description of drainage conditions applicable for a project site that is not affected by 1% AEP flows of 100 cfs, or more; and which is neither subject to stormwater DETENTION requirements nor impacted by flows from a significant WATERCOURSE. The objective of a DRAINAGE STATEMENT is to demonstrate adequate site drainage exists, and to establish FINISHED-FLOOR ELEVATIONS which assure that all STRUCTURES are free from flooding during a 1% AEP FLOOD.

DRAINAGEWAY is a well-defined flow path or WATERCOURSE along which storm runoff moves, or may move, to drain a catchment area.

EASEMENT CURVES, when used in the context of open-CHANNEL design, refer to the alignment transition curves that have a relatively large radius of curvature, and are located between a straight reach of a CHANNEL and a circular curve having a smaller radius of curvature.

ENCROACHMENT, EQUAL DEGREE OF means the advancement or infringement of land uses, fill, or STRUCTURES onto the FLOOD PLAIN in a manner which reduces the flow capacity of the CHANNEL and/or FLOOD PLAIN of a WATERCOURSE. An equal degree of ENCROACHMENT is a standard applied to the evaluation of the effects of DEVELOPMENT upon increases in FLOOD heights. This standard assumes that if a DEVELOPMENT is permitted to encroach onto a FLOOD PLAIN, the approval to do so confers upon all property owners on both sides of the WATERCOURSE an equal right to encroach to the same hydraulic degree within that reach. Since the factors affecting hydraulic efficiency are usually not uniform within a reach, this usually will not result in equal distances between the FLOODWAY limits and the sides of the WATERCOURSE.

ENERGY GRADE LINE (EGL) is the elevation line which represents the total unit energy of flowing water. Points on the EGL are located above the water-surface elevation a distance equal to the VELOCITY HEAD, plus the PRESSURE HEAD where applicable.

ENGINEER means a person who, by reason of special knowledge of the mathematical and physical sciences and the principles and methods of engineering analysis and design acquired by professional education and practical experience, is qualified to practice engineering as attested by his or her registration in the State of Arizona as a Professional Engineer.

ENGINEER-OF-RECORD means the ENGINEER who seals drawings, reports, or documents for a project. The seal shall acknowledge that the ENGINEER prepared, coordinated, or had subordinates prepare, under the direct supervision of the ENGINEER, drawings, reports, or documents for a project.

EROSION in the context of fluid hydraulics refers to the removal and transport of soil particles by flowing water.

FEMA FLOOD INSURANCE STUDIES are a compilation and presentation of flood-risk data for specific WATERCOURSES, lakes, and coastal hazard areas within a community. When a FLOOD study is completed for FEMA, under the National Flood Insurance Program (NFIP), the information and maps are assembled into a Flood Insurance Study (FIS).

FINISHED-FLOOR ELEVATION (FFE) is the top of the structural slab at its elevation above mean sea level.

FLOOD means a temporary rise in the flow or the stage of any CHANNEL, stream, WASH, or WATERCOURSE which results in stormwaters overtopping the adjoining banks and inundating adjacent areas.

FLOOD HYDROGRAPH is typically a graphical depiction of the flow discharge (vertical axis) versus flow duration (horizontal axis). The scale on the horizontal axis is usually in minutes, hours, or sometimes days; and the scale on the vertical axis is usually in cubic feet per second (cfs). A FLOOD HYDROGRAPH can also be presented in a tabular format.

FLOOD INSURANCE RATE MAP (FIRM) is a map prepared by the Federal Emergency Management Agency (FEMA) which represents the official map of a community on which FEMA has delineated both the special hazard areas and the risk premium zones applicable to the community.

FLOODPLAIN (or FLOOD PLAIN) means areas of land adjoining or near the CHANNEL of a WATERCOURSE which have been, or may be, covered by floodwaters.

FLOODWAY is a regulated area in and along a WATERCOURSE which allows passage of a REGULATORY FLOOD to occur without increasing FLOOD elevations by more than 1.0 foot after a hypothetical ENCROACHMENT has been made into the FLOODWAY FRINGE.

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FLOODWAY FRINGE is that portion of the REGULATORY FLOOD PLAIN that lies outside of the FLOODWAY but within the 1% AEP FLOOD PLAIN.

FLOW-THROUGH WALL OPENING (also referred to as a WEEP HOLE) is a relatively small wall opening placed at the bottom, or base, of perimeter walls. This type of opening is used both to accept runoff onto and/or release runoff out of DEVELOPMENTS enclosed by solid walls. A FLOW-THROUGH WALL OPENING is normally located in a surface depression such that the existing drainage patterns, both entering and leaving the developed parcel, can be maintained without significant ponding and/or without concentrating runoff.

FREEBOARD is the additional vertical distance that exists between the calculated maximum level of the design water surface within a CLOSED CONDUIT, CULVERT, reservoir, tank, DETENTION/RETENTION BASIN, CHANNEL, or canal, and the top of the confining STRUCTURE. FREEBOARD is provided to account for hydrologic and hydraulic uncertainties, waves or other movements of the water surface, and sediment deposition such that flow will not overtop such confining STRUCTURES. The term is also used when referring to the vertical distance from the calculated, maximum water level in a DETENTION/RETENTION BASIN, CHANNEL, or WASH, to the base of any man-made STRUCTURE, such as the minimum finished floor of a building.

FRICTION (HEAD) LOSSES are losses in the unit energy of flowing water attributable to friction between the flowing water and the perimeter of a CLOSED CONDUIT or open CHANNEL.

FRICTION SLOPE is the slope of the ENERGY GRADE LINE, if MINOR LOSSES are ignored.

FROUDE NUMBER is a dimensionless ratio used in hydraulic design which defines the relationship between inertial forces and gravitational forces of flowing water. Typically, a FROUDE NUMBER greater than 1.0 indicates SUPERCRITICAL FLOW conditions whereby flow depths are controlled by upstream hydraulic conditions. Similarly, a FROUDE NUMBER less than 1.0 indicates SUBCRITICAL FLOW conditions, whereby flow conditions are controlled by downstream hydraulic conditions. A FROUDE NUMBER equal to 1.0 occurs at CRITICAL DEPTH, when the total energy of the flow is at a minimum.

GRATE INLET is a pavement inlet, normally consisting of an iron or steel grate set flush with the pavement or GUTTER, in combination with an underground CATCH BASIN, which allows the entrance of stormwater runoff into the STORM-DRAIN SYSTEM.

GUTTER is the low area adjacent to the CURB of a crowned street, and is used for conveying stormwater runoff.

HEADCUTS are vertical, or nearly vertical, drops that typically occur along the profiles of earthen CHANNELS. HEADCUTS normally move in an upstream direction as a result of EROSION.

HEC-RAS is a program designed to verify the validity of an assortment of hydraulic parameters found in the United States Army Corps of Engineers' HEC-RAS hydraulic modeling program.

HYDRAULIC GRADE LINE (HGL) is a line which represents the static head plus PRESSURE HEAD of flowing water.

HYDRAULIC JUMP is an abrupt rise in the water surface which typically occurs in an open CHANNEL when water flowing under a state of SUPERCRITICAL FLOW is suddenly forced to flow under a state of SUBCRITICAL FLOW.

HYDROLOGIC SOIL GROUP (HSG) comprises a group of soils having similar runoff potential under similar storm and cover conditions. The NRCS classifies HSGs in order to determine a soil's associated runoff CN. A CN is used to estimate direct runoff from rainfall. The 4 types of HSGs assigned by NRCS soil scientists are HSG A, HSG B, HSG C, and HSG D.

HYDROLOGIST, for purposes of this DCM, means a person who is skilled in, professes, and deals with the properties, distribution, circulation, and methods of analysis and design of surface water on the earth's surface and water in the atmosphere, acquired by professional education and practical experience.

HYDROLOGY STUDY is a study required for DEVELOPMENTS that are not subject to stormwater DETENTION requirements, nor that require extensive structural improvements in order to accommodate drainage; but that are impacted by flows emanating from significant WATERCOURSES and/or affected by 1% AEP peak flows (i.e., 100-year peak discharges) of 100 cfs, or more. The primary objective of a HYDROLOGY STUDY is to establish FINISHED-FLOOR ELEVATIONS which assure that all STRUCTURES are free from flooding during a regulatory (1% AEP) FLOOD. Additional objectives of a HYDROLOGY STUDY are (1) to establish the size and configuration of FLOW-THROUGH WALL OPENINGS; (2) to accommodate other minor drainage features; and, if required, (3) to provide supplemental information in the preparation of a grading plan in order to demonstrate that adequate site drainage shall be provided.

IMPROVEMENT PLANS are plans typically required during the course of regulatory review of DEVELOPMENT applications, subdivisions, conditional use permits, design review and building permits. Such activities include, but are not limited to: construction, grading, infrastructure, buildings, major additions, and site improvements proposed on a parcel.

INVERT is the floor, bottom, or lowest portion of the internal cross-section of a CLOSED CONDUIT.

KEY-INS refer to the extensions of BANK PROTECTION either (1) below the surface of the ground at the top of the constructed or existing bank; or (2) at the upstream and downstream limits of a bank-protected reach.

LATERAL MIGRATION is the horizontal movement of a CHANNEL or a WATERCOURSE over time due to bank EROSION that is caused by the hydraulic characteristics of flowing water; for example, sediment-transport discontinuities and alignment curvature.

LETTER OF MAP REVISION (LOMR) is a document from FEMA which describes changes to effective FIRMs. The LOMR gives a detailed description of the BFE and graphic changes that will be

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made to the SFHA currently delineated on the effective FIRM and/or FLOOD HAZARD BOUNDARY MAP (FHBM). FEMA will then revise the effective Flood Insurance Study (FIS) to reflect the new information which shows the original FIS to be incorrect, such as physical changes which invalidate the original FIS analyses or presentation of data. Updated or corrected topographic mapping, hydrologic data, or hydraulic data constitutes new information which may warrant a revision. Flood-protection projects and any form of topographic alterations (e.g., cut and fill) constitute physical changes which may also warrant a map revision. The map-revision process cannot be initiated without the community's endorsement, since it is the community that adopts the effective FIS. Therefore, any individuals requesting a change to the FIS must first do so by processing the request through the community. The community, in turn, must support the request, via its approval, and forward the information to FEMA for evaluation and acceptance.

LEVEE is an embankment of compacted soil, often covered with an impermeable veneer, which is built to redirect or impede the flow of floodwaters.

LOCAL DETENTION/RETENTION BASIN is a relatively small-scale stormwater storage facility which is owned, built, and maintained by developers, or their assigns, in accordance with specific standards for the purpose of satisfying local floodplain regulations.

MAJOR WATERCOURSE or MAJOR WASH is any WATERCOURSE which has a contributing drainage area of less than 30 square miles and a 1% AEP peak discharge of 3,000 cubic feet per second (cfs), or greater.

MANNING ROUGHNESS COEFFICIENT (also called “n-value”) is a coefficient which represents the roughness, or friction, applied to the flow by the materials comprising, or the materials found along, the bed and banks of a CHANNEL/WATERCOURSE.

MAXIMUM PREDICTED SCOUR DEPTH is the maximum vertical lowering of a CHANNEL bed predicted to occur during a particular flow event or over a specified period of time due to sediment movement and the erosive process of the flowing water.

MINOR (HEAD) LOSSES are losses in energy of flowing water not attributable to FRICTION LOSSES (e.g., expansion losses, contraction losses, bend losses, etc.)

MINOR WATERCOURSE or MINOR WASH is a WATERCOURSE which has a 1% AEP peak discharge of less than 3,000 cfs, but more than 100 cfs.

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM (NPDES) requires industrial and municipal sources of pollution to hold permits before pollutants can be discharged into navigable waters (waters used for commerce and travel). NPDES permits are issued by authorized state agencies or the EPA.

NON-REGULATORY WATERCOURSE is a WATERCOURSE with a 1% AEP discharge less than 100 cfs which is located within the Town of Oro Valley and is not subject to local, state, or federal floodplain regulations.

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NORMAL FLOW is open-CHANNEL flow under uniform conditions of depth, discharge, slope, and CHANNEL cross section. Under NORMAL FLOW, the ENERGY GRADE LINE is parallel to the slope of the CHANNEL or CLOSED CONDUIT.

NOTICE OF INTENT (NOI) for a general permit is similar to a permit application, in that it is notification to the regulatory authority of a planned discharge for which coverage under a specific NPDES general permit is needed.

NOTICE OF TERMINATION (NOT) is a document which constitutes notice that the party identified is no longer authorized to discharge stormwater associated with a specified activity under the NPDES program for the facility identified.

OBSTRUCTION is any physical alteration in, along, across, or projecting into any CHANNEL, WATERCOURSE, stream, lake, or REGULATORY FLOOD PLAIN which may impede or divert floodwaters, either in itself or by catching or collecting debris carried by such floodwaters, or that is placed where a flow of water might carry the same downstream to the damage of life or property. Examples include, but are not limited to, the following: any dam, wall, embankment, LEVEE, dike, pile, abutment, projection, excavation, CHANNEL rectification, bridge, CLOSED CONDUIT, CULVERT, building, wire, fence, rock, gravel, refuse, fill, STRUCTURE or vegetation.

ONE-PERCENT ANNUAL EXCEEDANCE PROBABILITY FLOOD (1% AEP FLOOD) is a flood stage or height which, statistically, has a one-percent (1%) chance of being equaled or exceeded in ANY given year (often called a 100-year flood). Also called the BASE FLOOD.

ONE-PERCENT (1%) AEP FLOOD ELEVATION is the water-surface elevation of the 1% AEP FLOOD. For WATERCOURSES where SUPERCRITICAL FLOW velocities are encountered, the CRITICAL DEPTH of flow shall be used to establish the 1% AEP FLOOD ELEVATION (i.e., the BFE), rather than using the lower, SUPERCRITICAL FLOW water-surface elevation.

OVERNIGHT PARKING is defined as leaving a motor vehicle unattended during the hours from sunset to sunrise.

PARKING AREA ACCESS LANE is the area providing access to vehicular parking spaces. Sometimes this term is abbreviated as P.A.A.L. or PAAL.

PARKING LOT is an area devoted to more than four off-street parking spaces. The expression PARKING LOT is synonymous with the term PARKING AREA.

PAVEMENT INLET is an opening in the street, GUTTER, or CURB made for the purpose of removing water from the street cross-section.

PRESSURE FLOW is the flow of water within a CLOSED CONDUIT without a free surface open to atmospheric pressure.

PRESSURE HEAD is equal to water pressure, at a specific point, divided by the specific weight of water. PRESSURE HEAD is usually expressed in units of length.

REGIONAL STORMWATER DETENTION BASIN/FACILITY is a basin/facility that collects stormwater runoff from a relatively large area, and has been designed to use temporary storage as a means of reducing downstream flood peaks, reducing possible flood damage, or reducing downstream CHANNEL construction costs. Regional facilities are usually multi-purpose, and their operation and maintenance are normally the responsibility of either the Town of Oro Valley or the Pima County Regional Flood Control District.

REGIONAL REGRESSION EQUATIONS (RREs) are equations developed by the USGS that are based upon flood-frequency data obtained from multiple streamgages located within a specific region encompassing the area of interest. The RREs are used to estimate peak-discharge values for flood-recurrence intervals ranging from 50% AEP to 0.2% AEP floods. Currently, the Town of Oro Valley is located with hydrologic “Region 5” of Arizona (Paretti et al., 2014).

REGIONAL WATERCOURSE is a large, intermittent stream which has a contributing drainage area of 30 square miles, or greater.

REGULATORY FLOOD is a 1% AEP FLOOD with a peak discharge of 100 cubic feet per second (cfs), or greater, and which has a one-percent (1%) chance of being equaled or exceeded in any given year (i.e., a 1% AEP).

REGULATORY FLOOD ELEVATION (RFE) is the water-surface elevation of a REGULATORY FLOOD at a specific location along a REGULATORY WATERCOURSE. The REGULATORY FLOOD ELEVATION is located 1 foot above the BFE, as established by the state of Arizona for Floodplain Management purposes.

REGULATORY FLOOD PLAIN is any portion of a flood plain, as well as any areas which are subject to SHEET FLOODING, that would be inundated by a REGULATORY FLOOD.

REGULATORY FLOODWAY is a regulated area in and along a WATERCOURSE which allows passage of a REGULATORY FLOOD to occur without increasing FLOOD elevations by more than 1.0 foot after a hypothetical ENCROACHMENT has been made into the FLOODWAY FRINGE.

REGULATORY WASH is any WASH or WATERCOURSE within the Town of Oro Valley that has a 1% AEP peak discharge equal to, or in excess of, 100 cfs.

REGULATORY WATERCOURSE is any WATERCOURSE with a 1% AEP discharge greater than 100 cfs which is located within the Town of Oro Valley and is subject to local, state, or federal floodplain regulations.

RILL EROSION is a pattern of narrow, vertical troughs formed in relatively steep earthen embankments by floodwaters cascading down the embankment.

RIPRAP is mechanically-placed or hand-placed rock or other material used to armor streambeds, bridge abutments, pilings, and other hydraulic structures against scour and water waves that would otherwise create soil erosion.

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RISE TIME is the time from the initial rise of storm runoff to the time of occurrence of the peak discharge on a FLOOD HYDROGRAPH.

SAG is a specified low point, sometimes found within a street profile, where stormwater runoff water is expected to collect.

SECTION 404 PERMIT is a permit required under Section 404 of the Clean Water Act (CWA), which is a program that establishes a program to regulate the discharge of dredged or fill material into “WATERS OF THE U.S.,” including wetlands. Proposed activities are regulated through a SECTION 404 PERMIT review process.

SEDIMENTS are particulates that are carried by water and deposited along a WATERCOURSE or on the bottom of a body of water. For purposes of this DCM, SEDIMENTS refer specifically to sands, gravels, cobbles, and boulders normally found in the bed of an ALLUVIAL CHANNEL or WATERCOURSE (i.e., BED LOAD and BED-MATERIAL SUSPENDED LOAD).

SEDIMENT TRANSPORT is the movement of solid particles (SEDIMENT), typically due to a combination of gravity acting on the SEDIMENT, and/or the movement of the fluid in which the SEDIMENT is entrained. When used in this DCM, the term SEDIMENT TRANSPORT refers strictly to BED-MATERIAL transport of SEDIMENTS.

SETBACK is 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 1% AEP water-surface elevation, whichever is closer to the CHANNEL centerline.

SHEET FLOODING is a condition which occurs within those areas which are subject to flooding typically ranging from one foot to two feet in depth, more or less, during the occurrence of a REGULATORY FLOOD; and where a clearly-defined CHANNEL does not exist so that the path of the flooding is often unpredictable and indeterminate.

SHEETFLOW is shallow, diffuse runoff such as would be produced from rainfall on a large, flat surface. It is characterized by an approximately equal depth of runoff across a broad width of flow.

SLOTTED INLET is a pavement inlet consisting of a long, narrow slot, typically two to four inches in width, and usually welded to the SOFFIT of a corrugated metal pipe.

SOFFIT is the highest point within the cross section of a CLOSED CONDUIT.

SPECIAL FLOOD HAZARD AREA (SFHA) is the land area covered by the floodwaters of the 1% AEP FLOOD (i.e., the BASE FLOOD), as depicted on NFIP maps. The SFHA is the area where the NFIP's floodplain-management regulations must be enforced, and is also the area where the mandatory purchase of flood insurance applies.

SPECIFIC ENERGY (SPECIFIC HEAD) is the energy per pound of water at any section of a CHANNEL measured with respect to the CHANNEL bottom.

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STORM DRAIN (or STORM-DRAIN SYSTEM) is a combination of underground CLOSED CONDUITS and surface-inlet STRUCTURES constructed for the purpose of removing runoff from the ground surface, usually from street pavement, and conveying it to some downstream discharge point.

STORMWATER POLLUTION PREVENTION PLAN (SWPPP) is a plan which identifies opportunities to reduce or eliminate generation of hazardous waste and the use of toxic chemicals at the source, rather than treating or controlling these materials after they have been created or used.

STORMWATER UTILITY FEE is a fee imposed by the Town of Oro Valley Stormwater Utility Department on DEVELOPMENTS that degrade water quality due to the implicit creation of pollutants in stormwater runoff by anthropogenic sources. The greater the impact of a particular DEVELOPMENT on water quality, the higher is the fee imposed.

STRUCTURE is anything constructed or erected, the use of which requires either its location on the ground or its attachment to some foundation having a location on the ground.

SUBCRITICAL FLOW is tranquil flow that has a FROUDE NUMBER less than 1.0, and where gravitational forces are dominant over inertial forces. SUBCRITICAL FLOW is controlled by downstream conditions.

SUMP is a specified low point, sometimes found within a street profile, where stormwater runoff water is expected to collect. SUMP is synonymous with SAG.

SUPERCRITICAL FLOW is rapid flow that has a FROUDE NUMBER greater than 1.0, and where inertial forces are dominant over gravitational forces. SUPERCRITICAL FLOW rarely occurs in natural ALLUVIAL WATERCOURSES or within CHANNELS with natural stream bottoms. SUPERCRITICAL FLOW is controlled by upstream conditions.

SUSPENDED LOAD refers to the BED-MATERIAL SEDIMENTS suspended in the stormwater runoff during the process of sediment transport. BED-MATERIAL SEDIMENTS in the SUSPENDED LOAD are kept in suspension by flow turbulence. The SUSPENDED LOAD generally consists of smaller particles, such as silts and sands, larger than 0.0625 mm in size.

TAILWATER is the flow condition encountered at the downstream end of any hydraulic STRUCTURE, or hydraulic condition, under investigation.

TIME OF CONCENTRATION is the time required for storm runoff to flow from the hydraulically most remote point of a catchment or drainage area to the outlet or point under consideration.

TOEDOWN is the vertical extension of BANK PROTECTION that extends below the CHANNEL bed in order to prevent scour from undermining the protection on the CHANNEL sides.

TRANSITIONS are longitudinal sections of a CHANNEL within which the flow width is expanded or contracted in a predetermined manner.

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VELOCITY HEAD is the kinetic energy per pound of flowing water.

WASH refers to a natural WATERCOURSE that has not been disturbed to any significant extent by anthropogenic DEVELOPMENT, and therefore the native vegetation is still present.

WASH LOAD is comprised of very fine sediments with rates of transport that are not related to the flow rate. The very fine sediments that represent the wash load in alluvial channels are generally smaller than 0.0625 mm in size.

WATERCOURSE is any naturally occurring lake, river, stream, creek, WASH, arroyo, or other body of water or CHANNEL having banks and bed through which waters flow at least periodically and any depression serving to give direction to a current of storm water, provided that it shall also include other designated, naturally occurring areas where substantial FLOOD damage may occur.

WATERS OF THE U.S. means, in part, (1) all waters which are currently used, or were used in the past, or may be susceptible to use in interstate or foreign commerce, including all waters which are subject to the ebb and flow of the tide; (2) all interstate waters including interstate wetlands; and (3) all other waters such as intrastate lakes, rivers, streams (including intermittent streams), mudflats, sandflats, wetlands, sloughs, prairie potholes, wet meadows, playa lakes, or natural ponds, the use, degradation or destruction of which could affect interstate or foreign commerce, including any such waters.

WATERSHED BASIN FACTOR (n_b) is a hydrologic parameter used in the TIME OF CONCENTRATION formula applicable to the C1FHP in order to characterize WATERSHED-wide resistance to flow along upland, intermediate, and principal flow paths within a WATERSHED.

WEIR (BROAD-CRESTED) is an open-CHANNEL control section, with a horizontal crest above which fluid pressure may be considered hydrostatic.

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List of Symbols

(Note: Some symbols are used more than once, in those instances, multiple definitions are provided. User should refer to in-text references for further clarification.)

A	=	Cross-sectional area of flow, in feet, numerical subscripts 1, 2, 3 and 4 indicate flow depth at specific location (refer to figure or equation for location).
	=	Drainage area of watershed, in acres or square miles.
α	=	The angle formed by projection of the centerline of a channel from its beginning point of curvature to a point that meets the line tangent to the outer bank of channel within the bend. The relationship between α and RC/T is mathematically described as $\frac{R_c}{T} = \frac{cos\alpha}{4sin^2\left(\frac{\alpha}{2}\right)}$.
α_c	=	Angle of confluence of two watercourses, in degrees.
ARF	=	Areal Reduction Factor, dimensionless.
b	=	Bottom width of channel or box culvert, in feet. Numerical subscripts 1, 2, 3 and 4 indicate flow depth at specific location (refer to figure or equation for location).
β	=	Wave front angle $= sin^{-1}\left(\frac{1}{F_r}\right)$, in degrees.
b_{pd}	=	Bridge pier width with debris (= physical width + 2 feet of debris width on each side), in feet.
b_{pe}	=	Effective bridge pier width, in feet.
C	=	Runoff coefficient, dimensionless.
	=	Water surface superelevation coefficient, dimensionless.
CAF	=	Contributing Area Factor, dimensionless.
C_c	=	Coefficient of contraction, dimensionless.
C_e	=	Coefficient of expansion, dimensionless.
C_{LTD}	=	Long-term degradation coefficient, dimensionless.
CN	=	Curve Number, dimensionless.
CN_*	=	Curve Number for a thunderstorm less than 3 hrs in duration, dimensionless.

CN_{24}	=	Curve Number for a 24-hr General Summer Storm, dimensionless.
C_{nuf}	=	Coefficient to account for nonuniform flow (i.e., flow irregularities), dimensionless. This coefficient is meant to account for increased unit discharge due to flow irregularities.
$cos\varphi$	=	the angle of the embankment side-slopes, in degrees.
C_{rw}	=	Weighted creep ratio, dimensionless.
C_{TSS}	=	Long-term stable-slope time coefficient, dimensionless.
C_w	=	Watershed-weighted runoff coefficient for the design AEP flood, dimensionless.
$C_{w1\%}$	=	Watershed-weighted runoff coefficient for the 1% AEP flood, dimensionless.
D	=	Diameter of culvert, in feet.
D_{50}	=	Median (50%) rock size of riprap, in feet.
		Median (50%) size of streambed sediments, in millimeters.
D_{50}^*	=	Corrected median (50%) sediment size to account for differences in unit weight and side-slopes long a riprapped channel bank.
D_a	=	Armoring particle size, in millimeters.
D_o	=	Diameter of pipe culvert, in feet.
FB	=	Freeboard of channel, in feet.
F_r	=	Froude number, dimensionless.
F_{ru}	=	Upstream approach Froude number.
g	=	Gravitational constant = 32.2 feet/second^2.
G	=	Gradation Coefficient ($= 1/2\left[\frac{D_{50}}{D_{16}}\right] + 1/2\left[\frac{D_{84}}{D_{50}}\right]$).
γ_s	=	Unit weight of the riprap, in pounds/cubic foot.
H	=	Height of culvert, in feet.
	=	Drop height downstream of the peak grade-control structure, in feet.
h	=	Exposed height on downstream side of drop structure, in feet.

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h'	=	Vertical drop in the water surface through a junction, in feet.
ΔH	=	Change in elevation, in feet.
h_d	=	Vertical drop in the channel bottom through a junction, in feet.
H_s	=	Head on structure (headwater - tailwater), in feet.
H_T	=	Total drop in head (measured as the difference between upstream and downstream energy grade lines), in feet (normally, use the difference in WSELs, $Y_{1\%}$ - TW).
h_t	=	Transition losses, in feet.
h_{tc}	=	Transition losses in contracting reaches, in feet.
h_{te}	=	Transition losses in expanding reaches, in feet.
Δh_v	=	Difference in velocity head between the upstream and downstream ends of the transition, in feet.
H:V	=	Horizontal to vertical, dimensionless.
HW/D	=	Ratio of depth of headwater to culvert diameter, dimensionless.
HW/H	=	Ratio of depth of headwater to culvert height, dimensionless.
HW	=	Headwater depth, in feet.
i	=	Variable counter, usually denoted as a subscript, dimensionless.
$i_{1\%}$	=	Storm intensity at $T_{c1\%}$ during the 1% AEP flood over the watershed, inches/hour.
L	=	Length of channel, in feet.
		Length of hydraulic jump, in feet.
		Length of channel junction, in feet.
L'	=	Maximum distance of superelevation downstream of curvature in a channel with supercritical flow, in feet.
l	=	Sheetflow length, in feet.
	=	Length of the longest watercourse, in feet
ΔL	=	Change in length, in feet.

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L_c	=	Confluence length, in feet.
L_{ca}	=	Length from the watershed centroid to the watershed outlet, in feet.
L_{ct}	=	Total length of curve, from PC to PT, in feet.
L_{dc}	=	Estimated distance to downstream control, in feet.
L_e	=	Length of the easement curve, in feet.
	=	Encroachment length, in feet.
L_H	=	Horizontal, or flat, contact distance (less than 45°), in feet.
L_o	=	Distance between the point of tangency and the junction apex, in feet.
L_{pw}	=	Length of pier wall, in feet.
L_r	=	Reach length, or spacing, between adjacent grade-control strictures, in feet.
L_{SCUL}	=	Length of scour hole, in feet.
L_{TR}	=	Length of exiting transition section, in feet.
L_V	=	Vertical, or steep, contact distance (more than 45°), in feet.
M	=	Momentum of moving mass of water. Numerical subscripts 1, 2, 3 and 4 indicate flow depth at specific location (refer to figure or equation for location).
N	=	Number of circular culverts or box culverts, dimensionless.
n	=	Manning roughness coefficient, dimensionless.
	=	Basin Factor, dimensionless.
n_b	=	Watershed basin factor, dimensionless.
$n_{b1\%}$	=	Reach weighted basin factor for the watercourse for the 1% AEP peak discharge, dimensionless.
$n_{b(min)}$	=	Minimum basin factor, dimensionless.
n_{bw}	=	Reach weighted basin factor for the watercourse, dimensionless.
n_{min}	=	Minimum Manning roughness coefficient, dimensionless.
N_r	=	Number of reach segments, dimensionless.

n_{sf}	=	Manning roughness coefficient for sheetflow, dimensionless.
$P_{1\%}^{1-hr}$	=	Precipitation depth of a 1-hour duration storm event for a 1% AEP flood (either the point or areally-reduced), in inches.
$P_{1\%}^{24-hr}$	=	Precipitation depth of a 24-hour duration storm event for a 1% AEP flood (either the point or areally-reduced), in inches.
$P_{3\text{-}hr}$	=	The 3-hr rainfall depth, in inches.
PC	=	Point of Curvature.
P_c	=	Percent (%) of the material which is coarser than the armoring size.
P_{h1}	=	Hydrostatic pressure on Section 1, in pounds.
P_{h2}	=	Hydrostatic pressure on Section 2, in pounds.
P_{hf}	=	Retardation force of friction.
P_{hi}	=	Horizontal component of hydrostatic pressure on the channel invert.
P_{hw}	=	Axial component of hydrostatic pressure on the channel walls.
PI	=	Point of Intersection.
P_n	=	The n-hour rainfall depth for the design AEP flood, in inches.
P_o	=	Estimated time period over which streambed degradation will occur (i.e., a design life of typically 100 years), in years.
PT	=	Point of Tangency.
φ_p	=	Angle of approach flow to pier wall, in degrees.
q	=	Unit discharge of flow in channel, cfs/square foot.
$q_{1\%}$	=	Peak runoff-supply rate of the watershed for the 1% AEP flood.
Q	=	Channel discharge, in cubic feet per second. Numerical subscripts 1, 2, 3 and 4 indicate flow depth at specific location (refer to figure or equation for location).
$Q_{P1\%}$	=	1% AEP peak discharge, in cfs.

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$Q_{P10\%}$	=	10% AEP peak discharge, in cfs.
$Q_{Pu,10\%}$	=	10% AEP flood peak discharge for *urbanized* conditions, in cfs.
$Q_{Pn,10\%}$	=	10% AEP flood peak discharge for *natural* conditions, in cfs.
Q_{pe}	=	Effective flood-peak discharge, in cfs (may include overbank flows, where applicable).
Q_p	=	Peak discharge of the design AEP flood, in cubic feet per second (typically the 1% AEP peak discharge).
$q_{P1\%}$	=	Average peak discharge per unit width of channel during a 1% AEP flood, in cfs/foot.
Q_{sp}	=	Bed-material sediment-transport rate (unbulked), in cfs, at the flood-peak discharge.
R	=	Ratio of the n% AEP to the 1% AEP peak discharge, dimensionless.
	=	Hydraulic radius of flow (= A/WP), in feet.
R_c	=	Radius of curvature of channel centerline, in feet.
R_s	=	Sediment reduction factor for upstream sediment supply (i.e., typically the ratio of predicted long-term impervious area to total area. The value of R_s varies from 0.0 to 1.0.
SB	=	Erosion-hazard setback limit, in feet.
S_c	=	Channel/watercourse slope, in ft./ft.
	=	General slope, in feet/feet.
S_{eq}	=	Equilibrium channel slope, in feet per foot.
S, S_f	=	Friction slope, in feet per foot. Numerical subscripts 1, 2, 3 and 4 indicate flow depth at specific location (refer to figure or equation for location).
S_{ib}	=	Initial channel bed slope, in feet per foot.
S_n	=	Natural channel slope, in feet per foot.
S_o	=	Slope of the overland surface, in feet/foot.
Σ	=	Greek letter Sigma, used as an operator for summation.
t	=	Time, in minutes.
T	=	Top width of channel, in feet.

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ΔT	=	Change in top width, in feet.
T_c	=	Time of concentration, in minutes.
$T_{c1\%}$	=	Time of concentration for the 1% AEP storm, in minutes.
$T_{c10\%}$	=	Time of concentration for the 10% AEP storm, in minutes.
T_{cf}	=	Travel time for channel flow, in minutes.
$T_{cn\%}$	=	Time of concentration for the n% AEP storm, in minutes.
TCL	=	Total Creep Length, in feet.
T_r	=	Rise time of flood Hydrograph, in minutes.
T_{sf}	=	Travel time for Sheetflow, in minutes.
T_{scf}	=	Travel time for shallow concentrated flow, in minutes.
T_{SS}	=	Estimated time to achieve a stable slope (typically less than 100 years), in years.
TW	=	Tailwater depth (downstream depth of flow) during a 1% AEP flood, in feet.
θ	=	Central angle of the cross-wave pattern, in degrees.
	=	Transition angle, in degrees.
	=	Junction angle, in degrees.
θ_a	=	Slope angle of encroachment face (measured from horizontal), in degrees.
v	=	Accumulated runoff volume under the flood hydrograph, in acre-feet.
V	=	Total runoff volume under the flood hydrograph, in acre-feet.
	=	Flow velocity, in feet/sec. Numerical subscripts 1, 2, 3 and 4 indicate flow depth at specific location (refer to figure or equation for location).
$V_{1\%}$	=	Average velocity within the main channel during a 1% AEP flood, in feet/second.
V_c	=	Critical velocity of flow, in feet per second.
V_{min}	=	Minimum velocity required to suspend all bed-material sediments in the vertical flow profile of a concrete-lined channel or a smooth-lined culvert, in feet per second.

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V_p = Velocity of flow at peak discharge, in feet per second.

$\frac{v^2}{2g}$ = Velocity head of flow, in feet.

V_s = Bulked bed-material sediment volume, in cubic feet (based upon an assumed bulked weight of 100 pounds/cubic foot).

W_{SCUL} = Width of scour hole, in feet.

$W_{1\%}$ = Flow width of the main channel during a 1% AEP flood, in feet.

$W_{10\%}$ = Width of channel conveying the 10% AEP peak discharge (i.e., the so-called “dominant discharge”), in feet.

W_e = Effective flow width at peak, in feet (may include overbank flows, where applicable).

WP = Wetted perimeter of flow, in feet.

X_B = Distance from the end point of a bend to the point where channel scour is no longer influenced by flow curvature, in feet.

X_{sce} = Downstream distance to maximum depth of local scour, in feet.

Y = Depth of flow, in feet. Numerical subscripts 1, 2, 3 and 4 indicate flow depth at specific location (refer to figure or equation for location).

Y_h = Hydraulic depth of flow (area/top width), in feet.

Y_c = Critical depth, in feet.

ΔY = Rise in water-surface elevation (superelevation) around the outside of a channel bend, in feet.

Y_{MAX} = Maximum depth of flow in the channel measured from the thalweg elevation, in feet.

= Maximum depth of flow during a 1% AEP flood immediately upstream of a bend, in feet.

Y_{MC} = Maximum flow depth in the confluence scour hole during a 1% AEP flood, in feet.

Y_{MS} = Average flow depth, from the water surface to the mean scour depth, during a 1% AEP flood, in feet.

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Yp	=	Depth of flow at peak discharge, in feet.
$Y_{P1\%}$	=	Average depth of flow at peak discharge of a 1% AEP flood, in feet.
$Y_{1\%}$	=	Depth of flow during a 1% AEP flood, in feet.
Z	=	Channel side-slope (horizontal to vertical), in feet/foot. Numerical subscripts 1, 2, and 3 indicate flow depth at specific location (refer to figure or equation for location).
Z_A	=	Bedform (i.e., anti-dune) Scour, in feet.
Z_B	=	Bend Scour, in feet.
Z_C	=	Confluence Scour, in feet.
Z_{CUL}	=	Local Scour due to the presence of a culvert, in feet.
Z_G	=	General Scour, in feet.
Z_L	=	Local Scour, in feet.
Z_{LB}	=	Local Scour due to the presence of bridge piers/abutments, in feet.
	=	Local Scour contribution due to bridge piers with a pier shape reduction factor of 1.0 included, in feet.
Z_{LD}	=	Local Scour contribution from a flow drop (i.e., a drop structure or grade-control) measured from thalweg downstream of control-point, in feet.
Z_{LE}	=	Local Scour due to the presence of encroachments, in feet.
Z_{LFT}	=	Low-Flow Thalweg Scour, in feet.
Z_{LTD}	=	Long-Term Degradation (or Aggradation) depth, in feet.
Z_{MAX}	=	Upper limit of maximum predicted scour depth, in feet.
Z_{TSE}	=	Total Single-Event scour depth during a 1% AEP flood, in feet.

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Chapter 1. Introduction

This Drainage Criteria Manual (DCM) contains procedures and criteria approved by the Town of Oro Valley (the Town) for the analysis of drainage conditions and the design of drainage infrastructure. The DCM is required to be used in conjunction with all projects submitted to the Town on or after _______, 2020. The DCM shall serve as the basis for the preparation and submittal of all Hydrologic and Hydraulic Reports and Drainage Statements, as well as the design of all drainage and stormwater management facilities and drainage infrastructure located within the Town.

When deemed appropriate by the Town, the use of other recognized procedures may be used upon submittal, review, and acceptance of the procedure(s), in writing, by the Town.

This DCM is intended to guide Engineers, Hydrologists, and other professionals in properly locating, sizing, and designing drainage and stormwater management facilities and drainage infrastructure located within the Town. The previous edition of the DCM has been amended herein in order to emphasize the impact of sediment transport on stormwater systems located within the Town; and, in particular, the need for the proper assessment of erosion, sedimentation, and scour issues, which are key factors to consider in the design and construction of sustainable drainage features. Specifically, this edition of the DCM now includes additional technical guidance regarding evaluation of bed-material sediment transport relative to both short-term and long-term sediment impacts requiring consideration when designing stormwater systems located within the Town.

Standards, accepted methodologies, and design references are identified throughout this DCM. When encountering issues not specifically addressed and/or referenced in the DCM, the user is to apply the appropriate standards, procedures, guidelines, and/or methodologies contained within the applicable Chapter(s) and Section(s) of the most current version of the City of Tucson's (COT) *Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona* (COTDSM), prepared by Simons, Li & Associates, Inc. (Simons, Li & Associates, Inc. [SLA], 1989/1998). Whenever reference is made in the DCM to the COTDSM, text has been included indicating the location within the COTDSM where the specific subject matter can be found. Thus, it is neither the intent of the Town nor the purpose of this DCM to duplicate all of the standards, procedures, guidelines, references, and/or methodologies noted in the COTDSM.

As of the date of publication of this DCM, the COTDSM can be obtained online at:

https://www.tucsonaz.gov/tdot/maps-records-downloads

The Town understands that by virtue of the natural environment within the Town, and because of Town Ordinances, Standards, Policies, etc., there may be, from time to time, a need to use methods and applications that better assess a particular site, and which may differ from the accepted design references presented in this DCM. The Town may accept these methods, provided prior written approval is granted by the Town. In order to gain pre-approval, documentation sufficient to satisfy the Town must be submitted to the Department of Public Works for consideration. A minimum of 30 days is required for review and formal reply.

Ultimately, the design professional shall be responsible for all information contained within any report, technical memorandum, plat, or plan submitted to the Town. All information submitted to the Town for review shall be substantiated by documented research and calculation. In this regard, it is emphasized that the Engineer-of-Record is solely responsible for the completeness and accuracy of all information contained within any report, technical memorandum, plat, or plan submitted to the Town, and that review and permitting by the Town is based solely upon its assessment of the conformance of any submittal with adopted regulations, approved standards, and the current standard-of-practice for engineering.

In cases where the DCM is in conflict with noted standards, accepted methodologies, and design references, this DCM shall govern unless otherwise accepted, in writing, by the Town.

This DCM will be updated and modified, as needed, in order to reflect additional experiences, new technologies, and/or changes in stormwater-related regulations and policies. It shall be the responsibility of the user to ensure that they are using the most current edition of the DCM, as well as all documents referenced herein.

Every effort was made to ensure this document is free of discrepancies and errors. However, if the user believes they have found a discrepancy or error, and for additional information, please contact:

Mr. Paul Keesler, P.E., Town Engineer
Town Engineer's Office
Town of Oro Valley
1100 N. La Cañada Drive
Oro Valley, AZ 85737
(520) 229-4800

Updated _______, 2020

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References

Simons, Li & Associates, Inc. (SLA) (1998). *Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona*. (Original work published 1989.)

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Chapter 2. Drainage Analysis and Design Requirements

2.1 Introduction and Purpose

The purpose of this Chapter is to establish the format and content required for Town review and acceptance of Drainage Reports and Drainage Statements which will serve as the basis for proposals regarding the manner in which drainage is to be addressed and accommodated as part of any new or modified development.

When accepted, Drainage Reports and Drainage Statements serve as a permanent record of the basis of design for drainage conveyance and flood-control infrastructure and elements. Drainage Reports and Drainage Statements also document the Town review of Floodplain Permit Applications and the Town findings that proposed improvements meet the minimum floodplain-management requirements contained in the Code of Federal Regulations, 44 CFR. § 60.3(d), and in the Oro Valley Town Code, Section 17-4-3.

2.2 Activities Requiring a Drainage Report or Drainage Statement

Activities requiring a Drainage Report or Drainage Statement include the following, unless otherwise directed or waived by the Town:

1. Changes or amendments to property zoning or Planned Area Developments.
2. Changes or amendments to the General Plan.
3. Building and Site Development, including subdivisions, commercial, industrial, and multi-family residential developments.
4. Grading Permits, including Type 1 and Type 3 Grading Permits as described in the Oro Valley Code, Section 27.9.D.
5. Minor land divisions of unsubdivided land.
6. Disclosure statements prepared in conjunction with a condominium conversion.
7. Construction within or adjacent to the 1% Annual Exceedance Probability (AEP) floodplain and erosion-hazard setback of a regulatory watercourse, including construction of block walls or solid fencing in these flood- and erosion-hazard areas.
8. Restarting construction activities on undeveloped or partially developed portions of an approved plan after there have been no clearly discernable construction activities for two or more years.
9. As directed by the Town's Floodplain Administrator.

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2.3 Report Types

In general, drainage design criteria and reporting requirements are based on the relative size and complexity of the development and planned drainage system. Smaller, simpler projects require less detail and analysis than larger projects, and thus require only a Drainage Statement.

2.3.1 Drainage Statement

Drainage Statements are typically appropriate for all new Single Family or Duplex residential developments not associated with a new or recent subdivision and that disturb an area of less than one acre, as they are usually not subject to the stormwater-detention and water-quality criteria found elsewhere in this DCM.

Exceptions to the above, which would require a Drainage Report, include but are not limited to sites equal to or greater than one (1) acre in size, or individual sites less than one acre in size if they are part of a larger common plan of development that is greater than one (1) acre in size, even if multiple, separate and distinct, land-development activities may take place at different times and on different schedules. Any developments appearing to be a part of a greater project shall be subject to stormwater-detention and water-quality criteria found in this DCM.

2.3.2 Drainage Report and Drainage Report Addenda

Drainage Reports and their addenda are typically required for larger development projects involving site grading and paving, subdivisions, and multi-family and commercial developments.

2.3.3 Floodplain Studies

Floodplain mapping studies for approval by the Federal Emergency Management Agency (FEMA) that are submitted to the Town for Community Acknowledgement shall be prepared in accordance with Arizona Department of Water Resources (ADWR) State Standard 1, titled *Instructions for Organizing and Submitting Technical Support Data Notebooks (TSDN) for Flood Studies* (ADWR, 2012) as well as in accordance with those standards found in FEMA's *Guidelines and Standards for Flood Risk Analysis and Mapping* (FEMA, 2020) or its current equivalent.

2.4 General Report Requirements

The requirements presented in this section shall be used to aid the Design Engineer or applicant in the preparation of Drainage Reports, Drainage Statements, and construction drawings for new development.

These requirements are the minimum necessary, and will be used to evaluate the adequacy of all drainage submittals to the Town. Submittals with incomplete or absent information shall result in the report being returned to the Engineer without review.

1. Stand-Alone Document. Drainage Reports and other drainage submittals (e.g., Drainage Statements) shall be stand-alone documents. When references are made to, or assumptions

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are based on, previously submitted studies or reports, the Drainage Report or drainage submittal must include the appropriate excerpts, pages, tables, and maps containing the referenced information. A previous approval does not guarantee that the previous report is correct. Standards change, and models must meet current standards. Therefore, assumptions made in previous reports must be verified, substantiated, and clearly reasserted in all new reports.

2. Required Use of Town Checklists. A Design Checklist or a Report Checklist as defined and referenced in the individual sections of this DCM, and as available from the Town, must be completed and submitted with the Drainage Report. Appropriate notations shall be provided with the checklist so as to assist the reviewer in determining whether the design is complete (e.g., if a specific item is not addressed, an explanation should be provided). All Design Checklists or Report Checklists will be available on the Town website. New and/or revised checklists will be added as they are developed.
3. Electronic Submissions Preferred. It is preferred that Drainage Reports, Drainage Statements, and related supporting information, be provided to the Town for review as an electronic format via e-mail or via a secure file-management system accepted by the Town. Reports, maps, and figures should be in Portable Document Format (PDF). Submission of hydrologic and hydraulic computations should consist of the digital files of the computer models used, and be fully functionable upon delivery.
4. Response to Town Review Comments. Town comments made during review, if any, must be briefly answered by the submitting Engineer in a comment-response letter, and then again more thoroughly in the resubmitted report and attachments.
5. Use of computer software. A variety of computer programs are available to facilitate hydrologic and hydraulic calculations. The use of publicly available, non-proprietary software is generally acceptable, provided it closely follows the computational procedures specified in this DCM, is well-documented, and the Town currently owns or can readily acquire a current copy for use in its review. The use of noncommercial proprietary software, including specialized Excel spreadsheets, is discouraged, and its acceptance will be on a case-by-case basis, as time permits, subject to receiving and reviewing a user’s guide and source codes.

2.5 Drainage Statement Format

A Drainage Statement typically consists of a short one- to three-page document, with additional supporting technical information. It is usually written for a development site which is not affected by 1% AEP peak flows equal to or greater than 100 cubic feet per second (cfs), and is neither subject to stormwater detention requirements, nor impacted by flows from a significant watercourse. The objectives are to demonstrate adequate site drainage, establish Finished-Floor Elevations (FFEs), and to demonstrate all planned structures are reasonably safe from flooding and erosion.

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Because site conditions vary considerably, each Drainage Statement may be different in content and format. The Engineer preparing the statement may exercise his or her own judgement in presenting the technical information for review. In all cases, the Drainage Statement must be clearly written, sealed, and signed by an Arizona Registered Professional Civil Engineer; and should contain the following information:

1. A brief description of the type and size of the proposed development, including a legal description of the parcel or parcels being developed.
2. A brief description of the amount of runoff expected on, or near, the site.
3. A map with background topography, or aerial imagery, or both, at the largest possible common scale (with scale-bar) depicting the following: the subject parcel, the contributing drainage areas and their principal points of drainage concentration, previously mapped flood plains, drainage easements, and any other pertinent information related to the site design.
4. Hydrologic calculation sheets for each principal point of drainage concentration.
5. The appropriate hydraulic calculation sheets used in designing the proposed method of drainage disposal.
6. A Site Plan shown at the at the largest possible common scale (with scale-bar), for review and acceptance.

2.6 Drainage Report Format

Content of the Drainage Report will vary with the complexity of the development and corresponding engineering investigation; however, the report format shall, at a minimum and as site conditions dictate, follow the outline below, with sections being removed or expanded as needed for clarity. It is highly recommended that calculations, sketches, and plan notes be included in the report in order to show the design process. If plan sheets are used in lieu of design sketches in the report, then clear cross-references in the report and on the plan sheets are needed so the reviewer can quickly follow the design/calculation process. Careful adherence to this format, along with the inclusion of cross-references between calculations and plans, will aid in plan review, and will serve to hasten the process. The recommended report format and content follows.

Cover:

a. Name, address, section, township and range, and the Pima County Tax ID Code for the parcel, project, or development for which the report is being submitted, as well as the Town of Oro Valley Activity Number, when it becomes available after the first submittal.
b. Name and address of the client/owner of the parcel or development.
c. Name and address of the engineering/consulting firm preparing the report.
d. Submittal date, including any subsequent resubmittals, if any.

e. Name of person preparing the report, if other than registrant sealing the report.

f. Seal and signature of an Arizona Registered Professional Civil Engineer, which meets Arizona State Board of Technical Registration specifications and requirements (and/or place seal on the Table of Contents).

Table of Contents:

Provide an organized summary table, with page numbers, of the sections, subsections, and appendices to be found in the report. Include a list of summary tables, figures, appendices, references, and computer models used. Include the seal and signature of an Arizona Registered Professional Civil Engineer.

1. Introduction:

The introduction to the report is intended to be a broad overview, and will give a brief description of the subject property, the planned development activities on the property, and the engineering analyses used in its design, as follows.

a. Project name.

b. Project location, including legal description and vicinity map.

c. Site description, including parcel size and major drainage features.

d. Purpose of the report, such as Site Development, Grading Plan, etc.

e. Objectives of the report, including specific analyses and recommendations to be given.

f. FEMA and locally-mapped flood plains on or near the subject property.

g. Known development requirements, such as zoning, critical-basin stormwater detention, water-harvesting, first-flush treatment, preparation of a Stormwater Pollution Prevention Plan (SWPPP), and avoidance of Waters of the United States (WUS).

h. Identify previous drainage and floodplain-mapping studies of the project site and surrounding area, including any Master Drainage Reports or basin-management plans pertaining to the subject property.

i. Identify any easements or right-of-way requirements envisioned.

2. General and Special Conditions:

Identify and briefly discuss any general or special conditions that may apply, as follows.

a. General Conditions

   i. List general conditions to be satisfied by the project, such as stormwater harvesting, stormwater routing, stormwater detention, and installing first-flush devices.

   ii. Refer to and provide a Section 404 Permit Compliance Statement. This form is to be notarized, unless signed/sealed by an Arizona Registered Professional Engineer.

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iii. Refer to and provide a copy of the Flood Insurance Rate Map (FIRM) of the subject property. Show the location of the subject property.

b. Special Conditions

i. Identify whether the project is in an effective FEMA Special Flood Hazard Area, and whether it will require a Conditional Letter of Map Revision (CLOMR) or Letter of Map Revision (LOMR).

ii. Identify and discuss any special conditions that may require deviation from standard procedures and submittal requirements presented within this DCM and/or in the COTDSM (SLA, 1989/1998), including a stormwater detention waiver.

iii. Identify any other special conditions which may require design exceptions, stipulations, and permit issues (e.g., National Pollutant Discharge Elimination System [NPDES], FEMA), etc. Examples of special permits include:

1. Arizona State Permits, such as an Arizona Pollutant Discharge Elimination System (AZPDES) Permit, a Notice of Intent (NOI) Permit, or a Notice of Termination (NOT) Permit.

2. Pima County Regional Flood Control District (PCRFCD) Floodplain Use Permits, typically required for larger watercourses with flows of 3,000 cfs or greater and which are maintained by the PCRFCD.

3. Existing-Conditions Drainage:

This section of the Drainage Report shall provide a brief description of the existing drainage conditions within and adjacent to the subject property, described as follows. This should be a descriptive “roadmap” of where runoff from specific onsite areas originates, where it is collected and moved through conveyances, and ultimately where it is released.

a. Onsite Drainage

i. Identify the existing drainage network, flow patterns, and watershed boundaries.

ii. Identify existing onsite regulatory floodplain boundaries, as identified by the Town (i.e., local floodplains) and/or FEMA (i.e., federal floodplains). Include an annotated copy of the FIRM panel, and/or the applicable portion of the special study from which the locally administered flood plains have been developed.

(Note: A locally administered regulatory floodplain is one that is associated with 1% AEP peak discharge equal to or greater than 100 cfs.)

b. Offsite Drainage

i. Describe existing drainage conditions, as well as the watercourses or channels entering or exiting the project site. Also identify any sheetflow conditions affecting the project site.

ii. Identify how offsite conditions are based on future, projected land uses as identified on available planning and zoning documents.

iii. Describe contributing offsite watersheds and onsite regulatory floodplain boundaries, both federally mapped (i.e., FEMA) and locally mapped

iv. Identify and discuss existing drainage studies for the subject and adjacent properties. If a drainage study is not available for an adjacent site, provide a brief qualitative description instead.

v. Describe the local geomorphology of the existing watercourse or sheetflow areas. Identify areas where there have been historical changes in channel alignment and bed profile, if any. Identify any areas, either upstream or downstream from the subject development, where there is visible channel downcutting, particularly downstream from roadways.

4. Proposed-Conditions Drainage Design

This section of the Drainage Report will give a brief description of the proposed-conditions drainage within and adjacent to the subject property, as follows. This should be a descriptive "roadmap" of where runoff from specific onsite areas originates, where it is collected and moved through conveyances, and ultimately released.

a. Modified and Constructed Channels

i. Briefly describe the planned channels; and where, in general, they are to be located.

ii. Briefly describe how adjoining properties will be protected from erosion due to potential lateral channel migration.

iii. Describe how the constructed or modified channels will be designed to accept and convey incoming bed-material sediment load, and how this design specifically accommodates the local geomorphology, including long-term aggradation and degradation potential.

b. Proposed Drainage Infrastructure

i. Briefly describe the planned drainage infrastructure; and where, in general, it is to be located.

c. Proposed Stormwater Detention Facilities

i. Briefly describe the planned stormwater detention facilities; and where, in general, they are to be located.

5. Hydrologic Analyses of Existing and Proposed Conditions:

This section of the Drainage Report will give a brief description of the proposed-conditions drainage within and adjacent to the subject property, as follows. This section should include a detailed description of the method and input parameters used, and the reasonableness of the results.

a. Hydrologic Method

   i. Identify, and discuss in detail, procedures, methods, and assumptions used for hydrologic analysis. Specifically, refer to the use of the Oro Valley Category 1 Flood Hydrology Procedure (C1FHP) for smaller, uncontrolled watersheds; or, the use of the Category 2 Flood Hydrology Procedure (C2FHP) for larger, more-complex watersheds, as described in Chapter 3 of this DCM.

(Note: Hydrologic information contained within the *Oro Valley Town-Wide Drainage Study* [Kimley-Horn, 2007] may be used as a guide for comparison of watershed areas and input parameters. However, the Town-Wide Drainage Study ***shall not be used*** as the sole or unvalidated basis for floodplain mapping or design of public and private improvements within the Town.)

   ii. If, after written approval by the Town, another hydrologic method is used to calculate flood peaks, its results must be compared with those results obtained from using either C1FHP or C2FHP, as appropriate for the watershed.

   iii. When available computer software is used to help facilitate hydrologic analyses, its name and version number (e.g., Hydrologic Engineering Center Hydrologic Modeling System [HEC-HMS] version 4.4.1) shall be provided.

   iv. Identify predicted future-conditions land-use, and how such conditions are being represented in the hydrologic analyses for offsite areas. Hydrologic analysis shall consider future-condition flow rates that may result because of upstream development either within or outside of the Town. This analysis is imperative as future-condition flow rates either must be contained within the freeboard of any constructed channel; or, the natural floodplain must address future flood levels. Furthermore, minimum FFEs shall be set at least one foot higher than existing; or, if applicable, future flood levels.

b. Calculated Flood Peaks for Existing Conditions

   i. Provide a watershed map showing points of drainage concentration, watershed divides, soil types, and land uses.

   ii. Provide a summary table listing all of the input parameters used in the hydrologic analyses, including the drainage areas; land use categories; watershed basin factors; the 1-hour, 3-hour, or 24-hour rainfall amounts and frequencies; the Hydrologic Soil Groups (HSGs) and their area-weighted Curve Numbers; and the channel-routing geometry, as appropriate for the method used.

   iii. Specify where hydrologic calculations and their supporting data can be found in the report appendix.

iv. Provide one or more summary tables of the calculated peak discharges for the 50%, 20%, 10%, 4%, 2%, 1%, and 0.2% AEP floods. Include concentration point numbers and drainage areas.

v. If flood peaks are calculated by interpolation, or by some other hydrograph summation procedure, provide these calculations.

vi. Compare calculated flood peaks in the report with those found in previous reports for the same geographical area. Special emphasis should be given to comparing the current results with those given in FEMA Flood Insurance Studies, Basin-Management Studies, Drainage Reports for adjacent developments, and Design Concept Reports accompanying Drainage Plans or Roadway Improvement Plans.

c. Calculated Flood Peaks for Proposed Conditions

i. Provide a watershed map showing points of drainage concentration, watershed divides, soil types, and land uses.

ii. Provide a summary table listing all of the input parameters used in hydrologic analyses, including drainage areas; land use categories; watershed basin factors; the 1-hour, 3-hour, or 24-hour rainfall amounts and frequencies; the HSGs and their area-weighted Curve Numbers; and the channel-routing geometry, as appropriate for the method used.

iii. Specify where hydrologic calculations and their supporting data can be found in the report appendix.

iv. Provide one or more summary tables of calculated peak discharges for the 50%, 20%, 10%, 4%, 2%, 1%, and 0.2% AEP floods. Include concentration point numbers and drainage areas.

v. If flood peaks are calculated by interpolation, or some other hydrograph summation procedure, provide these calculations.

vi. Identify onsite concentration points that have calculated flood peaks modified by stormwater detention. Refer to summary tables for stormwater-detention-facility outflows.

vii. Compare calculated flood peaks in the report with those found in previous reports for the same geographical area. Special emphasis should be given to comparing the current results with those given in FEMA Flood Insurance Studies, Basin-Management Studies, Drainage Reports for adjacent developments, and Design Concept Reports accompanying Drainage Plans or Roadway Improvement Plans.

d. Comparison of Calculated Flood Peaks for Existing and Proposed Conditions

i. Provide one or more summary tables of numerical difference between existing-condition and proposed-condition peak discharges for the 50%, 20%, 10%, 4%, 2%,

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1%, and 0.2% AEP floods. Include the identification numbers of the points of drainage concentration, along with their corresponding drainage areas.

ii. Special emphasis should be given to comparing flood peaks at concentration points exiting the subject property.

iii. Identify those points of drainage concentration listed on the summary table where discharges are to be regulated by planned onsite stormwater detention.

iv. Identify those locations, if any, where proposed-conditions flood peaks are predicted to be less than those calculated for existing conditions.

v. Affirm that the minimum stormwater detention requirements for Critical Basins are met at the downstream terminus of the subject property.

6. Floodplain Mapping of Existing and Proposed Conditions:

This section of the report consists of a detailed description of the method and input parameters used to map floodplains, and the reasonableness of the results.

a. Hydraulic Methods and Procedures

i. Identify the computer software used to model and map the 1% AEP flood plain for both existing and proposed conditions. Give the software name and version number (e.g., Hydrologic Engineering Center River Analysis System [HEC-RAS] 2D version 5.0.7; FLO-2D Pro, build 18.12.20).

ii. Affirm that the floodplain models were developed in accordance with the policies and procedures contained in (1) the PCRFCD's *Standards for Floodplain Work Maps DS-305* (PCRFCD, 2014), and in (2) the ADWR's *State Standard for Floodplain Hydraulic Modeling SSA9-02* (ADWR, 2002).

iii. Affirm that floodplain maps were prepared in accordance with the policies and procedures contained in the PCRFCD's Standards for Floodplain Hydraulic Modeling Technical Policy, TECH-019 (PCRFCD, 2012).

iv. Identify the source of the topographic map, the year it was developed, and its basis of elevation (e.g., North American Vertical Datum of 1988 [NAVD-1988]). Also, provide its map projection, if available (e.g., North American Datum of 1983 [HARN], Arizona Central, International Feet).

v. Give the flood discharges used and their probability of occurrence (AEP), and identify whether the model was run steady state or as unsteady/variable flow. If discharges are changed along the river reach because of tributary inflows, identify the sequence of cross-sections where each of the varying discharges apply.

vi. Describe the physical characteristics and the Manning roughness coefficients (i.e., n-values) used to represent the sandbed channel, left and right overbank areas, and sheetflow areas. If applicable, describe how the sediment being transported influenced the selection of the Manning n-value. Include, in enough detail, a description of what the Manning n-value represents so that potential future changes, such as that caused by growth of in-channel vegetation, can be identified; and a decision can be made as to whether channel maintenance is needed in order to help restore the as-designed hydraulic capacity of the channel.

vii. Watercourses with 1% AEP flood peaks greater than 3,000 cfs, or also those that are maintained by the PCRFCD, will require additional coordination and acceptance by the PCRFCD, including possible revisions to the modeling and content of the Drainage Report. In these instances, provide a copy of the letter of acceptance for the Drainage Report from the PCRFCD.

b. Floodplain Mapping of Existing Conditions

i. For HEC-RAS 1D models, give the name of the model and plan geometry. Include one plan for existing conditions and a second plan for proposed conditions, all in the same model. Do not submit two separate models.

ii. For HEC-RAS 1D models, identify the reach boundary conditions and whether the flow regime of the model was set to subcritical, critical, or mixed. For floodplain mapping studies, subcritical analyses are preferred, unless the channel has ridged boundaries. Do not use supercritical analyses for channels with natural streambeds.

iii. For HEC-RAS 1D models that incorporate bridges or culverts, describe these structures and provide a copy of their plans/profiles. For culverts, describe the Manning n-value used to characterize the bottom of each; and indicate the area of the culvert that is blocked by sediment, if any.

iv. For HEC-RAS 1D models, give output summary tables, plotted cross-sections, and river/wash profiles for existing conditions.

v. For HEC-RAS 1D models, give a floodplain-boundary map, showing cross-sections and computed water-surface elevation(s) (WSEL).

vi. For HEC-RAS 1D models, affirm that HEC-RAS was used to verify the validity of input parameters, and that error and warning messages have been satisfactorily resolved. Provide a copy of the HEC-RAS report regarding possible problems with inputs and outputs.

vii. For HEC-RAS 2D models, describe the computational mesh, land classifications, break lines, boundary and initial conditions, and time-step used.

viii. For HEC-RAS 2D models, give a floodplain-boundary map showing limits of flooding and computed WSELs.

ix. For FLO-2D Pro models, identify the grid density, Manning n-values used and what they represent, and inflow/outflow nodes.

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x. For both HEC-RAS 2D and FLO-2D Pro models, give a floodplain-boundary map showing limits of flooding and computed WSELs.

xi. Provide a digital copy of the hydraulic model.

c. Floodplain Mapping of Proposed Conditions

i. Provide the same type of information requested for existing conditions, in Subsection b.i. through b.xi. above, for proposed conditions.

ii. Describe the changes in flow depths and velocities resulting from the planned modifications to the watercourse; and how this may impact bank erosion/lateral migration, local scour, and channel bottom aggradation/degradation.

iii. Quantify the changes in computed WSELs and velocities at the upstream and downstream boundaries of the subject property. Affirm that the planned encroachment complies with the floodway and floodway fringe area requirements found in Sections 17-5-7 and 17-5-8 of the Oro Valley Town Code, or as amended.

iv. Compare the calculated flood plains with those found in previous reports for the same geographical area. Special emphasis should be given to comparing the current results with those given in Flood Insurance Studies, Basin-Management Studies, Drainage Reports for adjacent developments, and Design Concept Reports accompanying Drainage Plans or Roadway Improvement Plans.

d. Floodplain Mapping for a Conditional Letter of Map Revision

i. Identify whether the project is in an effective FEMA Special Flood Hazard Area, and whether it will require a CLOMR or LOMR.

ii. Refer to another, separate Drainage Report prepared and formatted for submittal to FEMA for a CLOMR or LOMR.

7. Hydraulic Design of Drainage Improvements:

This section of the report shall consist of a detailed description of the methods and input parameters used to hydraulically design planned drainage improvements, and shall include justification of the reasonableness of the design.

a. Hydraulic Computational Methods and Procedures

i. Provide a list of hydraulic equations and procedures, primarily referring to the appropriate sections of this DCM, that were used to evaluate and design planned drainage improvements, including those for pavement drainage, at-grade crossings, curb cuts, storm-drain inlets, culverts, scuppers, and open channels.

ii. Include a description of the parameters, coefficients, and safety factors (e.g., clogging factors) used.

b. Proposed Drainage Improvements

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i. Give a detailed description of the planned drainage improvements, and where they are located within the watershed or sub-basin.

ii. Provide standard detail numbers, along with the name of the regulatory agency from which each standard detail originated.

iii. Identify where, on project improvement plans, proposed drainage improvements can be found (e.g., sheet number, detail number, roadway station).

iv. Provide hydraulic calculations for all proposed drainage improvements.

v. Provide a summary table of planned pavement drainage conditions, including location, street width, slope(s), design discharge(s), maximum flow depth(s), and a statement that the design meets all-weather access requirements.

vi. Provide a summary table of planned at-grade crossings of streets, including location, design discharge, and maximum flow depth, and a statement that the design meets all-weather requirements.

vii. Provide a summary table of planned curb openings, scuppers, and catch basins, including concentration points, structure types and dimensions, Pima Association of Governments (PAG) standard detail numbers, design discharges, interception and bypass flows, and maximum flow depths.

viii. Provide a summary table of planned culverts, including concentration points, types and dimensions, PAG standard detail numbers, design discharges, headwater depths, and outlet velocities.

ix. Provide summary table of planned storm drains, including concentration points, types and dimensions, design discharges, headwater depths, and outlet velocities.

x. Provide a summary table of planned open channels, including concentration points or cross-section numbers, design discharges, design velocities, freeboard depths, flow depths, design depths, bottom widths, transverse slopes, side-slopes, and Manning n-values.

8. Erosion Control:

The designer will need to provide the following information and analyses when a Regulatory Watercourse is modified in any way by the planned development, or as a consequence of long-term changes which may occur in the upstream watershed.

For scour within and along a Regulatory Watercourse, the following information is to be included in the Drainage Report, as development conditions warrant.

a. Provide a narrative discussion of visual trends and other factors that may be impacting the stability of the wash.

b. Provide an historic aerial photograph or topographic maps of the area that presents evidence, or the lack thereof, of channel stability over time.

c. Discuss the impact that proposed urbanization will have upon system sediment continuity and long-term (future) channel stability. Refer to regional planning documents for information regarding possible future development within the contributing watershed.

d. Evaluate the impacts of a proposed project with respect to either sediment deficit or sediment excess for any Regulatory Watercourses within the project area. A statement shall be given regarding any impacts or lack thereof. Generally, over the long term, the Design Engineer should expect that urbanization would lead to system sediment deficit.

e. Conduct sediment-transport modeling of large watercourses. For relatively large watercourses, such as the Cañada del Oro (CDO) Wash and major watercourses regulated by the PCRFCD, a sediment-transport model, such as HEC-6 or HEC-RAS, will be required unless it can be demonstrated to the satisfaction of the Town that such a model is not warranted. A sediment-transport analysis on these larger watercourses is required when:

    i. The constructed channel modifications will change the natural channel slope

    ii. Distributary flows will be combined into one or more channels, resulting in a larger unit discharge.

    iii. Construction and removal of features and/or vegetation will occur that otherwise control the natural channel slope.

    iv. Excavation of material is proposed within the channel bottom.

    v. Known areas of active degradation or aggradation are present.

    vi. A bridge or culvert structure is proposed.

f. Provide calculations and include a discussion of the maximum total scour to be used for long-term design purposes. Refer to the design procedures found in Chapter 5 of this DCM.

g. Provide calculations and a discussion of long-term aggradation and degradation, including equilibrium-slope calculations.

h. For bank-stabilization projects, provide a table of calculated toe-down depths along the channel banks, at grade-control structures, culverts, and other locations where the maximum total scour may vary. Identify where this design information can also be found on the improvement plans.

i. For projects with levees or levee-like structures, provide calculations for scour-protection toedowns, as well as top-of-bank elevations, including freeboard. Provide a maintenance plan, and identify who will be responsible for conducting regular maintenance.

j. For projects with planned sewerline crossings, provide calculations for Maximum Predicted Scour Depth using the procedures in Sec. 5.1.11(A) of the *Pima County Regional Wastewater Reclamation Department Engineering Design Standards* (Pima County Regional Wastewater Reclamation Department [PCRWRD], 2017), or as amended.

k. For any project with planned waterline crossings, provide calculations for Maximum Predicted Scour Depths using the design procedures in this DCM.

While assessment of erosion potential and required mitigation is specific to Regulatory Watercourses, it is recognized that Non-Regulatory Watercourses, given certain soils and/or topographic parameters, could be viewed as requiring erosion-mitigation elements as part of their improvement-plan development. In such cases, it is the responsibility of the designer to ascertain and, as warranted, address such conditions as part of any drainage submittal to the Town.

9. Stormwater Detention Facility Calculations:

This section of the report shall consist of a detailed description of the methods and input parameters used to hydraulically design stormwater detention facilities, and shall include validating the appropriateness of the design.

a. Identify the computer software used to perform the basin routing calculations. It is preferred that the most recent version of PC-ROUTE (PCRFCD, 2017) be used to route inflow hydrographs through stormwater detention facilities. Should other software be used, the software is to be identified and the method of analysis, and assumptions made, should be provided.

b. Identify the 50% AEP, the 10% AEP, and the 1% AEP flood inflow hydrographs, and their summarized location, in the Drainage Report.

c. Provide hydraulic calculations for the stormwater-detention-facility outlet structures, and summarize the results on a stage/storage/discharge table for each planned basin.

d. Provide routing calculations and a table summarizing results for each basin, including concentration point number, maximum inflow and outflow rates for the 50% AEP flood, the 10% AEP flood, and the 1% AEP flood, as well as maximum ponding depth and volume for the 1% AEP flood. Include graphic depictions of hydrographs, indicating the mitigation of each of these floods achieved through incorporation of stormwater detention facilities.

e. Describe how possible piggy-back storm events, and the potential for overflow conditions, will be addressed.

f. State that basins have at least one sediment-depth measurement device located where the majority of the sediment is likely to accumulate.

g. State that additional storage has been included to accommodate anticipated sediment delivery. Refer to the method of calculation found on Equation 3.8 of the *Pima County Stormwater Detention/Retention Manual* (PCRFCD, 1987).

h. Describe the basin safety and barricading structures/measures as well as maintenance access features.

i. Provide calculations and state that each basin will drain completely in 24 hours, with no water retained.

j. Request a stormwater detention waiver, if deemed appropriate, by showing the project is directly adjacent to a public wash and the outflow produced by the development can be demonstrated to not increase peak flow rates within the wash.

10. Storm Water Quality and Quantity:

a. Describe the first-flush devices to be installed, including the manufacturer's specifications and the supporting calculations.

b. Provide a summary table of planned first-flush devices, including the point of concentration, the manufacturer's make and model number, the design discharge, and the proposed maximum flow depth at the inlet.

c. Discuss water-harvesting measures that are utilized to help ensure compliance with the Oro Valley Town Code.

d. Provide a calculation of the total impervious area of the development, including conventionally surfaced streets, roofs, sidewalks, patios, driveways, and parking lots. This information serves as the basis for calculating the Stormwater Utility Fee for the project.

11. Long-Term Maintenance Plan:

Maintenance is required in order for drainage facilities to function as originally designed and constructed to ensure the service life of the facility is maximized. Common maintenance issues associated with drainage facilities include growth of undesirable vegetation, debris accumulation, sedimentation, erosion, scour, soil piping, soil settlement and structural damage. This section of the Drainage Report shall provide guidance to the designer regarding development of maintenance elements that will assure structural stability, operability, and aesthetics.

a. Provide written procedures for the inspection and maintenance of natural regulatory watercourses, including what, where, by who, and how often. This includes listing the BMPs needed as a means to maintain and/or restore the intended function of the infrastructure. Provide clear engineering matrices for when remediation becomes necessary.

b. Provide written procedures for the inspection and maintenance for all constructed drainage structures (e.g., channels, culverts, storm drains, catch basins, and stormwater

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detention facilities), including what, where, by who, and how often. This includes listing the BMPs needed as a means to maintain and/or restore the intended function of the infrastructure. Provide clear guidance regarding when remediation becomes necessary.

c. When one (1) foot of freeboard exists between the 1% AEP flood and the FFEs of adjoining structures, provide a clear metric to identify when vegetative growth and/or sediment accumulation must be cleared in order to reestablish the design hydraulic characteristics and maintain the minimum regulatory freeboard.

d. Clearly specify that drainage facilities owned and/or operated by private entities, including Homeowner's Associations, shall be properly maintained to promote performance of the drainage facilities consistent with the original design intent, including stormwater quality.

    i. Identify all drainage easements for these facilities, and identify whether they are public or private.

    ii. Provide a statement that all drainage channels, roadway culverts, and stormwater detention facilities have dedicated physical and legal access for maintenance.

    iii. Clearly indicate that the purpose of the drainage easement is for operation and maintenance of the stormwater facility. Clearly state whether the purpose of the drainage easement includes landscape maintenance and aesthetics; and, if so, to what degree.

(Note: Public drainage easements are generally discouraged, and must have prior written approval from the Town before being identified in the Drainage Report or shown on Improvement Plans or Final Plats. A copy of the written acceptance by the Town shall be included in the Drainage Report.)

e. Include the following two notes in the Drainage Report, Final Plat, Final Site Plan, and Covenants, Conditions, and Restrictions (CC&Rs).

    All drainage structures shall be inspected and a summary report prepared a minimum of once each year in accordance with the procedures in the approved Drainage Report. Copies of the annual inspection reports shall be made available to the Town upon request.

    All drainage structures shall be inspected and a summary report prepared by an Arizona Registered Professional Civil Engineer a minimum of once every five years in accordance with the procedures presented in the approved Drainage Report. Copies of the 5-year-interval inspection reports shall be made available to the Town upon request. The report shall identify the maintenance needs for the next 5-year period, including the anticipated annual cost of maintenance and repair.

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f. If specific maintenance is identified during the design of drainage infrastructure, for example that culverts may require occasional sediment removal, BMPs should be identified which will assure design operability.

12. Completeness and Certification:

The Engineer of Record for the project will provide the following:

a. Drainage Concept. A brief statement that the project substantially maintains the historical flow patterns, will ultimately discharge to its historic outfall, and that impacts to adjoining and downstream properties will be minor.

b. Compliance with Standards. Certification that the proposed drainage plan, once properly constructed, will adhere to applicable local, state, and federal floodplain regulations, including the DCM as well as the requirements in Chapter 17 of the Oro Valley Town Code (Floodplain and Erosion-Hazard Management).

c. Acceptance of Responsibility. Attest that all submittals to the Town have been thoroughly reviewed, in conformance with the firm's quality control plan, for possible errors, omissions, and/or discrepancies; as well as attest that all submittals achieve the "standard of care" defined by the Arizona State Board of Technical Registration.

13. References:

a. All references indicating sources of procedures used should be listed at the end of the Drainage Report or Drainage Statement.

The following is a list of appendices to append to the end of Drainage Reports, as appropriate for the subject development:

Appendix A—Report Checklist

Appendix B—Hydrologic Computations and Backup

Appendix C—Hydraulic Computations and Backup

Appendix D—Plans, Maps, and Figures

Appendix E—Computer Models

Appendix F—Drainage Report Addenda

2.7 Drainage Report Addenda Format

A Drainage Report Addenda provides supplemental information for, or revisions to, a previously accepted Drainage Report for the same project site. In order to facilitate review, the Drainage Addenda should be formatted to reflect sections of the Drainage Report, report figures, and supporting calculations to be revised. Furthermore, the Drainage Addenda shall include a brief cover letter describing why the report changes were needed. All addendums must be bound to

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the final Drainage Report prior to final acceptance of project construction by the Town and its release of assurances and/or bonds.

2.8 Quality of Submittals

The Arizona Registered Professional Civil Engineer who seals engineering reports and plans shall be held solely responsible for the correctness and adequacy of all data, drawings, calculations, and reports submitted to the Town for review and acceptance. In addition, the Engineer shall comply with all local, state, and federal floodplain regulations in the design of the development. Town staff will review the technical submittals for completeness and general compliance with all applicable floodplain regulations and drainage standards. Acceptance by the Town does not necessarily imply that the design is appropriate, nor that the development is in strict compliance with all applicable regulations and standards. Review and acceptance of drainage submittals shall not create liability on the part of the Town or its employees for any flood damages that may result from reliance upon any administrative decision made by the Town or its employees.

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References

Arizona Department of Water Resources (ADWR). (2002). *State Standard for Floodplain Hydraulic Modeling*. State Standard 9-02.

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

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Kimley-Horn (2007). *Oro Valley Town-Wide Drainage Study*.

Pima County Regional Flood Control District (PCRFCD). (1987). *Stormwater Detention/Retention Manual*.

Pima County Regional Flood Control District (PCRFCD). (2012). *Standards for Floodplain Hydraulic Modeling Technical Policy, TECH-019*.

Pima County Regional Flood Control District (PCRFCD). (2014). *Standards for Floodplain Work Maps DS-305*.

Pima County Regional Flood Control District (PCRFCD). (2017). *PC-ROUTE v. 5.0*. https://webcms.pima.gov/cms/one.aspx?portalId=169&pageId=60265

Pima County Regional Wastewater Reclamation Department (PCRWRD). (2017). *Engineering Design Standards*.

Simons, Li & Associates, Inc. (SLA). (1998). *Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona*. (Original work published 1989.)

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Chapter 3. Hydrology

3.1 Criteria

3.1.1 Overview

The determination of flood hydrology for designing stormwater facilities in the Town is to be performed according to the procedures set forth in this DCM. Deviations from the procedures outlined in this DCM require prior approval from the Town before proceeding with the determination of design hydrology. The primary source to be used for calibration of hydrologic calculations are the United States Geological Survey (USGS) Regional Regression Equations (RREs) that are applicable to the region encompassing the Town. That is, RREs are to be used as aids in the calibration of calculated design flood peaks.

The flood-hydrology procedure to be used for determining flood peaks in the Town consists of two separate categories, each dependent upon watershed size and hydrologic/hydraulic conditions. The two categories are described as follows:

> The Category 1 Flood Hydrology Procedure (C1FHP) shall be used based upon the size and the natural hydrologic/hydraulic characteristics of the contributing watershed. That is, the C1FHP shall be used for natural watersheds and for uncontrolled urban watersheds (e.g., those with no significant stormwater detention facilities) when contributing drainage areas do not exceed 1.0 square miles (640 acres) in size.
>
> The Category 2 Flood-Hydrology Procedure (C2FHP) can be used for any size watershed, but shall be used primarily for (1) natural watersheds larger than 1.0 square miles in size and (2) for controlled urban watersheds (e.g., those with significant stormwater detention facilities) of any size. A hydrologic model such as the U.S. Army Corps of Engineers (Corps) HEC-HMS is to be used in conjunction with the C2FHP.

Note that whether the C1FHP or the C2FHP is used, all calculated flood-peak results must be compared to the flood-peak results that are obtained using (1) USGS Rural RREs (Paretti et al., 2014) and (2) adapted Urban RREs from *USGS Open-File Report 82-365* (Stricker & Sauer, 1982) for the region encompassing the Town. This is to be done in order to verify the reasonableness of C1FHP or C2FHP results.

Table 3-1, on the following page, summarizes key elements of the two hydrology procedures that are to be used in the Town.

Table 3-1. Hydrology Procedures

Procedure	Parameters	Method	Comment
Category 1 (C1FHP) Streamlined Model	Duration and Intensity of Precipitation; Runoff Coefficient; Watershed Size and Geomorphology; and Time of Concentration	Streamlined Methodology, as Described in this Chapter; Used for Short-Duration (< 3-Hrs) Thunderstorms	Used for Watersheds Up to 1.0 Square Miles in Size that Consist of Natural Areas and/or Uncontrolled Urban Areas (e.g., No Significant Stormwater Detention or Channelization).
Category 2 (C2FHP) Precipitation-Runoff Model (e.g., HEC-1/HEC-HMS)	Precipitation	3-Hr Thunderstorm per National Oceanic and Atmospheric Administration (NOAA) Temporal Distribution (See Figure 3-3). 24-Hr Distribution per Natural Resources Conservation Service (NRCS) Type I 24-Hr Temporal Distribution (See Figure 3-4). **Areal Reduction Factors (ARFs) for Watersheds > 1.0 Square Miles in Size per Guidance in this DCM.**	A 3-Hr Thunderstorm Is to Be Used on Watersheds Less than 1.0 Square Miles in Size; and a 24-Hr General Summer Storm Is to Be Used on Watersheds Larger than 1.0 Square Miles in Size—Except that for Watersheds Larger than 10 Square Miles in Size the 3-Hr and the 24-Hr Results Shall Be Compared to USGS RRE Results, and the Most Accurate Results Shall Be Used.
	Loss Rate	Curve Number (CN) Method	Per Guidance Found in NRCS National Engineering Handbook (NEH), Chapter 10 (NEH, 2004).
	Time of Concentration	Velocity Method	Per Guidance Found in NRCS NEH, Chapter 15 (NEH, 2010).
	Direct Runoff	NRCS Unit Hydrograph	Per Guidance Found in Corps HEC-HMS User's Manual (USACE, 2018) and HEC-HMS Technical Reference Guide (USACE, 2000).
	Channel Routing	Modified-Puls Kinematic Wave	Per Guidance Found in Corps HEC-HMS User's Manual (USACE, 2018) and HEC-HMS Technical Reference Guide (USACE, 2000). Modified-Puls to be used in Natural Channels with Slopes Less than 1.5%. Kinematic Wave to be Used for Constructed and Natural Channels with Slopes Greater than 1%.
	Storage Routing	Per HEC-HMS Hydrologic Model	Per Guidance as Found in the Corps HEC-HMS Users' Manual (USACE, 2018) and HEC-HMS Technical Reference Guide (USACE, 2000).
	Transmission Losses	Not Applicable	Will only be considered for large watercourses (e.g., the CDO and Lower Big Wash), with sufficient data to substantiate the transmission loss (e.g., streamgage data or scientific instrumentation).

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The point-precipitation depths that are to be used with the Town hydrology procedure are those contained in the current online version of the National Oceanic and Atmospheric Administration (NOAA) Atlas 14. In all cases, *the upper 90% confidence-interval point values shall be used*. As of the date of publication of this DCM (2020), NOAA Atlas 14 point-precipitation depths can be found online at:

https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=az

3.1.2 Areal Reduction of Point Precipitation

For contributing watershed areas from 1 to 10 square miles in size, and when storms have durations of less than 3 hrs (i.e., thunderstorms), the following equation from the COTDSM (SLA, 1989/1998) shall be used to calculate the Areal Reduction Factor (ARF; unitless):

$$ARF = A^{-(0.027+0.07LogA)} \qquad \textit{(Equation 3.1)}$$

(Note: Apply the calculated ARF to all durations less than 3 hrs).

For contributing watershed areas from 10 to 500 square miles in size, and when storms have durations of less than 3 hrs (i.e., thunderstorms), the following equation adapted from NOAA Hydro 40 (NOAA, 1984) shall be used:

$$ARF = 1.0 - 0.2(LogA) \qquad \textit{(Equation 3.2)}$$

(Note: Apply the calculated ARF to all durations less than 3 hrs).

For contributing watershed areas greater than 10 square miles, and when storms have durations of 24 hrs, the following equation adapted from NOAA Hydro 40 (NOAA, 1984) shall be used:

$$ARF = 1.06 - 0.16(LogA) \qquad \textit{(Equation 3.3)}$$

Typically, thunderstorms will produce the larger flood peaks for contributing watershed areas up to 10 square miles in size. For contributing watershed areas greater than 20 square miles in size, though, it is likely that a 24-hr storm event would produce the larger flood peaks. For contributing watershed areas from 10 to 20 square miles in size, though, either a 3-hr thunderstorm or a 24-hr storm event *might* produce a larger flood peak. Thus, both a 3-hr and a 24-hr storm event shall be used to calculate flood peaks for contributing watershed areas from 10 to 20 square miles in size. The larger peak value shall be used for design.

Watersheds containing urban features (e.g., walled-in backyards) shall have their contributing areas reduced in size for purposes of peak-discharge calculations by applying an appropriate "Contributing Area Factor" (CAF) to the watershed area (per Chapter IV of COTDSM) (SLA, 1989/1998). The CAF to be applied shall be in accordance with the applicable factor-value listed in Table 3-2. In instances where a watershed is comprised of multiple land use types (i.e., mixed land uses), an area-weighted CAF value shall be calculated. Conversely, when sizing stormwater-detention facilities a CAF shall not be used when computing flood volumes. For mixed land uses, CAFs shall be weighted by their decimal percentages of watershed area.

Table 3-2. Contributing Area Factors*

Type of Land Use				
CAF (Natural/Rural)	**CAF (Suburban)**	**CAF (Moderately Urban)**	**CAF (Highly Urban)**	**CAF (Commercial/Industrial)**
1.00	0.90	0.70	0.80	0.90

*The CAF values in Table 3-2 assume maximum anticipated development will occur for applicable land uses located within upstream contributing watershed areas. Should this not be the anticipated case, then CAF values should be weighted by percent coverage of each applicable land use within upstream contributing watershed areas.

Note that CAFs should be applied independently of changes to flood peaks and flood volumes otherwise created by stormwater detention facilities placed within contributing watershed areas.

3.2 Storm Types

Two storm types are used in this DCM as precipitation input for the two categories of hydrology procedures described in this DCM. These two storm types include (1) a 24-hr NRCS Type I storm, and (2) a 3-hr thunderstorm. The 3-hr thunderstorm shall be utilized for all watersheds smaller than 1.0 square miles in size, and its temporal storm distribution shall be as described in this chapter of the DCM. Where watersheds range from 10 to 20 square miles in size, both the 3-hr thunderstorm and a 24-hr NRCS Type I storm shall be used to calculate flood peaks, and the larger peak value shall be adopted for design purposes. In evaluating larger watersheds (i.e., greater than 20 square miles in size), a 24-hr NRCS Type I storm distribution shall be used. Note that although a 24-hr NRCS Type II storm is recommended by the NRCS for most regions of the U.S., including the Town, it was determined after extensive analysis conducted as an element of the *Tucson Stormwater Management Study* (SLA, 1995) that the 24-hr NRCS Type I storm more closely replicates the temporal intensity of historical storms in the region encompassing the Town. Therefore, the 24-hr NRCS Type I storm shall be utilized for watersheds greater than 20 square miles in size.

3.3 Category 1 Flood Hydrology Procedure (C1FHP) – Streamlined Methodology

<u>Procedural Approach</u>

1. Using topographic maps of appropriate resolution, determine the contributing watershed area in square miles or in acres (usually in acres when the area is less than 1.0 square miles).
2. Using topographic maps and/or or aerial photography of appropriate resolution, determine the longest watercourse, L, in the watershed, in feet (ft), measured from its outlet to the most remote watershed divide.
3. Using topographic maps and/or or aerial photography of appropriate resolution, estimate the distance, in feet, of the watershed centroid, L_{ca}, from the outlet of the watershed.

4. Using topographic maps of appropriate resolution, determine the change in length, ΔL, in feet, versus the change in elevation, ΔH, in feet, of the streambed profile of the hydraulically longest watercourse in the watershed. The mean watercourse slope, S_c, in feet/foot (ft./ft.), shall then be calculated by dividing the total watercourse length into individual segments, usually four or more, with each segment containing more or less equal contour-interval spacings, and using the following relationship:

$$S_c = \left(\frac{L}{G}\right)^2 \qquad \textit{(Equation 3.4)}$$

Where,

$$G = \left(\frac{\Delta L_1^3}{\Delta H_1}\right)^{\frac{1}{2}} + \left(\frac{\Delta L_2^3}{\Delta H_2}\right)^{\frac{1}{2}} + \left(\frac{\Delta L_3^3}{\Delta H_3}\right)^{\frac{1}{2}} + \cdots\cdots + \qquad \textit{(Equation 3.5)}$$

And the symbols ΔL_1, ΔL_2, ΔL_3, ΔL_4, etc., and ΔH_1, ΔH_2, ΔH_3, ΔH_4, etc., represent the changes in the *lengths* and heights, respectively, of the individual segments along the hydraulically longest watercourse.

5. Determine the type of land use(s) and physical surface characteristics within the watershed under investigation. That is, designate whether the watershed consists primarily of *Natural or Rural* land uses; *Suburban* land uses; *Moderately Urban* land uses; *Highly Urban* land uses; or *Commercial/Industrial* land uses. This designation is necessary in order to assist in establishing the overall percentage of impervious surfaces either currently existing within the watershed or that are projected to exist in the future within the watershed, as well as to determine the CAF that shall be applied to the watershed area. The definitions provided in Table 3-3 shall be used to select the land use type(s).

6. From field investigations, aerial photography, and topographic maps, visually estimate the overall hydraulic roughness, or so-called "basin factor" (n_b), for principal watercourses within the watershed. Recommended values of n_b to use for various land uses and physical surface characteristics of watersheds located within the Town are provided in Table 3-3.

Table 3-3. Recommended Watershed Basin Factors

Typical Range of Watershed Basin Factor (n_b) Values for Use when Computing 1% AEP Floods for Various Land Uses and Channel Types Located within the Town												
Natural/Rural				**Suburban**				**Moderately Urban, Highly Urban, and Commercial/Industrial**				
D.F.	**N.C.**	**C.C.**	**I.C.**	**D.F.**	**N.C.**	**C.C.**	**I.C.**	**D.F.**	**N.C.**	**C.C.**	**I.C.**	**L.C.***
0.050 to 0.100	0.030 to 0.060	0.030 to 0.060	0.025 to 0.060	0.050 to 0.100	0.030 to 0.060	0.030 to 0.060	0.025 to 0.060	0.050 to 0.100	0.030 to 0.060	0.030 to 0.060	0.022 to 0.060	0.016 to 0.020

Where,

D.F. = Dispersed flow (e.g., sheetflow, street flow, etc.) predominates within the watershed;

N.C. = Natural channels (i.e., typically bankfull capacity [Q_{cap}] ≤ 1% AEP peak flow [$Q_{P1\%}$]) predominate within the watershed;

C.C. = Competent channels (i.e., $Q_{cap} \approx Q_{P1\%}$) predominate within the watershed;

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I.C. = Improved channels (i.e., concrete-lined banks and $Q_{cap} \approx Q_{P1\%}$) predominate within the watershed;

U.C. = Underfit channels ($Q_{cap} \leq 0.5Q_{P1\%}$) predominate within the watershed; and,

L.C. = Lined channels (concrete-lined bed and banks and $Q_{cap} \approx Q_{P1\%}$) predominate within the watershed. *(*The basin-factor values shown for lined channels are absent bedload sediments in the flow. If bedload is present, increase the basin-factor values by a minimum of 0.005.)*

Natural/Rural watersheds generally contain from no houses to less than 1 house per acre, and anticipated (future) stormwater improvements are negligible. Impervious surfaces generally cover less than 10% of the watershed area.

Suburban watersheds generally contain two houses, or less, per acre and typically have few stormwater improvements. Impervious surfaces typically cover 20% of the watershed.

Moderately Urban watersheds generally contain from three to five houses per acre, detached, with moderate to extensive stormwater improvements. Impervious surfaces typically cover 40% of the watershed.

Highly Urban watersheds typically contain six or more houses per acre, and include Commercial, Industrial, and Multiple-Dwelling uses, with extensive stormwater improvements present. Impervious surfaces within a highly urbanized watershed typically cover 70% for residential uses and 90% for commercial/industrial uses.

The use of n_b values other than those provided in Table 3-3 to compute the 1% AEP (regulatory) flood within the Town will only be permitted if technical evidence is provided to the Town, for separate review and written approval, justifying use of alternate n_b values for the specific hydrologic conditions existing within the watershed.

Note that a n_b value is *not equivalent* to the Manning n-value. A n_b value is a *hydrologic* parameter that represents the composite effects of flow retardance within a watershed; whereas, the Manning n-value is a *hydraulic* parameter that represents resistance to flow created by the wetted perimeter of a specific stormwater conveyance element over or through which the stormwater is flowing.

However, when the calculated mean watercourse slope exceeds 0.006 ft/ft (i.e., 0.6%), the *minimum* basin factor, $n_{b(min)}$, (which is typically due only to grain roughness and the presence of bed forms) shall not be less than the value computed using the following relationship:

$$n_{b(min)} = 0.2619(S_c)^{1/2} \qquad \textit{(Equation 3.6)}$$

7. A reach-weighted basin factor, n_{bw}, for the watercourse flow path can be calculated by using *Equation 3.7*. This calculation is accomplished by entering a basin factor for each segment of the watercourse flow path and entering the value in the appropriate cell on the calculation sheet.

$$n_{bw} = \sum_{i=1}^{N_r} \frac{(n_i L_i)\left(\frac{S_c}{S_i}\right)^{1/2}}{L} \qquad \textit{(Equation 3.7)}$$

Where,

n_{bw} = Reach-weighted basin factor for watercourse flow path, dimensionless.

n_i = Basin factor for each incremental reach segment, i, dimensionless.

i = Increment number of reach segment, dimensionless.

N_r = Total number of incremental reach segments, dimensionless.

L = Total length of channel, in feet.

L_i = Length of each incremental reach segment, in feet.

S_c = Mean slope of channel, in feet/foot, as calculated from Step 4.

S_i = Slope of each incremental reach segment, in feet/foot.

Note, however, that for alluvial channels with natural streambeds, the weighted basin factor for each incremental segment of the watercourse flow path should be equal to or greater than $0.2619\sqrt{S_i}$, as previously stated. If this is not the case, then the individual basin factor for each incremental segment of the watercourse flow path which is an alluvial channel with a natural streambed should be re-evaluated and adjusted accordingly until the basin factor for that incremental segment is equal to or greater than $0.2619\sqrt{S_i}$. The preceding criterion, though, is not applicable to any totally lined incremental segments of watercourse flow paths.

8. The 1-hr, 2-hr, and 3-hr 1% AEP upper 90% confidence-interval point-precipitation depths at the centroid of the watershed shall be determined using NOAA Atlas 14. If the watershed exceeds 1.0 square miles in size, then an ARF shall be applied using either *Equation 3.1* or *Equation 3.2*, as applicable.
9. The appropriate NRCS Hydrologic Soil Group (HSG), or groups, shall be determined from the NRCS Websoil Survey webpage. Figure 3-1 depicts various soil types within the Town limits obtained from the NRCS Websoil Survey as of the date of this publication. As of the date of publication of the current version of the DCM, the NRCS Web Soil Survey can be found at:

https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm

Note that whenever Type A soils are encountered, *Type B soils shall always be substituted for Type A soils*. The four HSGs are defined as follows:

HSG A:

These soils have low runoff potential and a high rate of water transmission and high infiltration rates, even when thoroughly wetted. The soils consist of deep, well to excessively well-draining sands or gravels, typically having less than 10 percent clay and more than 90 percent sand or gravel, and have gravel or sand textures.

HSG B:

These soils have moderately low runoff potential and a moderate rate of water transmission and moderate infiltration rates, even when thoroughly wetted. The soils consist chiefly of moderately deep to deep, moderately well to well-drained soils, with moderately fine to moderately coarse textures, and typically have between 10% and 20% clay, 50% to 90% sand, and have loamy sand or sandy loam textures.

HSG C:

These soils have moderately high runoff potential and a slow rate of water transmission and slow infiltration rates, even when thoroughly wetted. The soils consist chiefly of soils with a layer that impedes the downward movement of water, or soils with moderately fine to fine texture and a slow infiltration rate. Water transmission through this soil is somewhat restricted. These soils typically have between 20% and 40% clay, less than 50% sand, and have loam, silt loam, sandy clay loam, clay loam, and silty clay loam textures.

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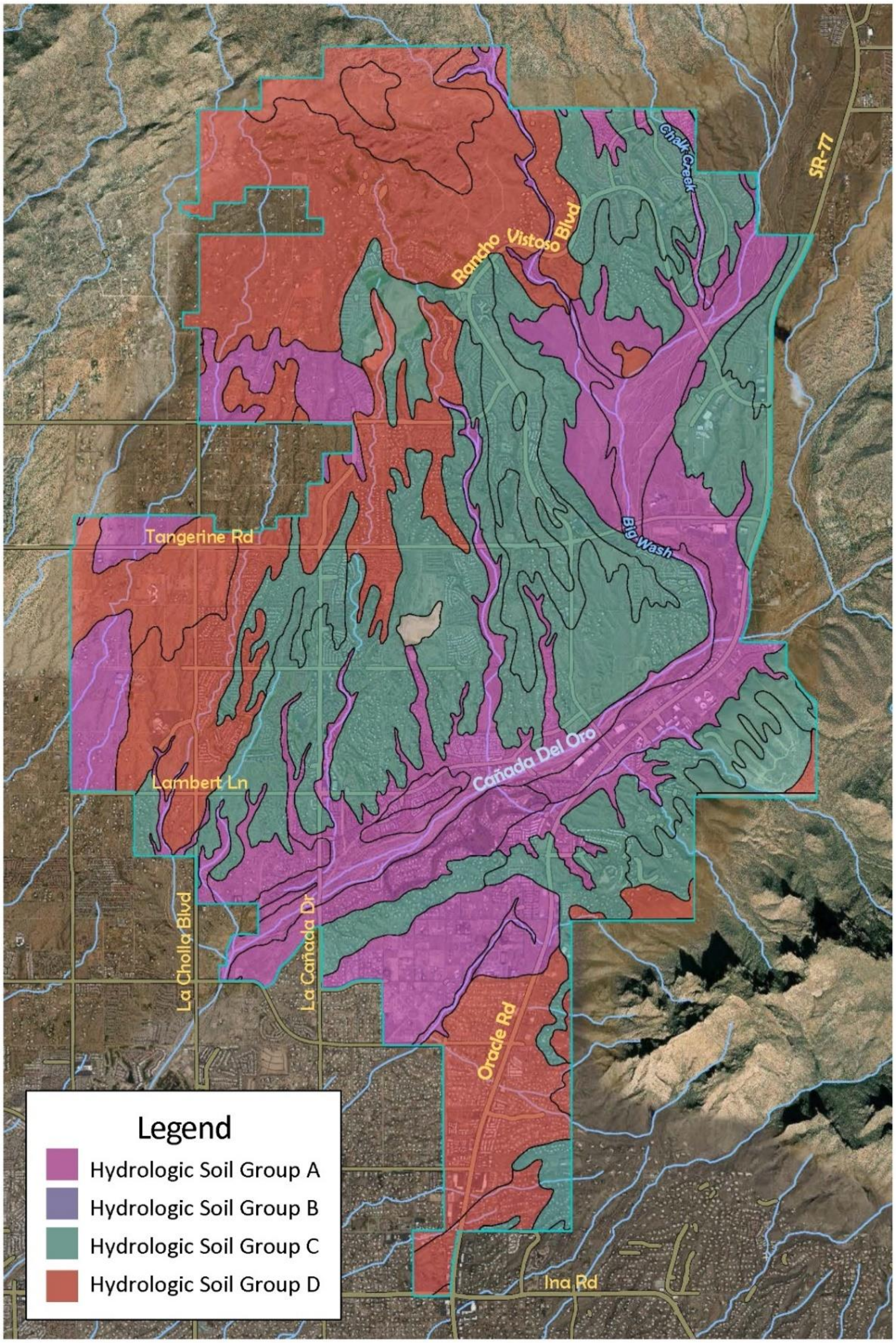

Figure 3-1. Overview of NRCS Soil Group Descriptions for the Town of Oro Valley

Note that the HSG delineations in Figure 3-1 indicate the “dominant condition.” Other HSGs may be present (i.e., mixed hydrologic soil groups). See Web Soil Survey for more details (NRCS, 2019). In areas within the Town that do not have an HSG, the nearest HSG should be used.

<u>HSG D</u>:

These soils have high runoff potential and a very slow rate of water transmission and very slow infiltration rates, even when thoroughly wetted. The soils consist chiefly of clay soils with a high swelling potential; soils with claypan or clay layer at or near the surface; and shallow soils over nearly impervious materials. Water movement through the soil is restricted or very restricted. These soils typically have greater than 40% clay, less than 50% sand, and have clayey textures.

10. From field investigation and/or aerial photography, zoning maps, area plans, neighborhood plans, etc., the existing and future imperviousness (in percent) shall be determined for maximum anticipated urbanization in the watershed. The percentage of impervious surfaces shall generally correspond to the values contained in Table 3-4. When calculating design peak discharges, the maximum anticipated urbanization on the contributing watershed area shall be assumed.

11. Based upon the HSG(s); the land-use type(s) within the watershed; and the 1% AEP 1-hr precipitation depth (using either the point value or areally-reduced value, as applicable), select a runoff coefficient, C, from Table 3-4. If necessary, a runoff coefficient shall be calculated for both the pervious and impervious surfaces, and a weighted runoff coefficient, C_W, shall then be determined using the weighting procedure described in the example following Table 3-4.

Table 3-4. Runoff Coefficients for Use with the C1FHP* (Based Upon 1-Hr Precipitation†)

1-Hr Precip (in)	(1) Applicable Hydrologic Soil Group (HSG)				(2) Natural/Rural (≤10%)			(3) Suburban (20%)			(4) Moderately Urban (40%)			(5) Highly Urban (70%)			(6) Comm/Industrial (90%)		
	B	C	D	I	B	C	D	B	C	D	B	C	D	B	C	D	B	C	D
0.80	0.08	0.18	0.28	0.86	0.16	0.25	0.34	0.24	0.32	0.40	0.39	0.46	0.52	0.63	0.66	0.69	0.78	0.80	0.81
0.90	0.11	0.22	0.32	0.88	0.18	0.29	0.38	0.26	0.35	0.43	0.41	0.48	0.55	0.65	0.68	0.71	0.80	0.81	0.82
1.00	0.13	0.25	0.36	0.89	0.21	0.32	0.41	0.28	0.38	0.47	0.43	0.51	0.57	0.66	0.70	0.73	0.81	0.82	0.84
1.10	0.16	0.28	0.39	0.90	0.23	0.35	0.44	0.31	0.41	0.49	0.45	0.53	0.59	0.68	0.71	0.75	0.82	0.84	0.85
1.20	0.18	0.31	0.42	0.91	0.26	0.37	0.47	0.33	0.43	0.52	0.47	0.55	0.61	0.69	0.73	0.76	0.83	0.85	0.86
1.30	0.21	0.34	0.45	0.91	0.28	0.40	0.49	0.35	0.45	0.54	0.49	0.57	0.63	0.70	0.74	0.77	0.84	0.86	0.87
1.40	0.23	0.36	0.47	0.92	0.30	0.42	0.52	0.37	0.48	0.56	0.51	0.59	0.65	0.71	0.75	0.78	0.85	0.86	0.87
1.50	0.25	0.39	0.49	0.92	0.32	0.44	0.54	0.39	0.49	0.58	0.52	0.60	0.67	0.72	0.76	0.79	0.86	0.87	0.88
1.60	0.27	0.41	0.51	0.93	0.34	0.46	0.56	0.40	0.51	0.60	0.54	0.62	0.68	0.73	0.77	0.80	0.86	0.88	0.89
1.70	0.29	0.43	0.53	0.93	0.36	0.48	0.57	0.42	0.53	0.61	0.55	0.63	0.69	0.74	0.78	0.81	0.87	0.88	0.89
1.80	0.31	0.45	0.55	0.94	0.37	0.50	0.59	0.44	0.55	0.63	0.56	0.64	0.70	0.75	0.79	0.82	0.87	0.89	0.90
1.90	0.33	0.47	0.57	0.94	0.39	0.51	0.60	0.45	0.56	0.64	0.57	0.66	0.72	0.76	0.80	0.83	0.88	0.89	0.90
2.00	0.35	0.48	0.58	0.94	0.41	0.53	0.62	0.47	0.57	0.65	0.59	0.67	0.73	0.76	0.80	0.83	0.88	0.90	0.91
2.10	0.36	0.50	0.60	0.94	0.42	0.54	0.63	0.48	0.59	0.67	0.60	0.68	0.74	0.77	0.81	0.84	0.89	0.90	0.91
2.20	0.38	0.51	0.61	0.95	0.44	0.56	0.64	0.49	0.60	0.68	0.61	0.69	0.74	0.78	0.82	0.85	0.89	0.90	0.91
2.30	0.39	0.53	0.62	0.95	0.45	0.57	0.65	0.51	0.61	0.69	0.62	0.70	0.75	0.78	0.82	0.85	0.89	0.91	0.92
2.40	0.41	0.54	0.63	0.95	0.46	0.58	0.67	0.52	0.62	0.70	0.63	0.70	0.76	0.79	0.83	0.86	0.90	0.91	0.92
2.50	0.42	0.55	0.64	0.95	0.48	0.59	0.68	0.53	0.63	0.71	0.63	0.71	0.77	0.79	0.83	0.86	0.90	0.91	0.92
2.60	0.44	0.56	0.65	0.95	0.49	0.60	0.68	0.54	0.64	0.71	0.64	0.72	0.77	0.80	0.84	0.86	0.90	0.92	0.92
2.70	0.45	0.58	0.66	0.96	0.50	0.61	0.69	0.55	0.65	0.72	0.65	0.73	0.78	0.80	0.84	0.87	0.91	0.92	0.93
2.80	0.46	0.59	0.67	0.96	0.51	0.62	0.70	0.56	0.66	0.73	0.66	0.73	0.79	0.81	0.85	0.87	0.91	0.92	0.93
2.90	0.47	0.60	0.68	0.96	0.52	0.63	0.71	0.57	0.67	0.74	0.67	0.74	0.79	0.81	0.85	0.88	0.91	0.92	0.93
3.00	0.48	0.61	0.69	0.96	0.53	0.64	0.72	0.58	0.68	0.74	0.67	0.75	0.80	0.82	0.85	0.88	0.91	0.93	0.93
3.10	0.49	0.62	0.70	0.96	0.54	0.65	0.72	0.59	0.68	0.75	0.68	0.75	0.80	0.82	0.86	0.88	0.92	0.93	0.94
3.20	0.50	0.62	0.71	0.96	0.55	0.66	0.73	0.59	0.69	0.76	0.69	0.76	0.81	0.83	0.86	0.89	0.92	0.93	0.94
3.30	0.51	0.63	0.71	0.96	0.56	0.67	0.74	0.60	0.70	0.76	0.69	0.77	0.81	0.83	0.86	0.89	0.92	0.93	0.94

*Percentages in parentheses represent impervious surfaces (I) for each type of land use within a watershed. Values in columns (2) through (6) are calculated from column (1) based on weighted percentages of impervious surfaces (I).
†Upper 90% confidence interval 1-hr point-precipitation value.

It is noted that while the placement of stormwater detention facilities within contributing watershed areas will change downstream flood peaks and flood volumes, such facilities will not change the percentage of imperviousness with such areas.

<u>Example 3.1</u>

Given:

A Moderately Urban Watershed with a 1% AEP upper 90% confidence-interval 1-hr point-precipitation value = 2.73 inches (in); HSG = 65% D and 35% B. The Pervious cover = 60% and the Impervious cover = 40%.

Calculate:

The weighted runoff coefficient, $C_{w1\%}$, for the 1% AEP storm, considering the HSG types present.

Solution:

From Table 3-4, for HSG D, C = 0.783 (interpolated), for HSG B, C = 0.653 (interpolated); and,

$C_{w1\%}$ = (0.65)(0.783) + (0.35)(0.653) = 0.738.

12. Calculate the Time of Concentration for the 1% AEP storm, $T_{c1\%}$, in minutes (min), of the watershed. The relationship to use for calculating the $T_{c1\%}$ is a regression on an equation presented on Page 5 of the original version of the *Pima County Hydrology Manual for Engineering Design and Floodplain Management* (PCDOT & PCRFCD, 1979). The regression equation is:

$$T_{c1\%} = 0.87(K_{1\%})^{[0.641+0.221(LogK_{1\%})]} \qquad \textit{(Equation 3.8)}$$

Where,

$$K_{1\%} = \frac{1.2n_{b1\%}(LL_{ca})^{0.3}}{(S_c P_{1\%}^{1-hr} C_{w1\%})^{0.4}} \qquad \textit{(Equation 3.9)}$$

and $n_{b1\%}$ is the reach weighted basin factor for the 1% AEP flood. Note that when using *Equation 3.8* and *Equation 3.9*, the minimum $T_{c1\%}$ shall not be less than 5 minutes, and the *maximum* $T_{c1\%}$ shall not exceed 180 minutes (3 hrs).

13. The peak runoff-supply rate of the watershed for the 1% AEP flood is represented by the variable $q_{1\%}$, where $q_{1\%}$ is equal to the product of $C_{w1\%}$ and $i_{1\%}$ [i.e., ($C_{w1\%}$)($i_{1\%}$)]. The variable $i_{1\%}$, in inches/hour (in/hr), represents the storm intensity at $T_{c1\%}$ during the occurrence of a 1% AEP flood over the watershed. The variable $i_{1\%}$ can be calculated directly from a regression derived from NOAA Atlas 14 precipitation values for storm durations of 5 minutes to 180 minutes (3-hrs). The regression equation for $i_{1\%}$ is:

$$i_{1\%} = 5.1(T_{c1\%})^{-[0.0369+0.2030(LogT_{c1\%})]}(P_{1\%}^{1-hr}) \qquad \textit{(Equation 3.10)}$$

Where $P_{1\%}^{1-hr}$ is the precipitation (either the point or areally-reduced value), in inches, of a 1-hr duration storm event for a 1% AEP flood.

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14. Once $i_{1\%}$ has been determined, the 1% AEP peak discharge, $Q_{P1\%}$, in cubic feet per second (cfs), is computed using either one of the following relationships:

When A is in acres, $$Q_{P1\%} = 1.008(q_{1\%})(A)(CAF)$$ *(Equation 3.11)*

When A is in square miles, $$Q_{P1\%} = 645.33(q_{1\%})(A)(CAF)$$ *(Equation 3.12)*

(Note that CAF is the "Contributing Area Factor," a factor accounting for the walled-in areas [i.e., backyard effects] that exist to varying degrees within urban watersheds [see Table 3-2].)

Example 3.2

Given:

A 320-acre Moderately Urban watershed with Competent channels has a watercourse that is 9,000 feet in length, measured from its most remote divide to its outlet. Along this watercourse, the length from the centroid of the watershed to the outlet is 4,500 feet. The watercourse streambed profile is divided into three segments, as follows: ΔL_1 = 3,000 feet, ΔH_1 = 100 feet; ΔL_2 = 3,000 feet, ΔH_2 = 50 feet; and ΔL_3 = 3,000 feet, ΔH_3 = 25 feet.

From NOAA Atlas 14, the upper 90% confidence-interval 1-hr point-precipitation value for a 1% AEP storm is 2.73 inches. The soil type within the watershed is 100% HSG "B." For a Moderately Urban watershed, 40% is selected as the percent of impervious cover.

Calculate:

The 1% AEP peak flow.

Solution:

Applying *Equation 3.4* and *Equation 3.5*, the mean watercourse slope of the watershed is calculated to be 0.0154 ft/ft.

Table 3-3 recommends a minimum n_b = 0.030 be used for a Competent channel; but *Equation 3.6* indicates that $n_{b(min)}$ shall not be less than 0.0325. Accordingly, n_b = 0.033 is selected, since the channel bottom will remain in its natural state, and the watershed is Moderately Urban in nature.

Using Table 3-4, the weighted runoff coefficient for this moderately-urbanized watershed during a 1% AEP 1-hr storm event of 2.73 inches is interpolated to be C_w = 0.653.

Given the physical and hydrologic characteristics of the watershed, using *Equation 3.8* and *Equation 3.9* yields $K_{1\%}$ = 32.95 and $T_{c1\%}$ = 26 minutes (rounded to the nearest minute).

For a $T_{c1\%}$ of 26 minutes, *Equation 3.10* yields a precipitation intensity, $i_{1\%}$, of 4.84 in/hr.

Accordingly, the 1% AEP peak discharge for the watershed in this particular example is computed to be $Q_{P1\%}$ = 1.008(0.653)(4.84)(320)(0.7) = 714 cfs (rounded to the nearest cfs).

The peak discharge should then be checked/compared to the applicable USGS Urban RRE for Moderately Urban conditions. The USGS Moderately Urban RRE yields a mean peak discharge of

720 cfs for the 1% AEP flood (see Section 3.4), and that the ratio C1FHP/RRE = 0.9917 (i.e., for this example, the ratio is ≈ 1.0).

15. In order to determine peak discharges for other flood-recurrence intervals (e.g., 50% AEP, 20% AEP, 10% AEP, etc.), ratios (derived from Region 5 USGS Rural RREs) for the predominate watershed type listed in Table 3-5 shall be used.

Table 3-5. Ratios of Other Flood-Recurrence Intervals to the 1% AEP Flood*

Predominate Watershed Type	Flood-Recurrence-Interval Ratios (R)						
	50% AEP	**20% AEP**	**10% AEP**	**4% AEP**	**2% AEP**	**0.5% AEP**	**0.2% AEP**
(1) Natural/Rural	0.09	0.22	0.35	0.56	0.77	1.28	1.70
(2) Suburban	0.12	0.27	0.40	0.59	0.78	1.24	1.55
(3) Moderately Urban	0.13	0.28	0.40	0.60	0.79	1.25	1.56
(4) Highly Urban	0.14	0.30	0.42	0.61	0.79	1.26	1.57
(5) Commercial/Industrial	0.14	0.30	0.42	0.61	0.79	1.26	1.57

*Ratios are based upon USGS Region 5 RREs at 1.0 square miles, as presented in Paretti et al. (2014)

For example, using Table 3-5 and applying it to Example 3.2, the 10% AEP flood peak for Moderately Urban conditions is calculated to be $Q_{P10\%}$ = 0.4$Q_{P1\%}$ = 0.4(714) = 285.6 cfs.

It should be pointed out that if the full C1FHP is used to calculate flood-recurrence intervals other than the 1% AEP flood-recurrence interval (i.e., rather than using Table 3-5), then the n_b values listed in Table 3-3 may no longer be appropriate to use, since those n_b values are for 1% AEP floods and their use *may* not be appropriate when calculating peak discharges for lesser flood events.

In this regard, if time of concentration (i.e., $T_{cn\%}$) values for flood-recurrence-intervals other than the 1% AEP flood are required, a relationship that can be used to determine *approximate* $T_{cn\%}$ values for such events is:

$$T_{cn\%} = (T_{c1\%})(R)^{-0.4} \qquad \textit{(Equation 3.13)}$$

Where,

$T_{cn\%}$ = Time of concentration of the n% AEP flood, in minutes (rounded to nearest minute).

$T_{c1\%}$ = Time of concentration of the 1% AEP flood, in minutes (rounded to nearest minute).

R = Ratio of the n% AEP to the 1% AEP peak discharge (from Table 3-5).

<u>Example 3.3</u>

Given:

Using Example 3.2, a moderately urban watershed is comprised of 100% HSG B soils with a contributing drainage area of 320 acres and a CAF = 0.7 (from Table 3-2). The percent impervious cover is 40% (from Table 3-4).

Calculate:

The time of concentration for a 10% AEP flood peak.

Solution:

From Table 3-5, $Q_{P10\%}$ = 0.4$Q_{P1\%}$; thus $Q_{P10\%}$ = 285.6 cfs. Using *Equation 3.13* and $T_{c1\%}$ = 26 minutes yields $T_{c10\%}$ = 38 minutes (i.e., the time of concentration for a 10% AEP flood event, rounded to the nearest minute) as the approximate time of concentration for a 10% AEP flood.

3.4 USGS Regional Regression Equations (RREs)

Reasonableness of calculated peak discharges using the C1FHP is to be verified via comparison with the peak discharges obtained using USGS Rural RREs and Urban RREs applicable for use in the Town. Because upper-90% point-precipitation values are used in the C1FHP, for purposes of this DCM "reasonable" is defined as calculated RRE 1% AEP peak discharges that fall within the upper and lower bounds of 1.50 x (mean RRE) and 0.67 x (mean RRE), respectively.

As of the date of publication of this DCM, the following Rural RREs and Urban RREs presented in Table 3-6 are to be used. The Rural RREs, in cfs, presented in Table 3-6 are taken directly from the USGS-designated Region 5 RREs, as presented in Paretti et al. (2014). The various Urban RREs, also in cfs, are adapted from *USGS Open-File Report 82-365*, prepared by Stricker & Sauer (1982). Note that when using RREs presented in Table 3-6, area, A, is measured in square miles. Also, CAFs are not used when computing flood peaks from the applicable RREs.

Table 3-6. USGS Region 5 Rural and Urban RREs†

Flood Frequency (%AEP)	Rural RRE	Suburban RRE	Moderately Urban RRE	Highly Urban RRE*
50	$10^{(6.363-4.368A^{-0.060})}$	$4.904A^{0.21}(Rural\ RRE_{50})^{0.73}$	$5.717A^{0.21}(Rural\ RRE_{50})^{0.73}$	$8.230A^{0.21}(Rural\ RRE_{50})^{0.73}$
20	$10^{(5.868-3.506A^{-0.080})}$	$4.318A^{0.17}(Rural\ RRE_{20})^{0.78}$	$4.963A^{0.17}(Rural\ RRE_{20})^{0.78}$	$6.906A^{0.17}(Rural\ RRE_{20})^{0.78}$
10	$10^{(5.778-3.218A^{-0.090})}$	$4.151A^{0.16}(Rural\ RRE_{10})^{0.79}$	$4.720A^{0.16}(Rural\ RRE_{10})^{0.79}$	$6.404A^{0.16}(Rural\ RRE_{10})^{0.79}$
4	$10^{(5.757-2.988A^{-0.100})}$	$3.968A^{0.15}(Rural\ RRE_{4})^{0.80}$	$4.479A^{0.15}(Rural\ RRE_{4})^{0.80}$	$5.975A^{0.15}(Rural\ RRE_{4})^{0.80}$
2	$10^{(5.696-2.795A^{-0.110})}$	$3.848A^{0.15}(Rural\ RRE_{2})^{0.81}$	$4.313A^{0.15}(Rural\ RRE_{2})^{0.81}$	$5.657A^{0.15}(Rural\ RRE_{2})^{0.81}$
1	$10^{(5.651-2.634A^{-0.120})}$	$3.686A^{0.15}(Rural\ RRE_{1})^{0.82}$	$4.131A^{0.15}(Rural\ RRE_{1})^{0.82}$	$5.418A^{0.15}(Rural\ RRE_{1})^{0.82}$
0.5	$10^{(5.761-2.638A^{-0.120})}$	$3.727A^{0.16}(Rural\ RRE_{0.5})^{0.82}$	$4.163A^{0.16}(Rural\ RRE_{0.5})^{0.82}$	$5.414A^{0.16}(Rural\ RRE_{0.5})^{0.82}$
0.2	$10^{(5.750-2.502A^{-0.130})}$	$3.744A^{0.16}(Rural\ RRE_{0.2})^{0.82}$	$4.167A^{0.16}(Rural\ RRE_{0.2})^{0.82}$	$5.373A^{0.16}(Rural\ RRE_{0.2})^{0.82}$

† A is measured in square miles.

*Includes Commercial and Industrial land uses.

Note: In Paretti et al. (2014), the USGS-stated "Range of Application" of the RREs is 0.155 square miles ≤ A ≤ 2925 square miles. (Note: 0.155 square miles ≈ 100 acres.)

Example 3.4

Given:

The 320-acre (0.50 square miles) Moderately Urban watershed presented in Example 3.2.

Calculate:

The 1% AEP flood peak using the applicable USGS Rural and Urban RREs from Table 3-6.

Solution:

Since the watershed is Moderately Urban, compute the contributing watershed area using the 1% AEP Moderately Urban RRE. Thus, the contributing watershed area to use with the Moderately Urban RRE, in this case, shall be 0.50 square miles. Note that CAFs are not used when computing flood peaks from the applicable RREs. The 1% AEP Rural RRE yields a flood peak of 615 cfs for a natural/rural watershed 0.50 square miles in size. Inserting the computed rural flood peak of 615 cfs into the "Moderately Urban RRE" listed in Table 3-6 for a 0.50-square-mile watershed yields 720 cfs for a 1% AEP flood peak. The RRE value for the "reasonable upper bound" of the 1% AEP is 1,080 cfs, and the lower bound is 482 cfs. Note, though, that in this particular example the calculated C1FHP value of 714 cfs (as calculated in Example 3.2) lies only slightly below the mean RRE value of 720 cfs.

3.5 Synthetic Flood Hydrograph (SFH)

When using the C1FHP, the Synthetic Flood Hydrograph (SFH) procedure adapted from Chapter IV of the COTDSM shall be used to develop flood hydrographs for any return period for *natural and uncontrolled urban watersheds* within the Town (SLA, 1989/1998). In order to develop a SFH, the curvilinear, dimensionless hydrograph ratios shown in tabular form in Table 3-7 shall be used for development of design flood hydrographs for natural or uncontrolled flows within the Town.

Table 3-7. Curvilinear, Dimensionless Synthetic Flood Hydrograph Ratios

t/T_r (in minutes)	Q/Q_p (in cfs)	v/V (in acre-feet)	t/T_r (in minutes)	Q/Q_p (in cfs)	v/V (in acre-feet)
0.0	0.000	0.000	1.6	0.545	0.671
0.1	0.025	0.002	1.7	0.482	0.707
0.2	0.087	0.007	1.8	0.424	0.742
0.3	0.160	0.020	1.9	0.372	0.773
0.4	0.243	0.036	2.0	0.323	0.799
0.5	0.346	0.063	2.2	0.241	0.841
0.6	0.451	0.096	2.4	0.179	0.875
0.7	0.576	0.136	2.6	0.136	0.900
0.8	0.738	0.180	2.8	0.102	0.917
0.9	0.887	0.253	3.0	0.078	0.932
1.0	1.000	0.325	3.4	0.049	0.953
1.1	0.924	0.400	3.8	0.030	0.965
1.2	0.839	0.464	4.2	0.020	0.973
1.3	0.756	0.523	4.6	0.012	0.979
1.4	0.678	0.578	5.0	0.008	0.983
1.5	0.604	0.627	7.0	0.000	1.000

Where,

t/T_r	=	The cumulative time (t) from beginning of the stormwater runoff, in minutes, divided by the hydrograph rise time (T_r), in minutes.
Q/Q_p	=	The instantaneous discharge (Q) at time t/T_r, in cfs, divided by peak discharge (Q_p), in cfs.
v/V	=	The accumulated runoff volume under the flood hydrograph (v), in acre-feet, divided by the total runoff volume under the flood hydrograph (V), in acre-feet.

The rise time of the design flood hydrograph, T_r, in minutes, is computed from:

$$T_r = 47.667 \frac{C_W(P_{3-hr})A}{Q_p}$$ *(Equation 3.14)*

Where,

T_r	=	The rise-time of the flood hydrograph, in minutes.
C_W	=	The watershed-weighted runoff coefficient, dimensionless.
$P_{3\text{-}hr}$	=	The 3-hr rainfall depth, in inches (from NOAA Atlas 14).
A	=	The uncontrolled contributing watershed area, in acres. (A CAF is not used when computing T_r since for flood-storage purposes it is conservatively assumed that the total watershed area will contribute to the stormwater runoff volume.)
Q_p	=	The peak discharge of the design flood, in cfs.

The volume of the design flood hydrograph, V, in acre feet, is computed from:

$$V = 0.00175 Q_p T_r$$ *(Equation 3.15)*

Where all other variables are as defined under *Equation 3.14*.

<u>Example 3.5</u>

Given:

A 320-acre uncontrolled moderately-urbanized watershed comprised of 100% HSG B soils with 40% impervious cover has a 1% AEP peak discharge of 714 cfs. The 1% AEP 3-hr point-precipitation depth, $P_{3\text{-}hr}$, for the watershed is 3.19 inches, obtained from NOAA Atlas 14. From Table 3-4, the weighted runoff coefficient, $C_{W1\%}$, for the watershed is 0.653 (interpolated).

Calculate:

The 1% AEP Synthetic Flood Hydrograph.

Solution:

Using *Equation 3.14*, the 3-hr hydrograph rise time, T_r, is computed to be 45 minutes (rounded to the nearest minute). Using *Equation 3.15*, the volume, V, of the 1% AEP design flood hydrograph is computed to be 56.2 acre-feet. Accordingly, tabular and graphical design flood hydrographs are provided in Table 3-8 and Figure 3-2, respectively.

Table 3-8. Tabular 1% AEP Design Flood Hydrograph (Using Ratios from Table 3-7)

Time (T_r = 45 min)	Discharge (Q_p = 714 cfs)	Σ Volume (V = 56.2 ac-ft)	Time (T_r = 45 min)	Discharge (Q_p = 714 cfs)	Σ Volume (V = 56.2 ac-ft)
0.0	0	0.0	72.0	389	37.7
4.5	18	0.1	76.5	344	39.7
9.0	62	0.4	81.0	304	41.7
13.5	115	1.1	85.5	265	43.4
18.0	174	2.0	90.0	231	44.9
22.5	247	3.5	99.0	173	47.3
27.0	322	5.4	108.0	128	49.2
31.5	410	7.6	117.0	98	50.6
36.0	526	10.1	126.0	72	51.5
40.5	634	14.2	135.0	56	52.4
45.0	714	18.3	153.0	35	53.6
49.5	660	22.5	171.0	21	54.2
54.0	599	26.1	189.0	14	54.7
58.5	539	29.4	207.0	9	55.0
63.0	483	32.5	225.0	6	55.2
67.5	431	35.2	315.0	0	56.2

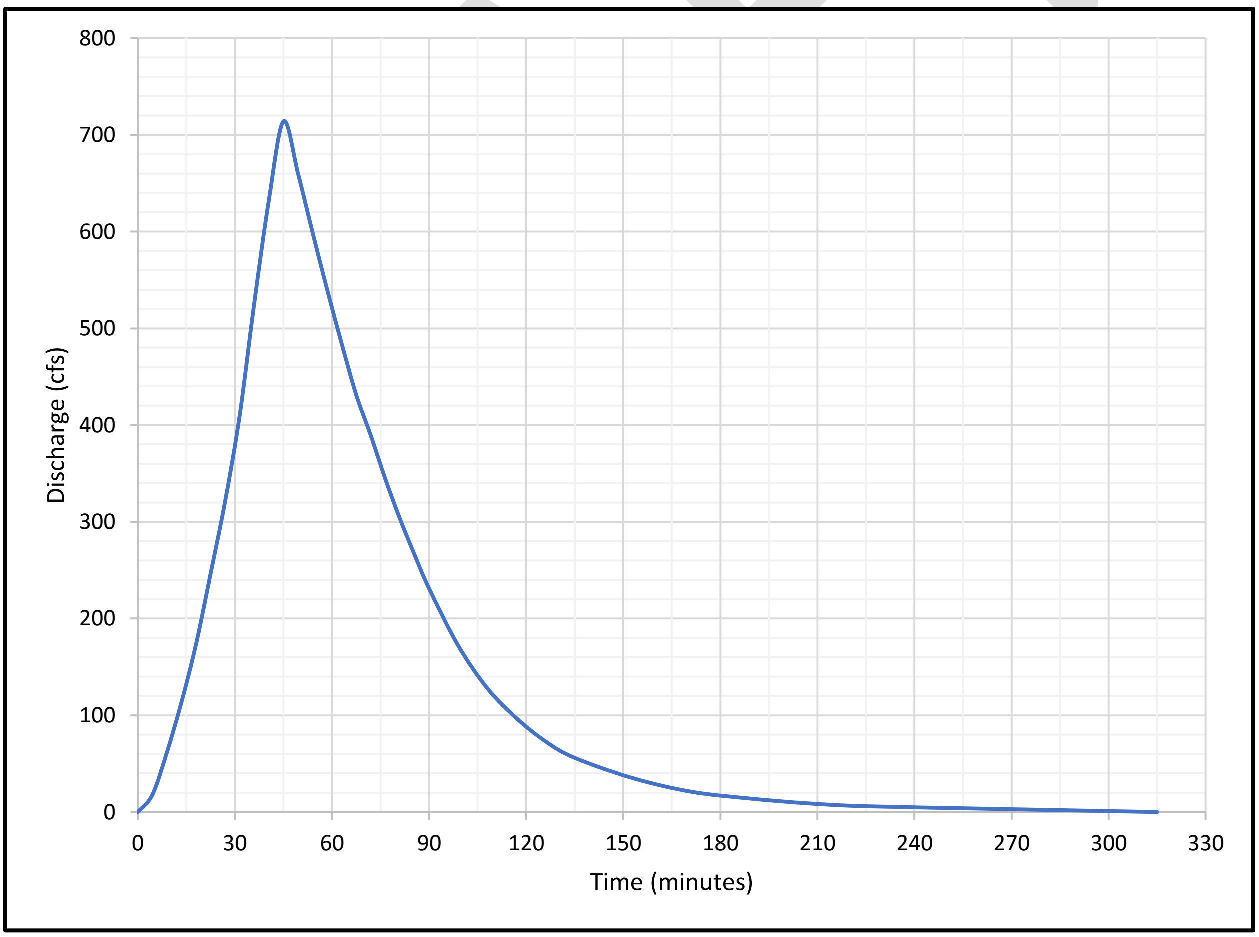

Figure 3-2. Graphical 1% AEP Design Flood Hydrograph (Using Ratios from Table 3-7)

3.6 Category 2 Flood Hydrology Procedure (C2FHP) – Detailed Hydrologic Modeling

For watersheds greater than 1.0 square miles in size, as well as all controlled watersheds (i.e., those watersheds containing significant stormwater detention), the use of detailed rainfall-runoff modeling shall be used to provide estimates of peak rates of flows, flood volumes, and runoff hydrographs. The preferred modeling tool is the Corps' HEC-HMS. HEC-1, which previously had been supported for many years by the Corps Hydrologic Engineering Center (HEC), is also acceptable; but it has been replaced by HEC-HMS, and is no longer used by the Corps. Copies of the latest HEC-HMS model and its supporting manuals can be downloaded from the HEC website at no cost. In addition, FLO-2D may be used to calculate flood peaks provided the model is formulated using the procedures outlined in PCRFCD's Technical Policy TECH-033, titled *Criteria for Two-Dimensional Modeling* (PCRFCD, 2013). The use of other detailed hydrologic models will be considered on a case-by-case basis by the Town. Designers are encouraged to seek approval of these alternative methods before initiating design work. Full support documentation will be required to be submitted to the Town prior to gaining the Town's approval when proposing the use of alternative modeling procedures. Input data and parameters, along with results, must be clearly and concisely annotated on printout sheets attached to the hydrologic/hydraulic report. Another important manual, that can be used as a technical reference for hydrologic modeling in desert-watersheds is the COTDSM (SLA, 1989/1998).

3.6.1 Precipitation

When conducting rainfall-runoff modeling, the designer shall use the 3-hr Thunderstorm when the total drainage area is less or equal to 10 square miles in size; and shall use the NRCS Type I 24-hr General Summer Storm when the total drainage area is more than 20 square miles. For drainage areas between 10 and 20 square miles, the designer shall evaluate both the 3-hr Thunderstorm and the 24-hr General Summer Storm results, and shall assure that the temporal distribution and duration of the precipitation selected is consistent with the results obtained using current USGS RREs (see Section 3.4 of this DCM).

The designer shall select the more conservative of the two results for application. In the event that the modeling result is unreasonably high or low, the designer may adjust the peak value. A determination of a "reasonable" peak value is based on the resulting peak lying within the upper and lower 90% confidence-interval bands associated with the applicable USGS 1% AEP RRE. As previously mentioned, these upper and lower bands are about 1.50 times the mean RRE value and 0.67 times the mean RRE value, respectively. However, if values are being lowered; it is the designer's responsibility to examine whether physical circumstances may be influencing the calculated higher result. Examples include watersheds that are steeper than typical; shaped such that the bulk of the drainage area is at the lower end of the watershed; contain significantly higher impervious factors related to soil, rock, and urbanization (e.g., walled-in areas); as well as other hydrologic factors. Such documentation shall be included in submittals (i.e. Drainage Reports or Drainage Statements) and provided to the Town for review and approval. Point-precipitation depths for local 3-hour Thunderstorms and 24-hour General Summer Storms are to be based upon upper 90% NOAA Atlas 14 point-precipitation values located at the centroid of the specified watershed. NOAA Atlas 14 point-precipitation values can be found online at:

https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=az

3.6.2 Temporal Storm Distributions

The temporal storm distributions to be used in the Town for a 3-hr Thunderstorm and a 24-hr General Summer Storm are provided in tabular format in Table 3-9 and in Table 3-10, respectively, and in graphical format in Figure 3-3 and in Figure 3-4, respectively. The factors in Table 3-9 were developed from a regression of current data from NOAA Atlas 14. The factors in Table 3-10 replicate the temporal distribution of the NRCS (formerly Soil Conservation Service [SCS]) Type I storm distribution. For details of the 3-hr and 24-hr distributions, the user is directed to NOAA Atlas 14 (2017) and NRCS (n.d.), respectively.

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Table 3-9. Tabular 3-Hr Thunderstorm Temporal Distribution

Time (min)	Σ % of 3-Hr Storm	Time (min)	Σ % of 3-Hr Storm	Time (min)	Σ % of 3-Hr Storm	Time (min)	Σ % of 3-Hr Storm
1	0.03	46	3.76	91	55.37	136	96.52
2	0.05	47	3.95	92	60.60	137	96.69
3	0.08	48	4.14	93	64.53	138	96.84
4	0.12	49	4.34	94	67.68	139	97.00
5	0.15	50	4.55	95	70.30	140	97.14
6	0.18	51	4.77	96	72.54	141	97.29
7	0.22	52	5.00	97	74.49	142	97.42
8	0.26	53	5.24	98	76.21	143	97.55
9	0.29	54	5.49	99	77.75	144	97.68
10	0.34	55	5.75	100	79.13	145	97.80
11	0.38	56	6.02	101	80.38	146	97.91
12	0.42	57	6.31	102	81.52	147	98.02
13	0.47	58	6.61	103	82.56	148	98.13
14	0.52	59	6.92	104	83.52	149	98.23
15	0.57	60	7.25	105	84.41	150	98.33
16	0.62	61	7.60	106	85.23	151	98.43
17	0.68	62	7.96	107	86.00	152	98.52
18	0.74	63	8.35	108	86.71	153	98.61
19	0.80	64	8.75	109	87.38	154	98.69
20	0.86	65	9.18	110	88.00	155	98.77
21	0.92	66	9.63	111	88.59	156	98.85
22	0.99	67	10.11	112	89.14	157	98.92
23	1.06	68	10.62	113	89.66	158	99.00
24	1.13	69	11.15	114	90.15	159	99.06
25	1.21	70	11.72	115	90.62	160	99.13
26	1.29	71	12.33	116	91.06	161	99.20
27	1.37	72	12.97	117	91.47	162	99.26
28	1.46	73	13.66	118	91.87	163	99.32
29	1.55	74	14.40	119	92.24	164	99.37
30	1.64	75	15.19	120	92.60	165	99.43
31	1.74	76	16.05	121	92.94	166	99.48
32	1.84	77	16.97	122	93.26	167	99.53
33	1.94	78	17.97	123	93.57	168	99.57
34	2.05	79	19.06	124	93.86	169	99.62
35	2.17	80	20.26	125	94.14	170	99.66
36	2.28	81	21.57	126	94.41	171	99.71
37	2.41	82	23.02	127	94.66	172	99.75
38	2.53	83	24.64	128	94.91	173	99.78
39	2.67	84	26.47	129	95.14	174	99.82
40	2.81	85	28.56	130	95.36	175	99.86
41	2.95	86	30.98	131	95.58	176	99.89
42	3.10	87	33.84	132	95.78	177	99.92
43	3.26	88	37.33	133	95.98	178	99.95
44	3.42	89	41.82	134	96.17	179	99.98
45	3.59	90	48.12	135	96.35	180	100.00

Source: Regression of Data Obtained from NOAA Atlas 14 (2017).

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Table 3-10. Tabular 24-Hr General Storm Temporal Distribution (NRCS Type 1 Storm)

Time (0.1 hrs)	Σ % of 24-Hr Storm	Time (0.1 hrs)	Σ % of 24-Hr Storm	Time (0.1 hrs)	Σ % of 24-Hr Storm	Time (0.1 hrs)	Σ % of 24-Hr Storm	Time (0.1 hrs)	Σ % of 24-Hr Storm
0.1	0.17	4.9	9.75	9.7	34.54	14.5	78.63	19.3	91.68
0.2	0.35	5.0	10.00	9.8	38.78	14.6	78.94	19.4	91.90
0.3	0.52	5.1	10.24	9.9	46.32	14.7	79.26	19.5	92.13
0.4	0.70	5.2	10.49	10.0	51.50	14.8	79.58	19.6	92.34
0.5	0.87	5.3	10.74	10.1	53.22	14.9	79.89	19.7	92.56
0.6	1.05	5.4	10.98	10.2	54.76	15.0	80.20	19.8	92.78
0.7	1.22	5.5	11.23	10.3	56.12	15.1	80.51	19.9	92.99
0.8	1.40	5.6	11.49	10.4	57.30	15.2	80.82	20.0	93.20
0.9	1.57	5.7	11.74	10.5	58.30	15.3	81.12	20.1	93.41
1.0	1.75	5.8	11.99	10.6	59.19	15.4	81.42	20.2	93.62
1.1	1.92	5.9	12.25	10.7	60.03	15.5	81.73	20.3	93.82
1.2	2.10	6.0	12.50	10.8	60.83	15.6	82.02	20.4	94.02
1.3	2.27	6.1	12.76	10.9	61.59	15.7	82.32	20.5	94.23
1.4	2.45	6.2	13.03	11.0	62.30	15.8	82.62	20.6	94.42
1.5	2.62	6.3	13.32	11.1	62.98	15.9	82.91	20.7	94.62
1.6	2.80	6.4	13.61	11.2	63.65	16.0	83.20	20.8	94.82
1.7	2.97	6.5	13.92	11.3	64.30	16.1	83.49	20.9	95.01
1.8	3.15	6.6	14.23	11.4	64.93	16.2	83.78	21.0	95.20
1.9	3.32	6.7	14.56	11.5	65.55	16.3	84.06	21.1	95.39
2.0	3.50	6.8	14.89	11.6	66.15	16.4	84.34	21.2	95.58
2.1	3.68	6.9	15.24	11.7	66.74	16.5	84.63	21.3	95.76
2.2	3.86	7.0	15.60	11.8	67.31	16.6	84.90	21.4	95.94
2.3	4.04	7.1	15.97	11.9	67.86	16.7	85.18	21.5	96.13
2.4	4.23	7.2	16.33	12.0	68.40	16.8	85.46	21.6	96.30
2.5	4.42	7.3	16.71	12.1	68.93	16.9	85.73	21.7	96.48
2.6	4.61	7.4	17.08	12.2	69.44	17.0	86.00	21.8	96.66
2.7	4.80	7.5	17.46	12.3	69.95	17.1	86.27	21.9	96.83
2.8	5.00	7.6	17.84	12.4	70.44	17.2	86.54	22.0	97.00
2.9	5.20	7.7	18.23	12.5	70.93	17.3	86.80	22.1	97.17
3.0	5.41	7.8	18.61	12.6	71.40	17.4	87.06	22.2	97.34
3.1	5.61	7.9	19.01	12.7	71.87	17.5	87.33	22.3	97.50
3.2	5.82	8.0	19.40	12.8	72.32	17.6	87.58	22.4	97.66
3.3	6.03	8.1	19.82	12.9	72.77	17.7	87.84	22.5	97.83
3.4	6.25	8.2	20.28	13.0	73.20	17.8	88.10	22.6	97.98
3.5	6.47	8.3	20.78	13.1	73.63	17.9	88.35	22.7	98.14
3.6	6.69	8.4	21.32	13.2	74.04	18.0	88.60	22.8	98.23
3.7	6.91	8.5	21.90	13.3	74.45	18.1	88.85	22.9	98.45
3.8	7.14	8.6	22.52	13.4	74.84	18.2	89.10	23.0	98.60
3.9	7.37	8.7	23.19	13.5	75.23	18.3	89.34	23.1	98.75
4.0	7.60	8.8	23.89	13.6	75.60	18.4	89.58	23.2	98.90
4.1	7.84	8.9	24.62	13.7	75.965	18.5	89.83	23.3	99.04
4.2	8.07	9.0	25.40	13.8	76.320	18.6	90.06	23.4	99.18
4.3	8.30	9.1	26.23	13.9	76.665	18.7	90.30	23.5	99.33
4.4	8.55	9.2	27.14	14.0	77.000	18.8	90.54	23.6	99.46
4.5	8.78	9.3	28.12	14.1	77.329	18.9	90.77	23.7	99.60
4.6	9.02	9.4	29.17	14.2	77.656	19.0	91.00	23.8	99.74
4.7	9.27	9.5	30.30	14.3	77.981	19.1	91.23	23.9	99.87
4.8	9.51	9.6	31.94	14.4	78.304	19.2	91.46	24.0	100.00

Source: NRCS Standard Rainfall Distributions (NRCS, n.d.).

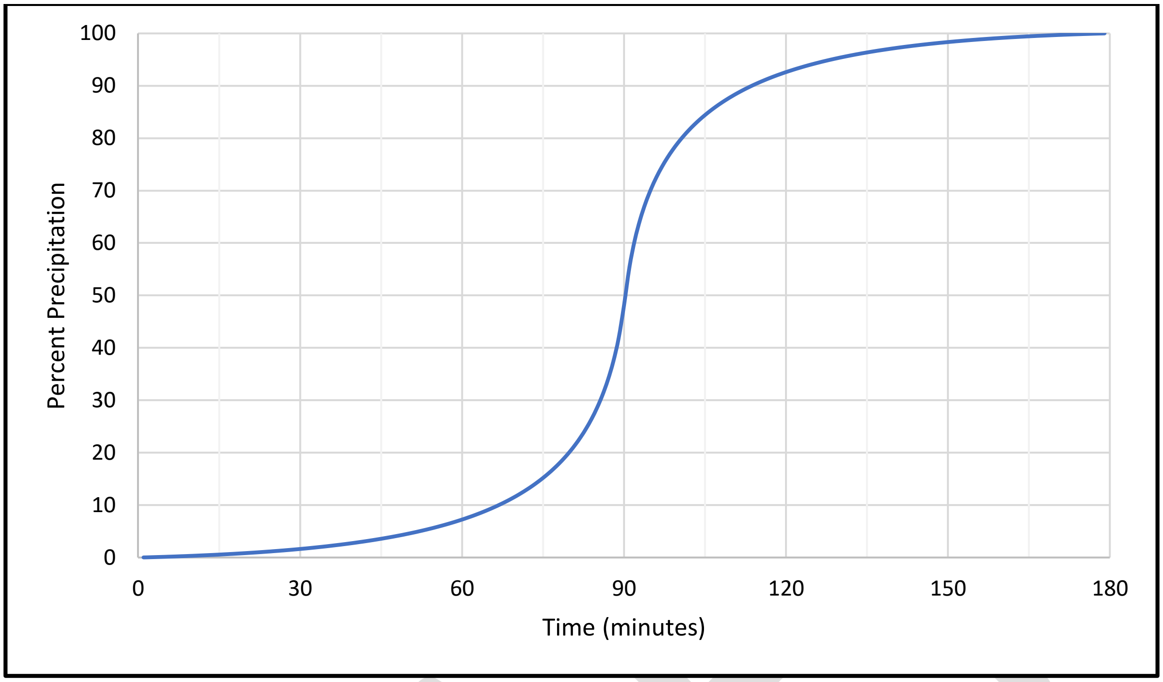

Figure 3-3. Graphical 3-Hr Thunderstorm Temporal Distribution.

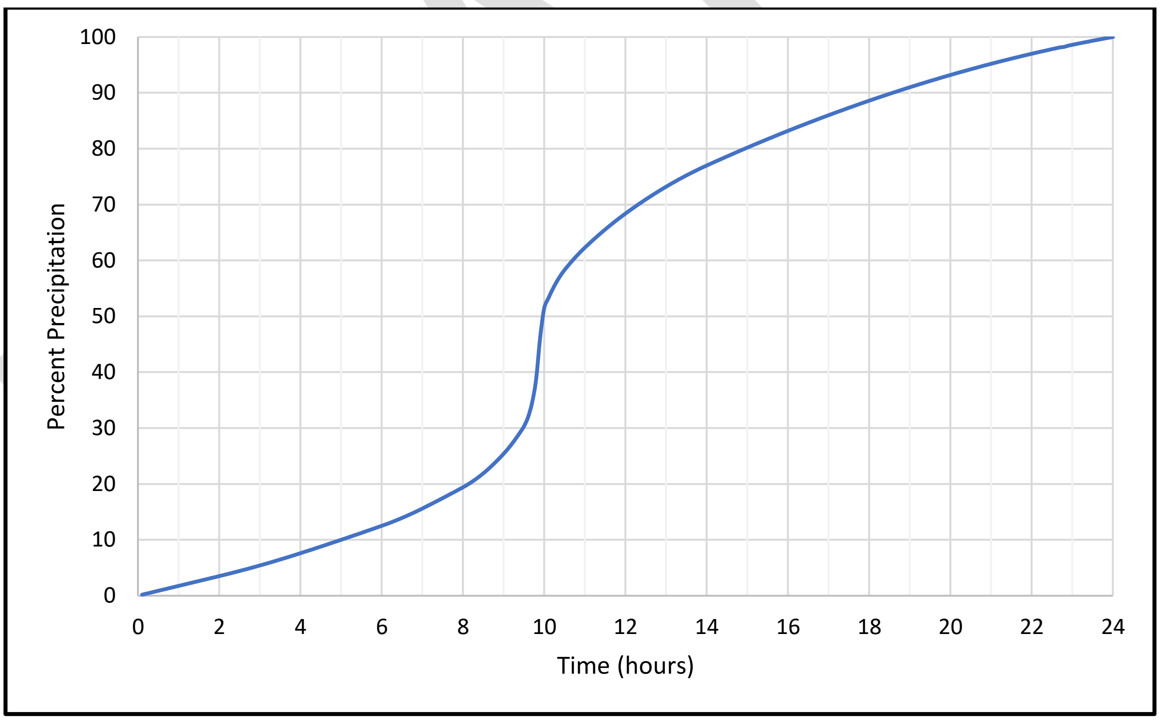

Figure 3-4. Graphical 24-Hr General Storm Temporal Distribution (NRCS Type 1 Storm)

3.6.3 Areal Reduction of Point Precipitation

When contributing watershed areas are between 1.0 square miles and 10 square miles in size, Thunderstorm ARFs shall be applied per *Equation 3.1*. However, when contributing watershed areas exceed 10 square miles in size, Thunderstorm ARFs shall be applied per *Equation 3.2* and

General Summer Storm ARFs shall be applied per *Equation 3.3* (note that General Summer Storms shall not be used when contributing watershed areas are less than 10 square miles in size). For this case, the use of the ARF that yields the largest peak flow should be used for design purposes. When contributing watershed areas exceed 20 square miles, *Equation 3.3* should be used to determine ARFs.

3.6.4 Loss Rate Methodology

When formulating a detailed hydrologic model, such as HEC-HMS, a loss-rate parameter is used to estimate the amount of precipitation that infiltrates (loss) and the amount of precipitation that runs off (excess). Various methods can be used to estimate the amount of loss; however, the Town has selected the NRCS Curve Number (CN) Method as the method to be used to determine precipitation loss rates. Figure 3-5 from the *PC-HYDRO User Manual* (PCRFCD, 2019) is to be used to select CN values for *3-hr Thunderstorms* which is based on HSG, cover type and percentage of vegetative cover. Note that although there are a various cover types depicted in Figure 3-5, the predominate cover type in the Town is Desert Brush. A CN = 99 is to be used for impervious surfaces (which is an invariant value).

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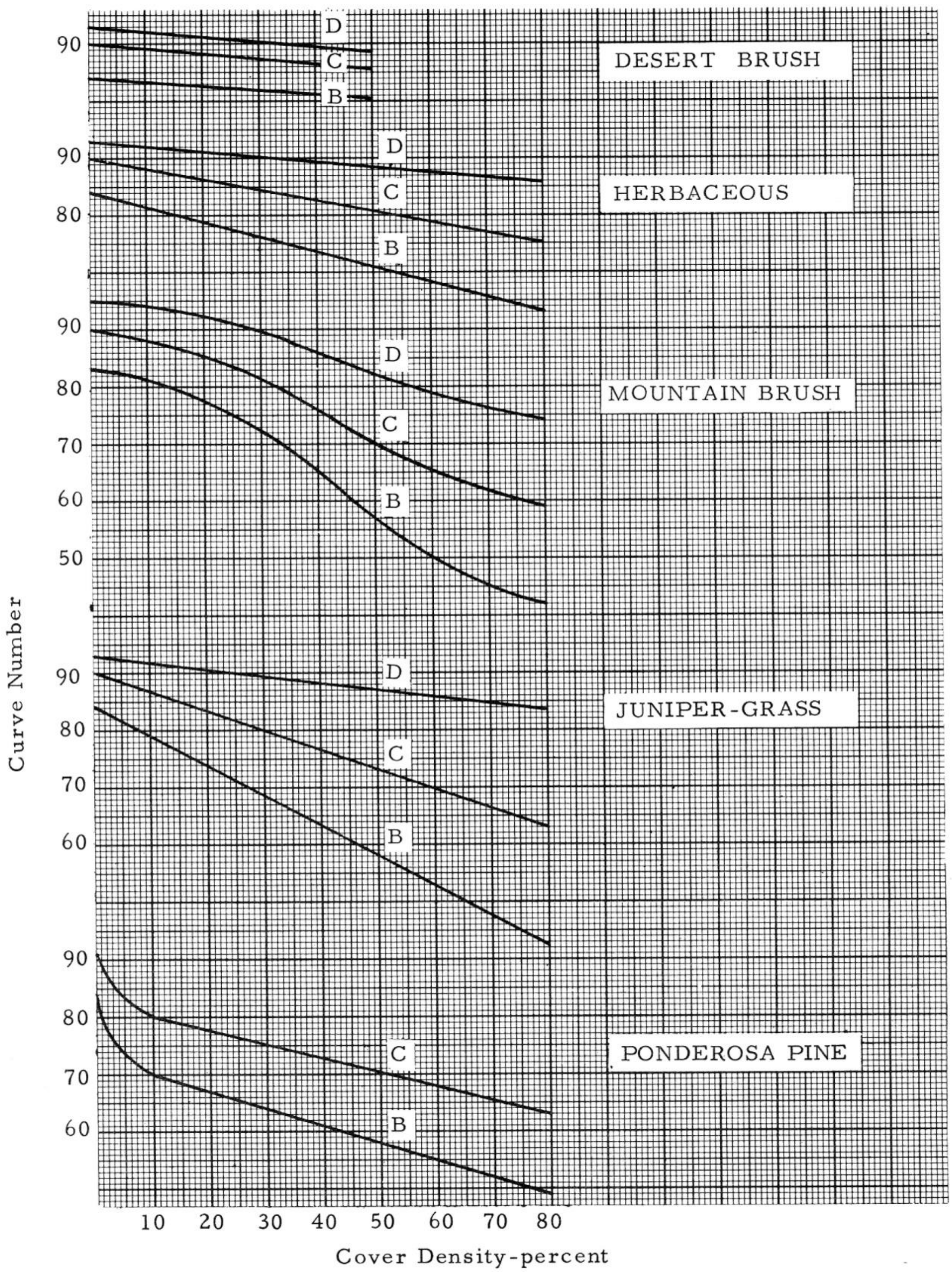

Figure 3-5. CN Chart

(Source: PCRFCD, 2019)

Because less-intense periods of rainfall occur within the temporal distribution of a 24-hr General Summer Storm, the CN values for a 3-hr storm (as derived from Figure 3-5) require an adjustment before they can be used for a 24-hr storm. A general relationship for the calculation of 24-hr CN values for HSG B, HSG C, and HSG D, is:

$$CN_{24} = 3.9884(CN_*) - 0.0151(CN_*)^2 - 152.52 \qquad \textit{(Equation 3.16)}$$

Where,

CN_* = The CN for a Thunderstorm less than 3 hrs in duration.

CN_{24} = The CN for a 24-hr General Summer Storm.

(Note: When using *Equation 3.16*, the value for CN_{24} shall be rounded to the nearest one-tenth.)

Equation 3.16 is a best-fit regression equation of CN_{24} versus CN_* data excerpted from *Runoff Curve Numbers for Semiarid Range and Forest Conditions* (Woodward, 1973).

Example 3.6

Given:

A watershed with a Desert Brush cover type (predominate cover type in the Town).

Calculate:

3-hr and 24-hr CN values for HSG B, HSG C, HSG D for 0%, 20%, 30%, 40% and 50% vegetative cover densities.

Solution:

Using Figure 3-5, 3-hr CN values are estimated from the chart for each HSG and vegetative cover type. *Equation 3-15* is then used to estimate the 24-hr CN values. Calculated answers are provided in Table 3-11.

Table 3-11. 3-Hr and 24-Hr CN Values for Varying Desert-Brush Cover Densities

Vegetation Cover Density	0%		10%		20%		30%		40%		50%	
CN for Storm Duration	< 3-hrs	24-hrs	< 3-hrs	24-hrs	< 3-hrs	24-hrs	< 3-hrs	24-hrs	< 3-hrs	24-hrs	< 3-hrs	24-hrs
HSG B	84.0	76.0	83.5	75.2	82.5	73.7	82	73.0	81.5	72.2	80.5	70.7
HSG C	90.0	84.1	89.0	82.8	88.5	82.2	87.5	80.9	86.5	79.5	85.5	78.1
HSG D	93.0	87.8	92.0	86.6	91.0	85.4	90.5	84.8	89.5	83.5	88.5	82.2

3.6.5 Transformation Methodology (Hydrograph Generation)

When conducting rainfall-runoff modeling using HEC-HMS, it is necessary to transform the runoff into a hydrograph. The modeled hydrologic process within a sub-basin (i.e., development of a hydrograph) shall be generated using the NRCS Unit Hydrograph model. For more detailed information regarding the NRCS Unit Hydrograph model refer to the *HEC-HMS User's Manual* (USACE, 2018) and the *HEC-HMS Technical Reference Manual* (USACE, 2000).

The modeled watershed shall be divided into a sufficient number of sub-basins to define its geometry in a manner that replicates the physical and hydrologic/hydraulic characteristics of the overall watershed area. At a minimum, the size of each sub-basin shall be equal to or less than 0.1 times the total watershed area, except that a sub-basin need not be less than 0.1 square miles (64 acres) in size for watersheds equal to or smaller than one square mile (640 acres) in size.

3.6.6 Channel Routing, Flood Storage, and Transmission Losses

Once a sub-basin hydrograph is developed, it is routed downstream so that it can be combined with other sub-basin hydrographs prior to the ultimate point of concentration. There are essentially two types of routing that occur: (1) channel routing, and (2) storage routing. Three natural phenomena are being mathematically simulated in routing; flood-wave translation (the movement of the flood wave in the system); attenuation (how the flood wave might be dampened due to in-channel or reservoir storage functions), and transmission losses (i.e., losses due to streambed infiltration). For studies involving large watershed sizes, transmission losses (i.e., streambed infiltration) can be significant. However, for most projects involving urban development, transmission losses are usually considered to be insignificant. Thus, transmission losses for projects in the Town will be considered on a case-by-case basis; and then only if the designer has supporting data to justify such losses. In addition, the method of calculating such losses must be approved in advance, and in writing, by the Town. Note that if transmission losses are assumed by the designer, the method must be used in both the existing-conditions and developed-conditions models—adjusted, as appropriate, to account for developed-conditions changes due to urban development (e.g., channel linings).

There are a number of routing methods embedded within HEC-HMS that are available for hydrologic modeling. The Town prefers that the designer use two methods: (1) the Modified-Puls Method (MPM) for natural channels; and (2) the Kinematic Wave Method (KWM) for constructed channels and natural channels.

The MPM should be used for natural channels where the slopes are generally less than 1.5%; but may also be used for steeper channels. The MPM uses conservation of mass and a relationship between storage and discharge to route flow through the stream reach. Attenuation is achieved though the storage and delayed release of water in the reach instead of through a rigorous conservation of momentum approach. It can be useful for representing backwater due to flow constrictions in a channel; that is, so long as the backwater affects are fully contained within the channel reach. A storage-discharge relationship is required for the MPM. Storage-discharge relationships can be developed using cross-sections and slopes derived from a Manning normal-depth analysis or from HEC-RAS. For more-detailed information on the MPM and development of storage-discharge relationships refer to the *HEC-HMS User's Manual* (USACE, 2018) and *HEC-HMS Technical Reference Manual* (USACE, 2000).

The KWM may be used for constructed channels and natural channels with slopes greater than 1%. The KWM for flow routing approximates the full unsteady flow equations by ignoring inertial and pressure forces. It also assumes that the energy slope is equal to the bed slope; thus, *this method is best suited for use on fairly-steep to steep streams* (i.e., typically those streams with slopes greater than 1.0%), which are typical within the Town. In addition, the KWM is well suited to urban areas where natural channels have been modified to have regular shapes and slopes (USACE, 2000, 2018). More detail regarding the application of the KWM for can be found in the *HEC-HMS User's Manual* (USACE, 2018) and *HEC-HMS Technical Reference Manual* (USACE, 2000).

3.6.7 Velocity Method Time of Concentration

Time of concentration (T_c) shall be calculated by summing the travel time for Sheetflow (T_{sf}), shallow concentrated flow (T_{scf}), and channel flow (T_{cf}) along the primary flow path, as follows:

$$T_c = T_{sf} + T_{scf} + T_{cf} \qquad \textit{(Equation 3.17)}$$

The following sections discuss the calculation methodology for each of the three (3) flow parameters contained in *Equation 3.17* for calculation of T_c.

3.6.7.1 Sheetflow

Manning roughness coefficients for sheetflow shall be obtained from Table 15-1 (see Chapter 15 of the NEH [NRCS, 2010]), reproduced in Table 3-12. Maximum slope length for sheetflow shall not exceed 100 feet, unless additional justification is provided. A modified version of the KWM equation, contained in Chapter 15 of the NEH, shall be used to estimate the travel time for sheetflow (also reproduced below).

A modified version of the Manning KWM equation may be used to compute travel time for sheetflow. The equation is modified from the NRCS NEH Chapter 15 equation, and is defined as:

$$T_{sf} = \frac{0.42\left(\left[n_{sf}\right]l\right)^{0.8}}{\left(P_{1\%}^{24-hr}\right)^{0.5} S_o^{0.4}} \qquad \textit{(Equation 3.18)}$$

Where,

T_{sf}	=	Sheetflow travel time, in minutes.
n_{sf}	=	Manning roughness coefficient for sheetflow (from Table 3-12).
l	=	Sheetflow length, in feet.
$P_{1\%}^{24-hr}$	=	The 1% AEP 24-hr rainfall, in inches.
S_o	=	Slope of the *overland surface*, in feet/foot.

(Note: T_{sf} has been modified to account for increased sheetflow flow depth and flow velocity that is expected to occur during a 1% AEP flood event.)

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Table 3-12. Manning Roughness Coefficients for Sheetflow (Depth Generally ≤ 0.1 ft)

Surface Description	n-value[Y]
Smooth surface (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤ 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short-grass prairie	0.15
Dense grasses	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods:	
Light underbrush	0.40
Dense underbrush	0.80

Reproduced from Table 15-1 of the NRCS NEH (NRCS, 2010)

[Y]When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheetflow. Note that within the Town, undeveloped natural areas are typically classified as "Range."

3.6.7.2 Shallow Concentrated Flow

The travel time for shallow concentrated flow (T_{scf}) shall be obtained by dividing the travel distance by the velocity determined from the applicable relationships found in Table 3-13.

Table 3-13. Shallow Concentrated Flow Parameters

Flow Type	Depth (feet)	TOV n-Value	Velocity Equation (ft./sec.)
Pavement and Small Upland Gullies[†]	**0.2**	**0.025**	$\boldsymbol{V = 20.328(S_C)^{0.5}}$
Grassed Waterways	0.4	0.050	$V = 16.135(S_C)^{0.5}$
Nearly Bare and Untilled (Overland Flow); and Alluvial Fans in Western Mountain Regions[†]	**0.2**	**0.051**	$\boldsymbol{V = 9.965(S_C)^{0.5}}$
Cultivated Straight Row Crops	0.2	0.058	$V = 8.762(S_C)^{0.5}$
Short-Grass Pasture	0.2	0.073	$V = 6.962(S_C)^{0.5}$
Minimum Tillage Cultivation, Contour or Strip-Cropped, and Woodlands	0.2	0.101	$V = 5.032(S_C)^{0.5}$
Forest with Heavy Ground Litter and Hay Meadows	0.2	0.202	$V = 2.516(S_C)^{0.5}$

*Reproduced from Table 15-3 of the NRCS NEH (NRCS, 2010)

[†]The two bold categories in Table 3-13 are best suited to the hydraulic conditions which typically exist in the Town.

3.6.7.3 Channel Flow

The travel time for channel flow (T_{cf}) shall be obtained by dividing the travel distance by the velocity determined from the Manning Equation for open-channel flows. Manning roughness coefficients for channel flows shall be obtained from Table 4-1 of this DCM, except that for alluvial channels with natural streambeds the minimum Manning n-value selected shall not be less than $n_{min} = 0.2619(S_c)^{1/2}$, where S_c = channel slope, in feet per foot (e.g., if S_c = 0.015; n_{min} = 0.032.)

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3.6.8 Reservoir and Storage Routing

Storage routing is used primarily to simulate reservoirs or stormwater detention structures. HEC-HMS provides thorough documentation on the application of reservoir routing. Designers are encouraged, however, to input their own rating curves for outflow rather than allowing the model to compute pipe flows and overtopping flows. The primary reasoning for this is that models utilize simplified assumptions in computing outflow curves. These assumptions are best suited for situations where outflow is clearly controlled at the entrance to the pipe. If the designer opts to have the models compute pipe flows and overtopping flows, then a supporting curve must be provided in the hydrologic/hydraulic report.

3.6.9 Evaluation of Modeling Results

The Design Engineer shall conduct an independent evaluation of model results via comparison to results using current, applicable USGS Rural RREs and Urban RREs (Paretti et al., 2014; Stricker & Sauer, 1982). If model results are more than 1.50 times higher or less than 0.67 times lower than results obtained using the current, applicable USGS RREs for Rural and Urban areas, the Town may require additional justification in terms of modeling parameters and the technical approach adopted by the Design Engineer.

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References

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Chapter 4. Open-Channel Hydraulics

4.1 Introduction

Open channels serve multiple purposes. In addition to moving floodwaters; they are linear corridors that are essential to wildlife and valued for their recreation potential. They can be a sanctuary for habitat that is both necessary and aesthetically pleasing. For these reasons, it is the policy of the Town to treat open channels as multiple-use areas. Accordingly, it is necessary for designers to consider multiple-use functions when designing open channels. For purposes of implementation, open channels are divided between those that naturally occur and those that are constructed. A naturally occurring channel is one in which the channel bottom is natural, it perhaps has an occasional road crossing, and any bank protection is limited to protecting the existing bank, in place. A constructed channel is one in which the channel banks and bottom have been stabilized (i.e., hardened) and the channel alignment has been straightened and/or shifted to avoid conflict with urban development.

4.2 Criteria

While this chapter discusses Open-Channel Hydraulics, note that it is not the intent of this DCM to encourage modifications to natural channels in order to meet hydraulic design criteria. Table 4-1 summarizes key design criteria and standards for natural channels and design channels that are to be used in the Town. Each of these standards will be discussed in more detail throughout Chapter 4.

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Table 4-1. Natural vs. Design Channels

Policy	Parameter	Standard	Comment
NATURAL CHANNELS	Point of Ingress and Egress to Property	No Change in Location Allowed	
	Increase in WSEL due to Floodplain Encroachment	A Maximum Allowable Increase of 0.1 feet	As Measured Off Site
	Increase in Velocity	Not More than One-Tenth (1/10) of Average Velocity, but Less than 1.0 Foot per Second	As Measured Off Site
	Loss of Flood Storage	Cannot Significantly Impact Flow Rate	Analysis May Be Required (at Discretion of Town)
	Long-Term Degradation and/or Lateral Migration of Stream Channel	Must Not Be Accelerated	
	Future-Condition Flow Rates	Contained within Freeboard, with Lowest FFE at least 1 foot above the Regulatory WSEL	Per the Town Floodplain and Erosion Hazard Management Ordinance (2020)
DESIGN CHANNELS	Maximum Flow Depth	Based upon Applicable Hydraulic Calculations	Must Include Calculated Freeboard
	Maximum Flow Velocity	Based Upon Applicable Hydraulic Calculations	Avoid Supercritical Flow Conditions to Extent Possible
	Side-Slopes	Dependent upon Type of Lining Material(s)	*3H:1V*, or Flatter, for Loose Linings (e.g., Riprap)
	Froude Number (F_r)	Preference: $F_r \leq 0.8$ or $F_r \geq 1.20$	Avoid Unstable Flow, when Possible (i.e., $F_r \neq 1.0$)
	Channel Curvature	Centerline Radius of Curvature Over Top Width (R_c/T) ≥ $4F_r^2$; but use (R_c/T) ≥ 3 whenever $F_r \leq 0.86$	Source: COTDSM (SLA, 1989/1998)
	Freeboard (FB)	$FB = 2.0 + (0.025)(V_p)(Y_p)^{1/3}$ along Designed Channels and Drainageways > 4 feet deep. Otherwise, $FB = 1.0 + 0.25Y_p$. Levees: 3 feet of FB, with 4 feet of FB at and 100 feet upstream or downstream of a bridge low cord or cross-drainage structure; and 3.5 feet, minimum, at the upstream end of the levees.	Sources of FB Equations: Bureau of Reclamation (USBR) (USBR, 2014) and Pima County Department of Transportation (PCDOT) and PCRFCD (PCDOT & PCRFCD, 1984) Levee Requirements from FEMA (FEMA, 2019)
	Lateral Migration	DCM Guidelines for (1) Straight Channels and (2) Channels with Curvature	Adapted from Regional (i.e., COT and Pima County) Guidelines
	Channel Stabilization	Demonstrates that Channel is Stable During Design Flood	
	Riprap and Other Lining Materials	DCM Guidelines	

4.3 Channel Construction and Alterations

Prior to commencing channel construction and alteration, it is required that all permits be obtained from the Town, along with any permits required from either the state of Arizona and/or the federal government. When altering a floodplain that is mapped and appears on a FEMA FIRM, a CLOMR must be submitted to FEMA prior to initiating construction. In addition, the CLOMR

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must be accepted and approved by FEMA prior to issuance of a floodplain use permit by the Town. Conditions which apply to the issuance of a CLOMR follow:

1. The Property Owner or Developer acknowledges and accepts the risk of having homes in the effective Zone A floodplain, requiring flood insurance for an indefinite period of time until a LOMR is granted, if ever.
2. A Type 2 Permit for mass grading will be allowed after:
   a. The site drainage report, bank-protection improvement plans, and CLOMR report have been accepted by the Town (and Pima County as needed); and,
   b. The CLOMR report has been submitted to, and approved by, FEMA.

   For authority, the user should refer to Town Code Section 17-4-3.B.1.d., which allows FPUP's to be issued for structures in pre-LOMR Zone A floodplain areas provided the CLOMR has been accepted and issued by FEMA.
3. The lowest finished-floor elevations of affected structures constructed for human habitation, whether on a full- or part-time basis, shall be elevated to or above the Regulatory Flood Elevation (i.e., one foot above the Base Flood Elevation) of the pre- or post-development flood plain, whichever is higher. For residential structures, the drainage report shall list each affected lot, along with their minimum FFEs and Regulatory Flood Elevations (both pre-development and post-development). Floodplain Use Permits shall be issued, with clear conditions stated, for each affected lot (but not as a block of lots).
4. Two FEMA Elevation Certificates shall be prepared and submitted for each affected lot. The first Elevation Certificate shall be submitted prior to pouring the concrete floor slab (P2S); and the second Elevation Certificate shall be submitted prior to final inspection (P2F). Further construction shall be "on hold" until these two Elevation Certificates have been submitted and accepted by the Town.
5. Clearly inform home buyers and others of this risk and its potential consequences by showing the 100-year floodplain limits for BOTH the effective Zone A and the proposed CLOMR limits on the Final Plat.
6. Furthermore, the following note shall be added to the Final Plat:

   LOTS XX THROUGH YY ARE LOCATED WITHIN A FEMA ZONE(S) [____] SPECIAL FLOOD HAZARD AREA (SFHA), AS SHOWN ON FEMA FLOOD INSURANCE RATE MAP (FIRM) PANEL NUMBER ____________________, WITH EFFECTIVE DATE [DATE]. A CONDITIONAL LETTER OF MAP REVISION (CLOMR) (CASE # 09-XX-XXXX) HAS BEEN APPROVED BY THE TOWN OF ORO VALLEY AND SUBMITTED TO FEMA ON [DATE] WITH THE INTENT OF REMOVING THESE LOTS FROM THE FEMA FLOODPLAIN. BECAUSE THE CLOMR DOES NOT GUARANTEE THAT THESE LOTS WILL BE REMOVED FROM THE FEMA FLOODPLAIN, UNLESS AND UNTIL THE EFFECTIVE FEMA FIRM HAS BEEN REVISED, TOWN OF ORO VALLEY REVIEW AND APPROVAL OF BUILDING PERMITS FOR THESE LOTS IS REQUIRED PRIOR TO PERMIT ISSUANCE. IF THE EFFECTIVE FIRM

HAS NOT BEEN REVISED, FLOODPLAIN USE PERMITS AND ELEVATION CERTIFICATES FOR ALL STRUCTURES SHALL BE REQUIRED. FEDERAL LAW REQUIRES THAT A FLOOD INSURANCE POLICY BE OBTAINED AS A CONDITION OF A FEDERALLY-BACKED MORTGAGE OR LOAN THAT IS SECURED BY A BUILDING LOCATED WITHIN AN EFFECTIVE SFHA.

REVISING THE EFFECTIVE FIRM REQUIRES COMPLETION OF THE INFRASTRUCTURE SHOWN IN THE CLOMR; PREPARATION OF AS-BUILT PLANS FOR THE INFRASTRUCTURE; TOWN OF ORO VALLEY AND FEMA REVIEW AND APPROVAL OF THE AS-BUILT PLANS; FEMA ISSUANCE OF A LETTER OF MAP REVISION (LOMR); AND COMPLETION OF A PUBLIC COMMENT PERIOD. UNLESS AND UNTIL THE EFFECTIVE FIRM HAS BEEN REVISED, THERE IS SOME RISK TO AN OWNER OF ANY LOTS LOCATED WITHIN THE EFFECTIVE FEMA FLOODPLAIN THAT THE LIMITS OF THE FLOODPLAIN WILL NOT BE REVISED.

7. After the LOMR has been approved by FEMA, an Administrative Plan Note shall be added to the recorded Final Plat and then re-recorded. Suggested wording for this Administrative Plat Note follows:

   THE EFFECTIVE FEMA FLOODPLAIN SHOWN ON FIRM PANEL ___________________________ HAS BEEN REVISED BY LOMR CASE NUMBER 09- XX-XXXX, WITH EFFECTIVE DATE [DATE]. THE REVISED EFFECTIVE FLOODPLAIN IS SHOWN AS THE POST-DEVELOPED FLOODPLAIN ON THE PLAT, AND TOWN OF ORO VALLEY REVIEW AND APPROVAL PRIOR TO ISSUANCE OF BUILDING PERMITS FOR LOTS XX THROUGH YY IS NO LONGER REQUIRED.

It is the policy of the Town that any proposed channel construction/modifications do not result in adverse flood-related impacts to other property owners. The following criteria shall apply for all proposed channel construction/modifications:

1. Channel construction/modifications shall not alter the historical point where flows enter and leave the affected property.
2. Mass fill and grading within the channel and associated floodplain shall not result in the increase of existing off-site-condition WSELs that are in excess of 0.1 feet, as determined by a backwater analysis.
3. Any excess increase in flow velocity shall not occur offsite of the property on which channel modifications are constructed. An "offsite excess increase in flow velocity" is defined as a post-construction velocity in excess of 10% of the average pre-construction velocity; except that, in all cases, no offsite increase in flow velocity shall exceed 1.0 foot per second.
4. Any fill placed in a regulated floodplain shall require an analysis that demonstrates that there will be no significant increase in WSELs (pursuant to the Town Floodplain and Erosion Hazard Management Ordinance [2020]) and no increase in flow rates.
5. When a natural channel is to be straightened or stabilized (e.g., grade controlled or bank protected), a sediment-transport or fluvial-geomorphology analysis is to be prepared. The analysis shall demonstrate that the alteration of the natural channel will not lead to long-

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term channel degradation or aggradation that is in excess of that which is expected to occur under existing conditions; or that the alteration will not lead to the exacerbation of lateral channel migration or change the watershed hydrology, resulting in increased flow rates.

6. The analysis of an open channel shall consider future-condition flow rates that may result due to upstream development either within or outside of the Town. The future-condition flow rates shall be contained within the freeboard of a constructed channel; or, a natural floodplain with future-condition flow rates shall establish future flood levels. All FFEs of habitable structures shall be set at least one foot above existing Base Flood Elevations (BFEs) of the 1% AEP design flood (i.e., to Regulatory Flood Elevations [RFEs]). FFEs should not be lower than RFEs.

   In addition, it is the responsibility of the designer to comply with all other applicable local, state, or federal permits.

In the event that adverse impacts cannot be avoided, modifications can proceed providing that one of the following has occurred:

1. Adversely impacted property owners have provided written permission in the form of an easement or other similar vehicle.
2. The modification is part of a regional plan that has undergone appropriate public notice and is adopted by the Town Council.
3. Another form of structural or nonstructural mitigation is provided.

4.4 Channel Analysis and Design

Constructed channels should be designed to convey the 1% AEP design flood, with sufficient freeboard to accommodate hydraulic uncertainties. Velocities for channels with outfalls into natural watercourses should not exceed the recommended velocities listed in this DCM. Channel side-slopes typically should be 3H:1V, or flatter. Exceptions include:

1. Authorization given by the Town based on demonstration that the channels will incorporate proper safety measures, that the channel has proper maintenance access, and that the channel can be maintained.
2. The channel modification is defined as the stabilization of a natural wash bank.
3. The constructed channel is similar in dimension to the natural channel it replaces.

The Town prefers the use of backwater methods for establishing velocity and WSELs along natural watercourses and designed open channels. Computer models such as HEC-2 and HEC-RAS, as well as other similar backwater models, may be used in lieu of hand calculations or spreadsheets. Normal-depth methods (e.g., the Manning Equation) may be used for sizing of small prismatic channel sections (less than 400 cfs), provided that backwater is not anticipated. However, if normal-depth computations are used, the WSELs calculated should be based upon the flow depth, Y, plus the velocity head, $\frac{v^2}{2g}$ (i.e., the specific head, $Y + \frac{v^2}{2g}$). Examples of potential backwater situations would include tributaries, crossings, grade breaks, and variations in the cross-section, to name a few.

The designer should include the Froude numbers with the calculations for the designed channel sections. Channel flows that produce Froude numbers in the range of 0.8 to 1.2 during a 1% AEP design flood should be avoided, if and when possible. Otherwise, it must be demonstrated that the flow profile is stable; or, with adequate freeboard remains confined within the channel banks. That is, if a natural channel or designed channel has a Froude number approaching 1, the designer is to evaluate and present the implications of wave instabilities (e.g., surface waves, such as antidunes) as they relate to changes in WSELs and energy-loss potential. If it is not possible for the designer to avoid this constraint, the designer must provide adequate freeboard to confine the design flood within the channel, and indicate potential areas where wave disturbances and weak hydraulic jumps might occur.

When designing a channel curve, superelevation effects can increase the water surface and must be considered. In order to mitigate superelevation effects, the maximum centerline radius of curvature (R_c), in feet, should be determined from the relationship $R_c = 4F_r^2 T$, where T is the channel top width, and F_r is the Froude number. Note that in no case should R_c be less than $3T$ when $F_r \leq 0.86$.

Freeboard is to be provided in all channel and levee designs. Freeboard (FB), in feet, along straight channels (i.e., with no curvature) which are greater than 4 feet in depth should be determined by use of the following equation (USBR, 2014):

$$FB = 2.0 + 0.025(V_p)(Y_p)^{1/3} \qquad \textit{(Equation 4.1)}$$

Where,

V_p = Velocity of flow at peak discharge, in feet per second

Y_p = Depth of flow at peak discharge, in feet.

Equation 4.1 is typically associated with a 1% AEP design discharge, and in intended to account for near-critical and supercritical flow conditions, for flow surface roughness, for wave action, for air bulking, for splash, and for spray.

Freeboard along straight channels (i.e., with no curvature) which are less than 4 feet in depth should be determined by use of the following equation (PCDOT & PCRFCD, 1984):

$$FB = 1.0 + 0.25Y_p \qquad \textit{(Equation 4.2)}$$

When $Y_p \equiv 4.0$ feet, *Equation 4.1* should be used.

Flood-control levees are to have a minimum of 3 feet of freeboard, including at least 3.5 feet of freeboard at their upstream ends; and 4 feet of freeboard at and 100 feet upstream and downstream of bridges and cross-drainage structures.

Lateral-migration setbacks for natural and existing channels are to be established based upon DCM guidelines as discussed in Chapter 5, which are adapted from regional (i.e., COT and Pima County) guidelines.

The designer of constructed channels is responsible for demonstrating the need, or lack thereof, for bank protection or invert stabilization. As most watercourse slopes in the Town are steep, it is assumed that most constructed channels will require some form of bank stabilization. Designers are encouraged to soften the "engineered" appearance of these constructed channels. The Town will also consider bioengineered stabilization measures, landscaped channels, channels with stabilization features that are buried and then covered and landscaped, or channels designed to mimic a natural channel using the methods of fluvial geomorphology to establish both a low-flow and a floodplain channel section.

4.5 Guidelines for Construction of Open Channels

The open-channel guidelines (including most Equations and Figures) contained within this section of Chapter 4 were excerpted directly from Chapter VIII, Section 8.5, of the COTDSM (SLA, 1989/1998), with the exception of minor revisions required in order to update the information contained in this chapter, where needed. When equations in this chapter are sourced from references other than the COTDSM, it is appropriately noted herein.

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 allow 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 guidelines for the design of constructed open channels.

4.5.1 Channel Cross-Section 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 Town.

4.5.2 Side-Slopes

Side-slopes for constructed earthen or riprap channels shall be no steeper than 3H:1V, unless an approved soils analysis demonstrates that steeper side-slopes are stable. Side-slopes for lined channels may be steeper, depending upon the soils analysis and structural stability of the lining, although a maximum side-slope for lined channels should not exceed 1H:1V unless structurally reinforced concrete lining is used, in which case vertical side-slopes may be used provided that the design is adequate to prevent failure from hydrostatic or earth pressures. For steep side-slopes (i.e., steeper than 3H:1V), safety barriers shall be required along channel perimeters. Shotcrete may be placed on side-slopes as steep as 1H:1V; but only if these side-slopes are not significantly steeper than the natural angle of repose of the soil. A stair-stepped soil-cement lining may be placed on 1H:1V side-slopes, provided it is of sufficient thickness to be structurally stable. The minimum thickness of soil cement on a 1H:1V 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 4H:1V. Actually, for ease of construction, even flatter side-slopes (e.g., 6H:1V) are desirable under such circumstances.

4.5.3 Width

Ordinarily, the minimum bottom width of a channel must be ten feet before it will be accepted for maintenance by the Town. Occasionally, bottom widths as narrow as eight feet may be allowable in certain cases, with prior approval from the Town. 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); but it is highly recommended that privately-maintained channels meet the same minimum-width requirements within the Town as required for publicly maintained channels.

The bottom-width of constructed channels which lack bed and/or bank protection should not vary by more than fifteen percent in width 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 (l) 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 1% AEP peak discharge, would convey the more-frequent discharges (e.g., 2-year flood) at very shallow depths, assuming there would be an equal flow distribution across the entire flow cross-section. However, due to the laws of physics, 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 the more frequent discharges. 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, when practicable:

$$\frac{b}{(V_{P1\%\,AEP})(Y_{P1\%\,AEP})} \leq 1.15 \qquad \textit{(Equation 4.3)}$$

Where,

b	=	Bottom width of channel, in feet.
$V_{P1\%AEP}$	=	Average velocity of flow at the peak of a 1% AEP flood, in feet per second.
$Y_{P1\%AEP}$	=	Average depth of flow at the peak of a 1% AEP, in feet.

See Section 6.6.3 of the COTDSM.

4.5.4 Depth

The peak depth of flow in channels, where relatively steady uniform-flow conditions exist, can be computed by an iterative solution of the Manning Equation:

$$Q_p = \frac{1.486}{n} AR^{2/3} S_f^{1/2} \qquad \textit{(Equation 4.4)}$$

Where,

Q_p	=	Peak discharge, in cubic feet per second.
n	=	Manning roughness coefficient (see Table 4.2).
R	=	Hydraulic radius of flow (= A/WP), in feet.
S_f	=	Friction slope, in feet per foot.
A	=	Cross-sectional area of flow, in feet.
WP	=	Wetted perimeter of flow, in feet.

The depth of flow is implicit in *Equation 4.4*, within the terms A and R. In order 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, WP, and R from the channel cross-section characteristics; then solve for Q using the Manning 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 to each other.

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 the Manning Equation.

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It is noted that uniform flow does not exist under most design conditions, due to disturbances caused by changes in the channel width, discharge, or slope. Furthermore, the presence of channel bends, transitions, junctions, or obstructions (such as culverts), can create conditions which lead to nonuniform flow. The effect of such disturbances can propagate far upstream, or downstream, depending upon whether or not the flow regime 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.

4.5.5 Manning Roughness Coefficients (n-values)

Table 4-2 on the following page provides a listing of Manning roughness coefficients (n-values) that are typically used in conjunction with various types of channel materials. The designer must take into account the increased n-values that are associated with sediment-laden flows. In addition, the user must keep in mind that the minimum Manning n-value to be selected for use with natural alluvial channels and constructed channels with alluvial bottoms shall not be less than $0.2619(S_c^{0.5})$, where S_c = the channel slope, in feet per foot (i.e., with an alluvial channel bottom, the steeper the channel slope the greater should be the minimum n-value).

For concrete-lined channels and smooth-lined culverts, it is important to recognize that the transport of significant bed load (i.e., bed-material sediment in contact with the channel or culvert bottom) will produce a Manning n-value representative more so of the bottom sediments than representative of the existing lining material. Therefore, unless the velocity in the concrete-lined channel or smooth-lined culvert exceeds the minimum velocity required, as calculated using *Equation 4.5*, and thus suspends all bed-material sediments in the vertical flow profile, it should be assumed that 0.020 is the minimum Manning n-value to be used for computing the hydraulics within the barrels of concrete-lined channels and smooth-lined culverts conveying sediment-laden flows.

$$V_{min} = 13.8771Y^{(0.5538 - 0.2140LogY)} \qquad \textit{(Equation 4.5)}$$

Where V_{min} = minimum velocity required to suspend all bed-material sediments in the vertical flow profile of a concrete-lined channel or a smooth-lined culvert, in ft./sec.; and Y = depth of flow, in ft. ($Y \leq 8$ ft). *Equation 4.5* is a best-fit regression equation, adapted from *Velocity to Prevent Sedimentation, Based on Sediment Size* which is contained in ASCE Manual 54, *Sedimentation Engineering* (1975). For the development of *Equation 4.5*, a D_{85} of 7 mm was selected as the characteristic sediment size, which more or less represents an upper limit on the D_{85} sediment sizes found in alluvial channels within the Town.

Table 4-2. Manning Roughness Coefficients (n-values)†

CHANNEL MATERIAL	Roughness Coefficient (n)*		
	Minimum	Normal	Maximum
Corrugated Metal	0.021	0.025	0.030
Concrete (no bed-material sediment in the flow)			
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
Concrete (bed-material sediments in the flow)	0.017	0.020§	0.022
Asphalt (no bed-material sediments in the flow, and assumes cars are present under normal conditions)	0.016	0.020¶	0.022
Soil Cement	0.018	0.020	0.025
Riprap (bottom and sides, where D_{50} is in feet)	-------	$0.04(D_{50})^{1/6}$	-------
Constructed Channels, with earth or sand bottoms and 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 bottoms and side of:			
1) Trees and Shrubs	0.025	0.035	0.045
2) Rock	0.024	0.032	0.040
Natural Channels, braided with shallow flows (< 2 feet)	0.030	0.045	0.080
Natural Channels, with rock bottoms (cobbles/boulders)	0.040	0.060	0.090
Overbank flood plains			
1) Desert brush, normal density	0.040	0.060	0.080
2) Dense vegetation	0.070	0.100	0.160

†Note: For an unlined alluvial channel bottom, the minimum n-value should not be less than $0.2619(S_c^{0.5})$.
*Most table n-values adapted from Chow (1959) and Aldridge and Garrett (1973).
§Adapted from the Corps (USACE, 1994).
¶Assumes that cars are parked curbside, disrupting flow patterns.

Another issue related to the selection of Manning n-values is the physical aging process of fluvial systems. Accordingly, for all newly-constructed developments "normal" Manning n-values should be selected by the Design Engineer using guidance from Table 4-2 (for example, when performing scour calculations or designing bank-stabilization measures). When delineating flood plains, however, the Design Engineer should use the "maximum" Manning n-values listed in Table 4-2 in order to account for possible channel changes and other unknowns occurring in the future (e.g., hydrologic uncertainty, increased vegetal growth, and watercourse aggradation). This is especially true for braided, shallow watercourses. Alternatively, the Design Engineer should use "minimum" recommended Manning n-values for conducting sediment-transport analyses.

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4.5.6 Backwater Calculations

Backwater calculations proceed in an upstream direction for subcritical flow and in a downstream direction for supercritical flow. A "control section" must be established for computations to begin. A control section is a section at a place of known WSEL. Control sections can be at such places as channel confluences, culvert inlets, or at a location where the flow goes through critical depth. Critical depth occurs when the Froude number (F_r) is equal to 1.0.

The Froude number is calculated from:

$$F_r = \frac{V}{(gY_h)^{1/2}} \qquad \textit{(Equation 4.6)}$$

Where,

V	=	Average velocity of flow, in feet per second.
g	=	Acceleration due to gravity (= 32.2 feet per second-squared).
Y_h	=	Hydraulic depth of flow (i.e., area/top width), in feet.

Equation 4.6 should be used with care whenever there is overbank flooding or nonuniform variations which cause the flow velocity to vary horizontally within the cross-section. In such cases, critical depth should be estimated (see, for example, the graphical method of solution 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 is represented by 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 channel slope, as well as 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 a computer (e.g., HEC-RAS) or hand-held calculator. For those interested in performing these calculations manually, a very good discussion and description of the Direct Step Method can be found on Page 262 of Chow (1959).

4.5.7 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 incorporate a safety factor that accounts for unanticipated hydraulic disturbances such as waves, unforeseen 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 against (2) floods that are larger than the design flood. The freeboard requirement for channels located within the Town which are greater than 4 feet in depth should be computed using *Equation 4.1*; while for channels less than 4 feet in depth, *Equation 4.2* should be used.

The freeboard requirements described above are for uniform channel reaches where no unusual flow disturbances are anticipated. Additional freeboard is required at channel bends to account for superelevation, at 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 Subsections 4.5.21 and Subsection 4.5.26 of this chapter. At those locations where a hydraulic jump could form, additional freeboard shall be provided in order to contain the jump in accordance with the guidelines provided in Subsection 4.5.14 of this chapter.

Along regional watercourses within the Town, such as the Cañada del Oro Wash, freeboard 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 Town.

4.5.8 Safety Considerations

Deep open 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 open-channel design. The design of hazardous open channels should be avoided, if possible. All channels greater than four-feet deep which have side-slopes steeper than 2H:1V shall have emergency escape ladders consisting of a series of iron rungs located every 600 feet apart on each channel bank. Other site-specific safety measures shall be installed, as deemed necessary, by either the Design Engineer or the Town.

4.5.9 Rights-of-Way

All channels that are to be maintained by the Town must be dedicated to the Town of Oro Valley. Dedication may be either in fee title or in the form of an easement. The width of a publicly-dedicated channel right-of-way shall be the width of the channel, including key-ins, plus the width of a maintenance access lane located on each side of the channel (Note: privately-maintained open channels shall also require an adequate access lane for maintenance purposes). The minimum maintenance access width for regional watercourses is fifty 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 (i.e., where the 1% AEP is greater than 1,000 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 Town, 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-wide 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 a drainage master plan(s) recommends a particular channel alignment, or an alignment 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 drainage master plan(s), 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 indicate that there are no conflicts or adverse effects with existing upstream and/or downstream drainage improvements.

4.5.10 Bank-Protection Key-Ins and Minor Side Drainage

Bank-protection key-ins refer to the additional material provided at and beneath the surface of the ground at the top of the bank protection in order to prevent undermining due to interflow and seepage otherwise created by overbank flows cascading down (and behind) channel banks. 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 4-1. 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 can be justified. Key-ins for stair-stepped 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 that is acceptable to the Town.

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.

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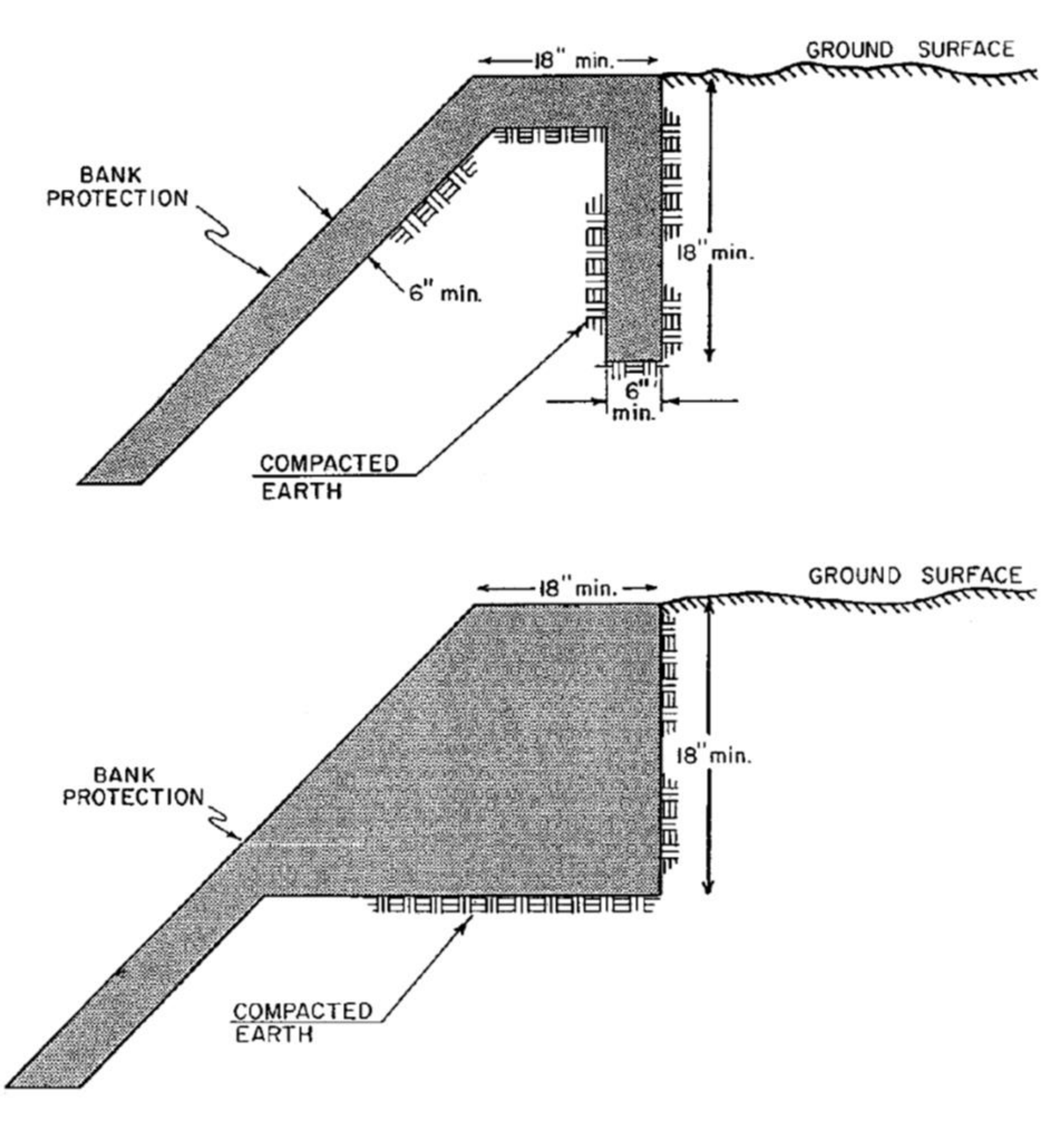

Figure 4-1. Typical Bank Protection Key-In (Not-To-Scale)

4.5.11 Bank-Protection Toedowns

Bank-protection toedowns refer to the extension of bank protection below the channel bed to prevent undermining due to scour. Although shallow (i.e., 6.0 feet) toedowns normally can be vertical, they sometimes are extended below the channel bottom at the same side-slope as the bank itself. Since the purpose of a toedown is to prevent failure of the bank protection due to scour (including long-term degradation) of the channel bed, bank-protection toedowns shall extend to the combined depth associated with general scour, bend scour, local scour, low-flow-thalweg 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 5 of the DCM. Immediately downstream of any grade-control structure, the toedown shall conform to the geometry of the scour hole, determined by the method also presented in Chapter 5 of the DCM.

4.5.12 Upstream and Downstream Controls

The upstream end of constructed channels must be designed to collect the entire design discharge without raising WSELs on adjacent properties. This may be accomplished by providing wide entrance transitions, collector channels, or guide banks at the upstream end. See Subsection 4.5.25.1 of this chapter for information on entrance transitions.

The downstream end of constructed channels 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 appropriately-sized box culverts at street crossings. See Subsection 4.5.25.2 of this chapter 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 Town is obtained from all affected property owners.

4.5.13 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 the 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 concrete bottoms may be flatter, but not significantly so. 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.

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Whenever possible, channels should be designed to convey the incoming sediment supply without causing aggradation or degradation. Refer to Chapter 5 of this DCM, which addresses erosion and sedimentation, for more detailed information.

Channels with design Froude numbers between 0.86 and 1.13 should be avoided, if at all possible, because of the instability associated with flow at or near critical conditions.

Most channels located within the Town that have earthen beds are constructed on slopes that are steeper than their equilibrium slopes. In such cases, grade-control structures are typically required. Refer to Chapter 5 of the DCM for grade-control design guidelines.

4.5.14 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 the outlets of drainageway 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 highly 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 properties.

The type of hydraulic jump that forms, and the amount of energy it dissipates, is dependent upon the upstream Froude number, F_{ru} (see *Equation 4.6* for definition of the Froude number).

The various types of hydraulic jumps that can occur are listed below:

<u>Undular Jump</u>: $1 < F_{ru} \leq 1.7$. This jump produces an energy loss that varies from 0% to 5%.
<u>Weak Jump</u>: $1.7 < F_{ru} \leq 2.5$. This jump produces an energy loss that varies from 5% to 18%.
<u>Oscillating Jump</u>: $2.5 < F_{ru} \leq 4.5$. This jump produces an energy loss that varies from 18% to 44%.
<u>Steady Jump</u>: $4.5 < F_{ru} \leq 9$. This jump produces an energy loss that varies from 44% to 70%.
<u>Strong Jump</u>: $F_{ru} > 9$. This jump produces an energy loss that varies from 70% to 85%.

4.5.15 Height of 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 (i.e., the height of the hydraulic jump) can be computed by use of the following equation:

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$$Y_2 = \frac{1}{2} Y_1 ([1 + 8F_{ru}^2]^{\frac{1}{2}} - 1)$$ *(Equation 4.7)*

Where,

Y_1 = Initial (upstream) flow depth, in feet.

Y_2 = Sequent (downstream) flow depth, in feet (i.e., the jump height).

F_{ru} = Froude number upstream of hydraulic jump (see *Equation 4.6*).

The sequent depth of a hydraulic jump in a trapezoidal channel is much more complicated than for a rectangular channel. In general, open channels should not be designed with jumps in trapezoidal sections because of complex flow patterns and increased jump lengths that would otherwise occur. However, the solution for sequent depth in trapezoidal channels can be obtained from a trial-and-error solution of *Equation 4.8*. *Equation 4.8* is derived from momentum equations (see Morris & Wiggert [1972]). It is also acceptable, for design purposes, to determine the sequent depth in trapezoidal channels from *Equation 4.7*. *Equation 4.7* is much simpler to solve, and produces only slightly greater values for sequent depth for trapezoidal channels than does *Equation 4.8*.

$$\frac{ZY_1^3}{3} + \frac{bY_1^2}{2} + \frac{Q^2}{gA_1} = \frac{ZY_2^3}{3} + \frac{bY_2^2}{2} + \frac{Q^2}{gA_2}$$ *(Equation 4.8)*

Where,

Y_1 & Y_2 are as defined in *Equation 4.7*.

b = Channel bottom width, in feet.

Z = Channel side-slope (horizontal to vertical), in feet per foot.

Q = Channel discharge, in cubic feet per second.

A_1 = Cross-sectional area of flow upstream of the hydraulic jump, in square feet.

A_2 = Cross-sectional area of flow downstream of the hydraulic jump, in square feet.

Figure 4-2 and Figure 4-3 can also be used to determine the height of a hydraulic jump.

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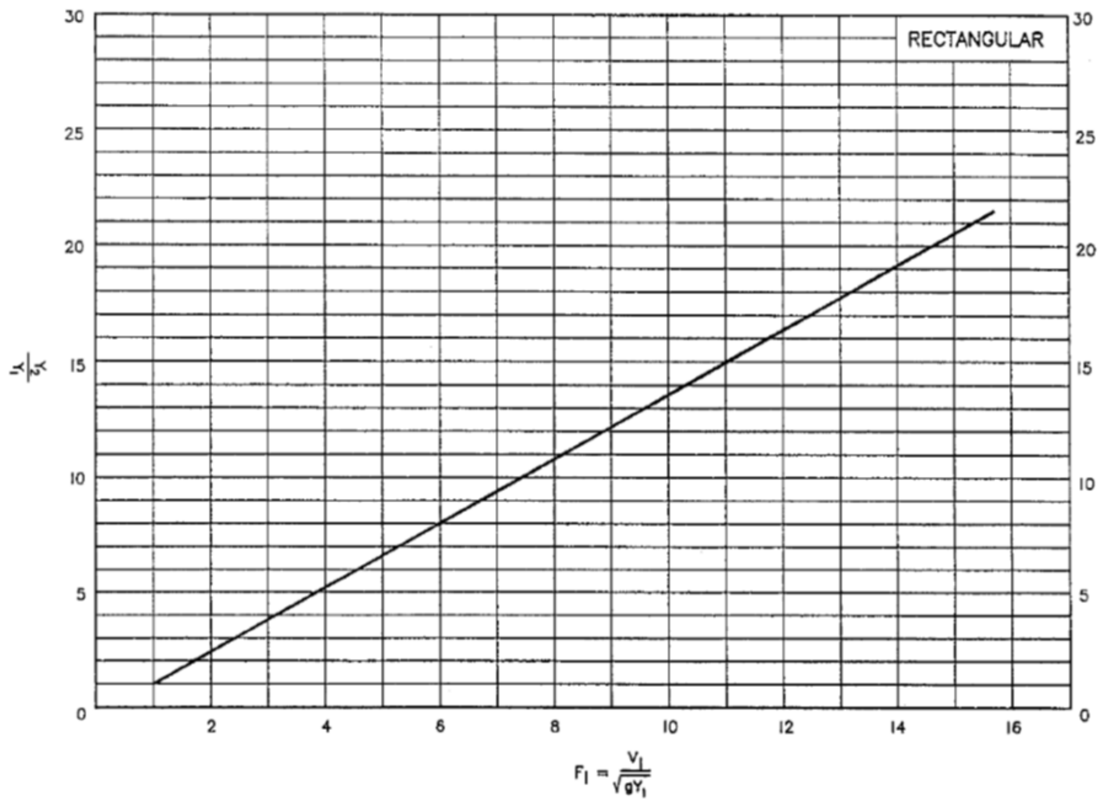

Figure 4-2. Height of a Hydraulic Jump for a Horizontal, Rectangular Channel

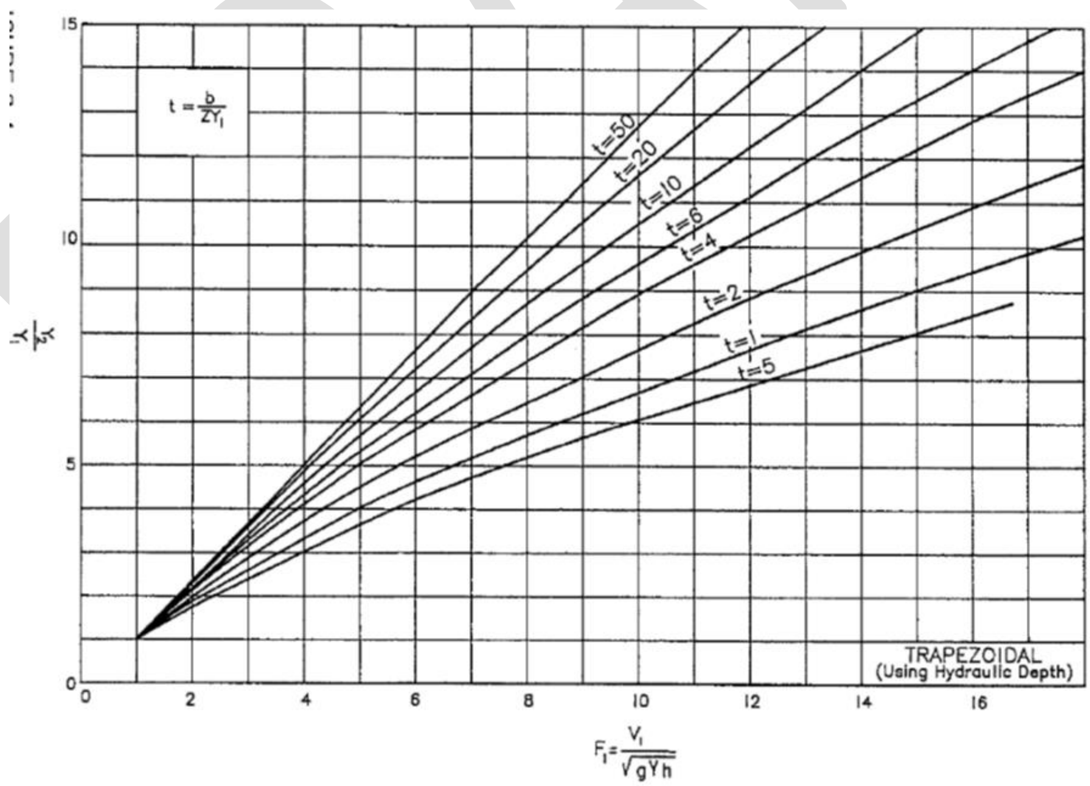

Figure 4-3. Height of a Hydraulic Jump for a Horizontal, Trapezoidal Channel (Using Hydraulic Depth)

4.5.16 Length of 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 Figure 4-4 and Figure 4-5.

4.5.17 Surface Profile of a Hydraulic Jump

The surface profile of a hydraulic jump may be needed in order to design the profile of additional bank protection, or training walls, required for the purpose of containing the jump. The surface profile can be obtained from Figure 4-6.

4.5.18 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 4.7*. 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.

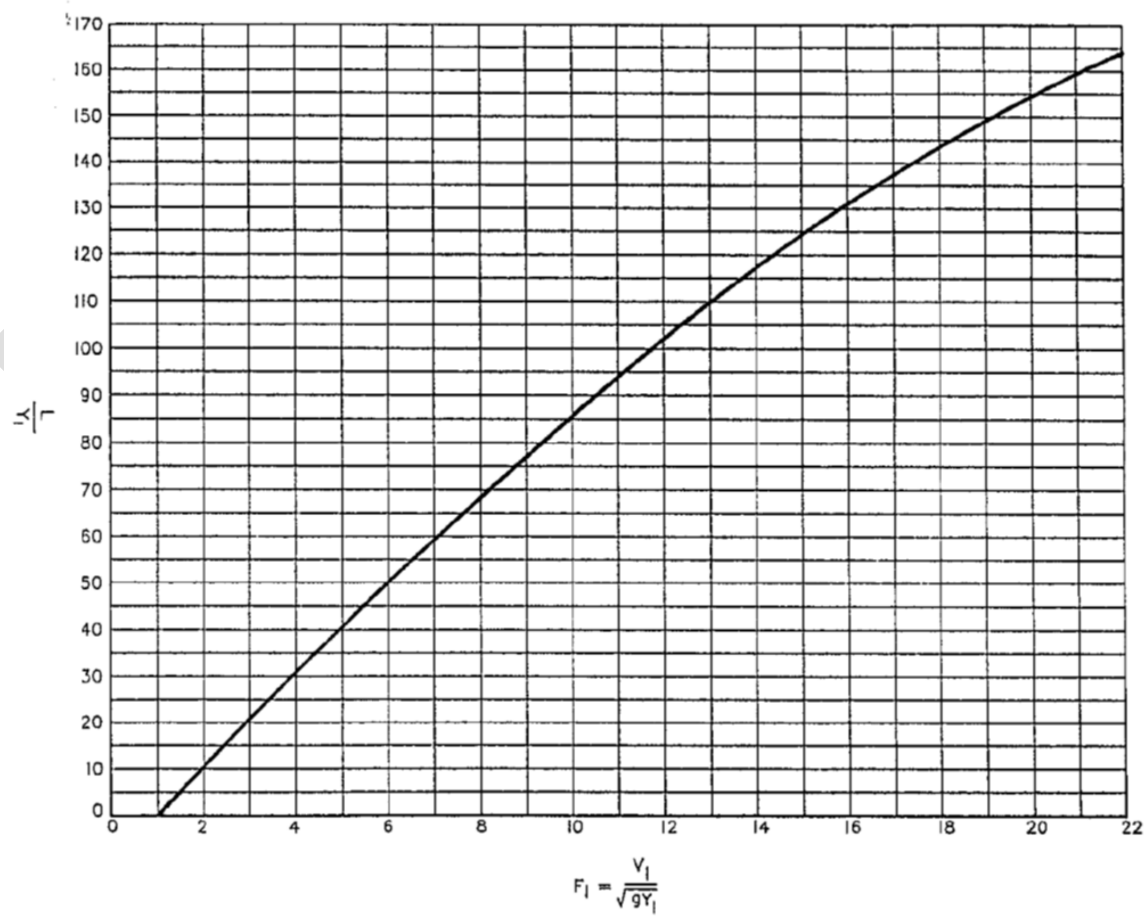

Figure 4-4. Length of a Hydraulic Jump for a Rectangular Channel

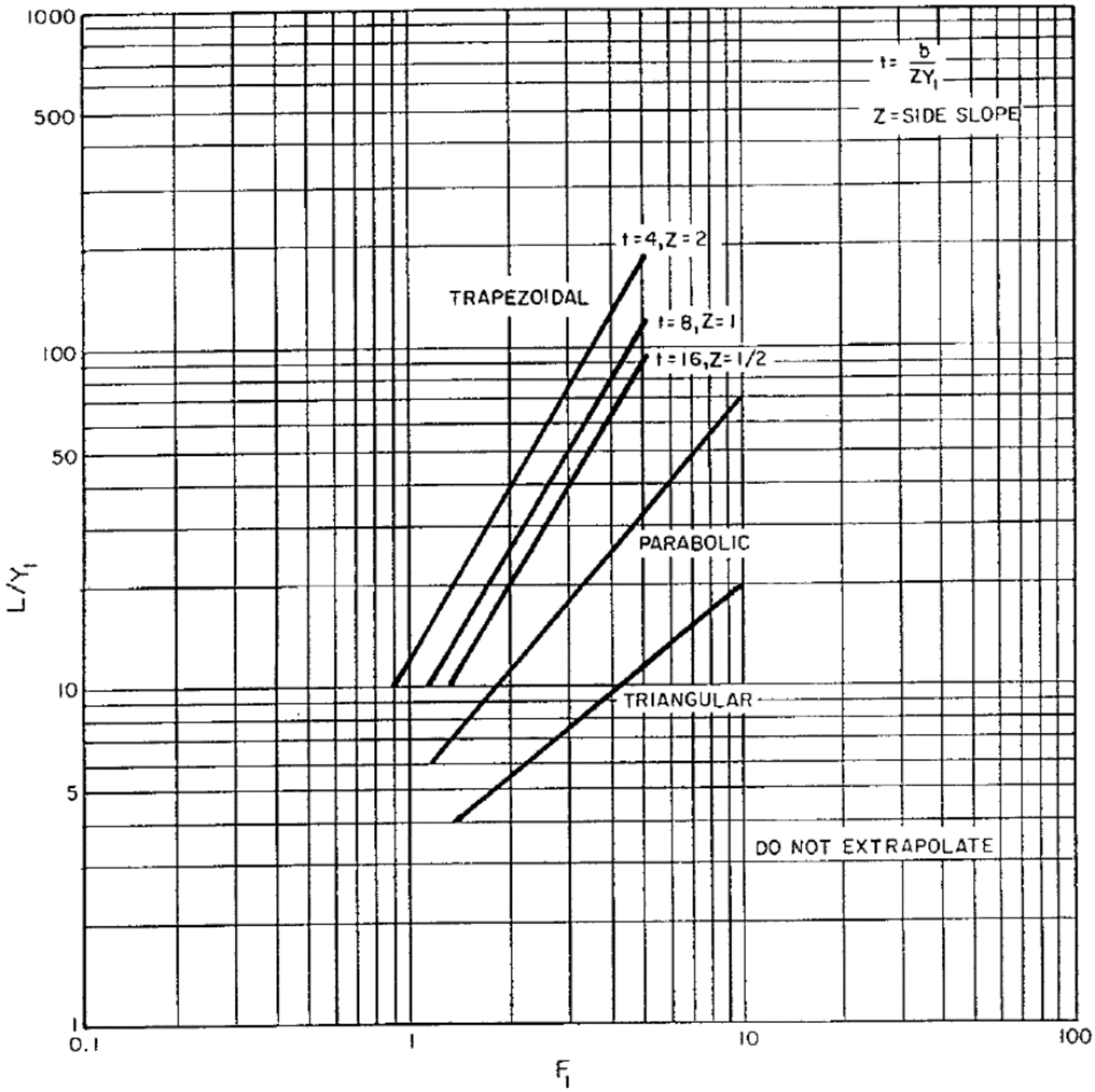

Figure 4-5. Length of a Hydraulic Jump for Different Nonrectangular Channels

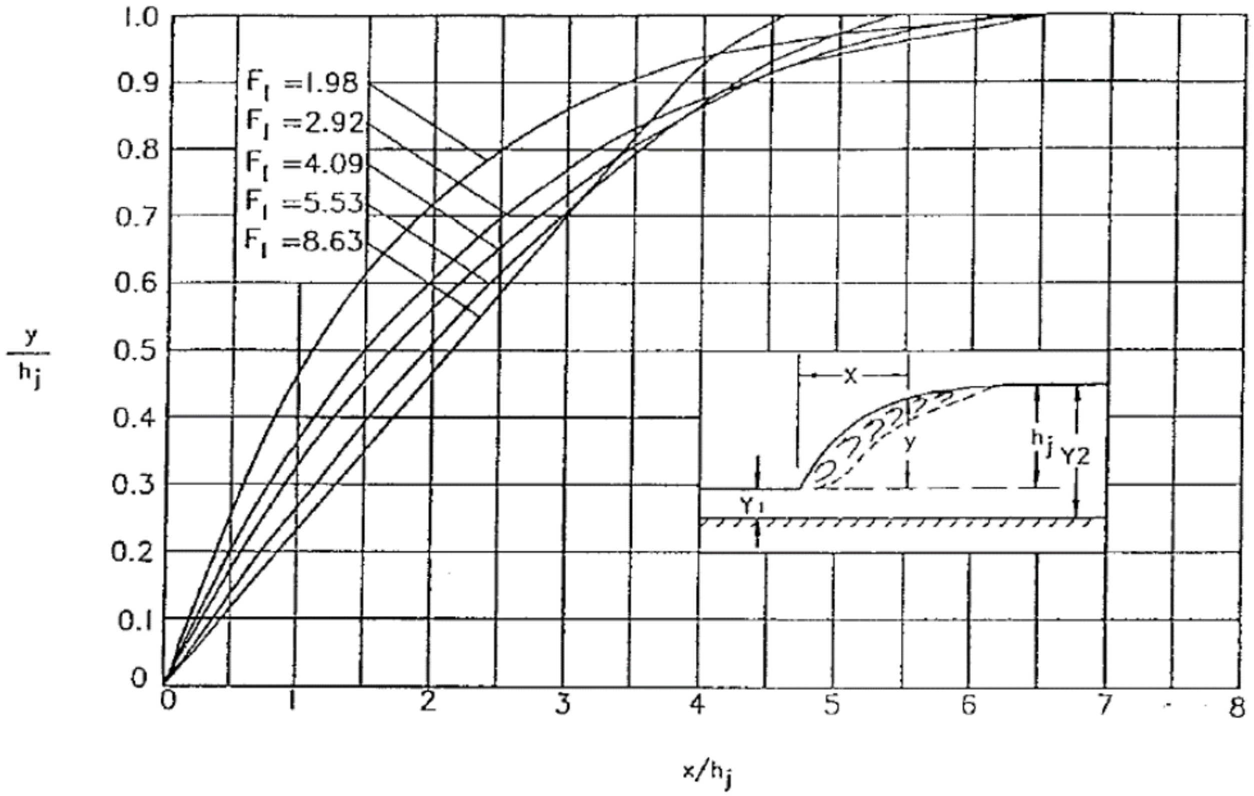

Figure 4-6. Surface Profile of a Hydraulic Jump

4.5.19 Undular Hydraulic Jump

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 4.7*. Therefore, the height of this wave should be determined from the following equation (USACE, 1994):

$$\frac{Y_2}{Y_1} = F_{ru}^2 - 1 \qquad \textit{(Equation 4.9)}$$

Where all terms are as previously described. (See Corps publication EM 1110-2-1601 [USACE, 1994] for source of *Equation 4.9*.)

4.5.20 Flow in a Channel with Curvature

Flow in a channel with curvature will create centrifugal forces, causing a rise in the water surface along the concave bank (e.g., along the outside of a bend). At the same time, a corresponding depression will be created in the water surface along the convex bank (e.g., along the inside of a bend). In addition, spiral secondary currents tend to form within the bends. These secondary currents can cause scour to occur along the concave bank, and cause deposition to occur along the convex bank. 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.

4.5.21 Superelevation

Superelevation is the rise in the WSEL along a concave bank of a channel in curvature (i.e., along the outside of a channel bend), with an accompanying lowering of the water surface along the convex bank (i.e., along the inside of a channel bend). The outside rise in the water surface is generally measured with respect to the mean depth of flow in an equivalent straight reach of the channel, usually located immediately upstream of the beginning of curvature. Additional freeboard is required along the concave bank (e.g., the outside of a channel bend) in order to account for this rise (see Figure 4-7). Superelevation is computed as follows:

$$\Delta Y = \frac{1.5CV^2T}{gR_c} \qquad \textit{(Equation 4.10)}$$

Where,

ΔY = Rise in WSEL (superelevation) around the outside of a channel bend, in feet.

C = A coefficient (see Table 4.3).

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 feet per second-squared.

R_c = Radius of curvature of channel centerline, in feet.

1.5 = Factor-of-safety to account for alluvial-channel flow conditions.

(See Corps publication EM 1110-2-1601 [USACE, 1994] for source of *Equation 4.10*.)

Table 4-3. Coefficients for Use in *Equation 4.10*

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 contained in Corps publication EM 1110-2-1601 (USACE, 1994). 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 4-7 shows a typical cross-wave pattern. The central angle of the cross-wave pattern, θ, is computed by use of the following equation:

$$\theta = tan^{-1}\left[\frac{2b}{(2R_c+b)tan\beta}\right] \quad \textit{(Equation 4.11)}$$

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 $= sin^{-1}\left(\frac{1}{F_r}\right)$, in degrees.
F_r	=	Froude number.

(See *Engineering Hydraulics* [Rouse, 1950] for source of *Equation 4.11*.)

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 4.12*.

$$L' = \frac{3T}{tan\beta} \quad \textit{(Equation 4.12)}$$

Where,

L'	=	Maximum distance of superelevation downstream of curvature 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 $0.67L'$.

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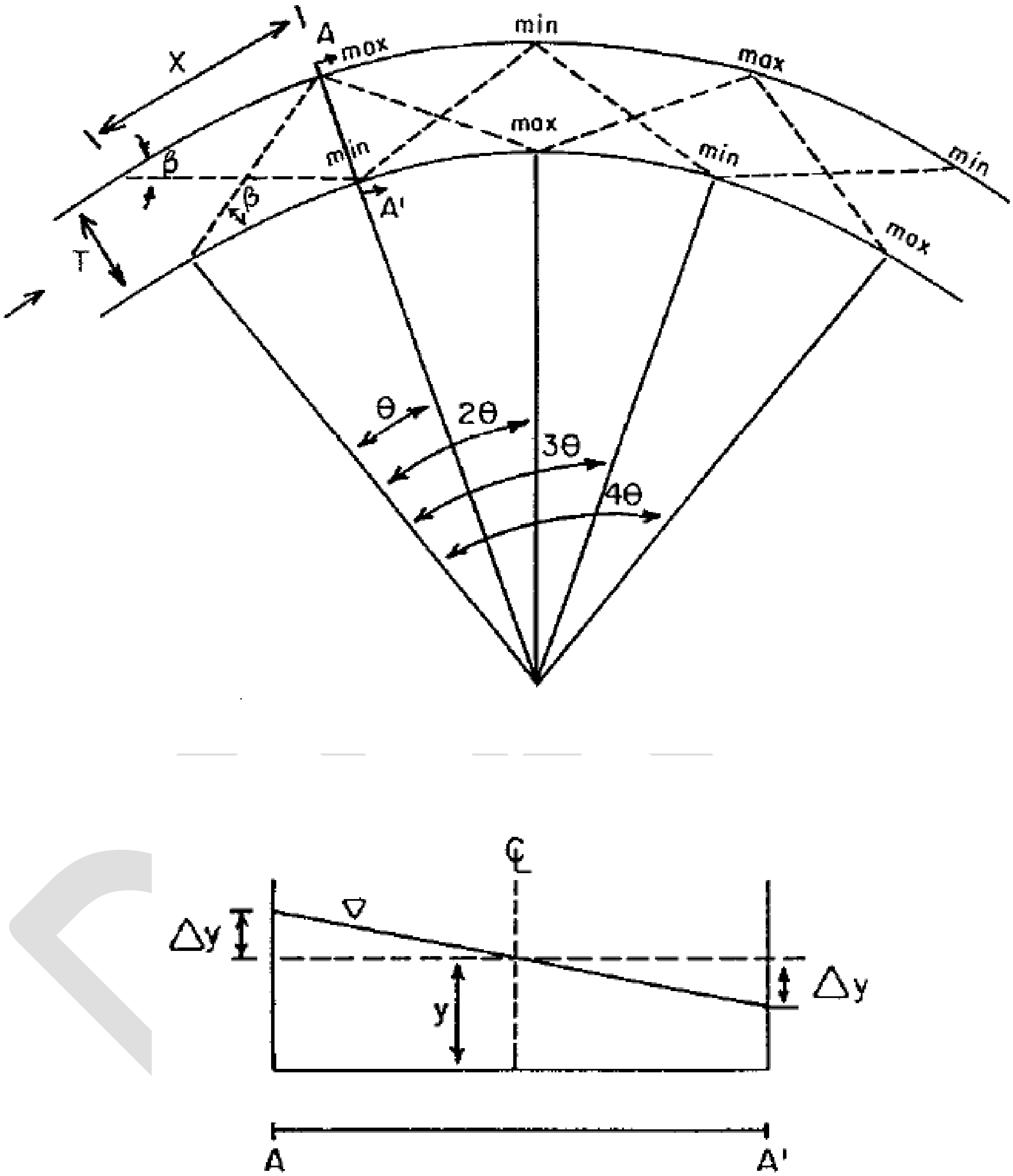

Figure 4-7. Cross-Wave Pattern for Supercritical Flow in a Channel with Curvature

4.5.22 Easement Curves

Easement curves can be used to reduce cross waves in bends experiencing supercritical flow (see Table 4.3). 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 from *Equation 4.13*.

$$L_e = \frac{0.32TV}{Y^{1/2}} \qquad \textit{(Equation 4.13)}$$

Where all terms are as previously described.

4.5.23 Banking

Banking is an alternative to providing additional freeboard in order to contain superelevated flows passing through 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, it is the policy of the Town that *banking should only be used in conjunction with the design of totally lined channels* (preferably smooth-lined).

4.5.24 Limiting Curvature

For flow with a design Froude number less than 0.86, the minimum radius of curvature along the centerline of the channel shall not be less than 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:

$$R_c = \frac{4V^2T}{gY_h} \qquad \textit{(Equation 4.14)}$$

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.
Y_h	=	Hydraulic depth of flow, in feet.

(See Corps publication EM 1110-2-1601 [USACE, 1994] for source of *Equation 4.14*.)

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

4.5.25 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

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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.

4.5.25.1 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 centerline of the proposed channel and the transitioning levee, or bank, is computed by use of *Equation 4.15*.

$$\theta = tan^{-1}\left[\frac{1}{3.375F_{ru}}\right] \qquad \textit{(Equation 4.15)}$$

Where,

θ	=	Transition angle, in degrees (see Figure 4-8).
F_{ru}	=	Upstream Froude number.

(See the PCDOT & PCRFCD [1984] for Source of *Equation 4.15*.)

The length of the transition is computed using *Equation 4.16*.

$$L = \frac{\Delta T}{2tan\theta} \qquad \textit{(Equation 4.16)}$$

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

(See Corps publication EM 1110-2-1601 [USACE, 1994] for source of *Equation 4.16*.)

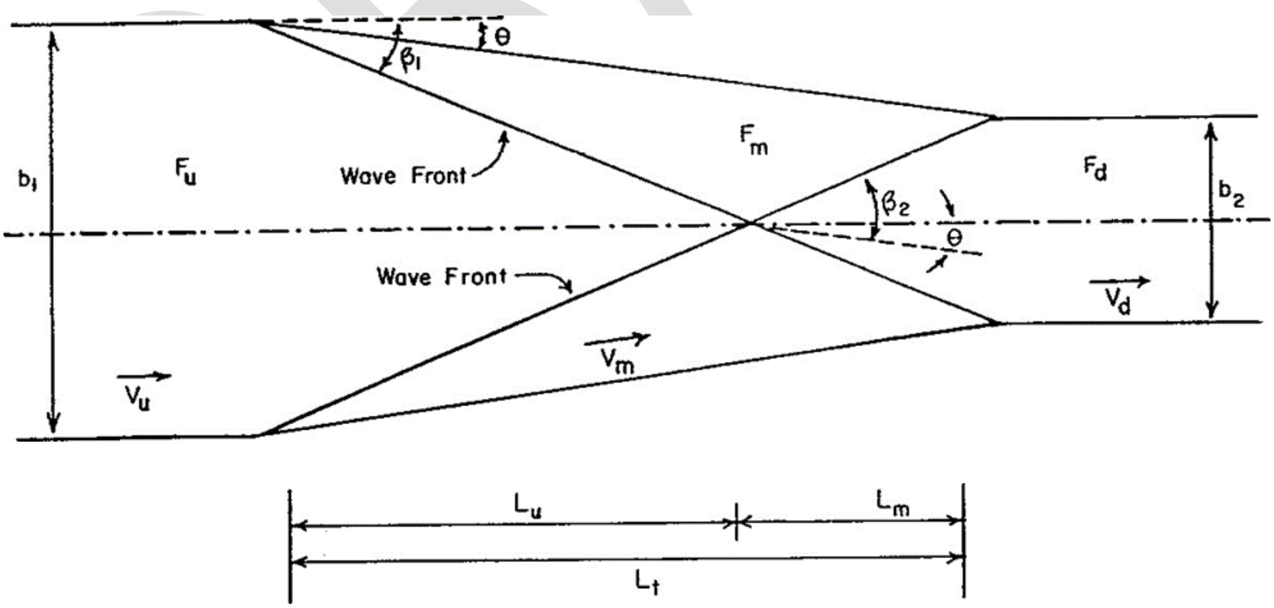

Figure 4-8. Transition for Channel Contractions in Supercritical Flow

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

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The maximum allowable transition angle is thirty degrees, unless supplemental engineering calculations or hydraulic modeling demonstrate to the satisfaction of the Town 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 *Equation 4.17a* or *Equation 4.17b*:

$$h_{tc} = C_c \Delta h_v \qquad \textit{(Equation 4.17a)}$$

$$h_{te} = C_e \Delta h_v \qquad \textit{(Equation 4.17b)}$$

Where,

h_{tc}, h_{te}	=	Transition losses in contracting and expanding reaches, respectively, in feet.
C_c	=	Coefficient of contraction, dimensionless.
C_e	=	Coefficient of expansion, dimensionless.
Δh_v	=	Difference in velocity head between the upstream and downstream ends of the transition, in feet.

(See Corps publication EM 1110-2-1601 [USACE, 1994] for source of *Equation 4.17a* and *Equation 4.17b.*)

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

Table 4-4. Head-Loss Coefficients for Use with Transitions in Open Channels

Type of Transition	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 4-8), must be chosen to minimize cross-wave action. In order to be able to accomplish this, *Equation 4.18a*, *Equation 4.18b*, *Equation 4.18c*, and *Equation 4.18d* must also be satisfied.

$$L_1 = \frac{b_1}{2tan\beta_1} \qquad \textit{(Equation 4.18a)}$$

$$L_2 = \frac{b_2}{2tan(\beta_2 - \theta)} \qquad \textit{(Equation 4.18b)}$$

$$L = L_1 + L_2 \qquad \textit{(Equation 4.18c)}$$

$$L = \frac{b_1 - b_2}{2tan\theta} \qquad \textit{(Equation 4.18d)}$$

Where all terms are as defined in Figure 4-8. (See Corps publication EM 1110-2-1601 [USACE, 1994] for source of *Equation 4.17a* and *Equation 4.17b*.)

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

1. First, using the upstream Froude number, F_{ru}, compute the wave-front angle, β_1, from *Equation 4.19*.

$$\beta_1 = sin^{-1}(\frac{1}{F_{ru}}) \qquad \textit{(Equation 4.19)}$$

   (See Figure 4-8 and *Equation 4.11* for definition).
2. Compute the distance L_1 from *Equation 4.18a*.
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_{rt}, from hydraulic conditions at the distance L_1.
6. From F_{rt}, compute a new wave-front angle, β_2, using *Equation 4.19*.
7. Compute L_2 according to *Equation 4.18b*.
8. Repeat Step 3 through Step 7 until *Equation 4.19c* and *Equation 4.19d* are both satisfied.

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

Table 4-5. Recommended Convergence Rates for Supercritical Flow Entrance Transitions

Mean Channel Velocity (feet per second)	**Wall Flare** (Horizontal to Vertical)	θ (Degrees)
10 – 15	1:10	5.71
15 – 30	1:15	3.81
30 – 40	1:20	2.86

4.5.25.2 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 equations:

For subcritical flow ($F_{ru} \leq 0.86$), $L_{TR} = 6.5(X_2 - 0.7X_1)$ *(Equation 4.20)*

For supercritical flow (F_{ru} > 0.86), $L_{TR} = 6.5F_{ru}(X_2 - 0.7X_1)$ *(Equation 4.21)*

Where terms for both equations are depicted in Figure 4-9 (note that in Figure 4.9 F_u = F_{ru} = upstream Froude number).

Equation 4.20 and *Equation 4.21* are modified from equations found on Plate B-24 of Corps publication EM 1110-2-1601 (USACE, 1994).

Exit transition sections are necessary in order 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 Town, can be made with all affected downstream property owners; or, (2) a drainage master plan has been developed for the watercourse which specifies a particular outlet configuration.

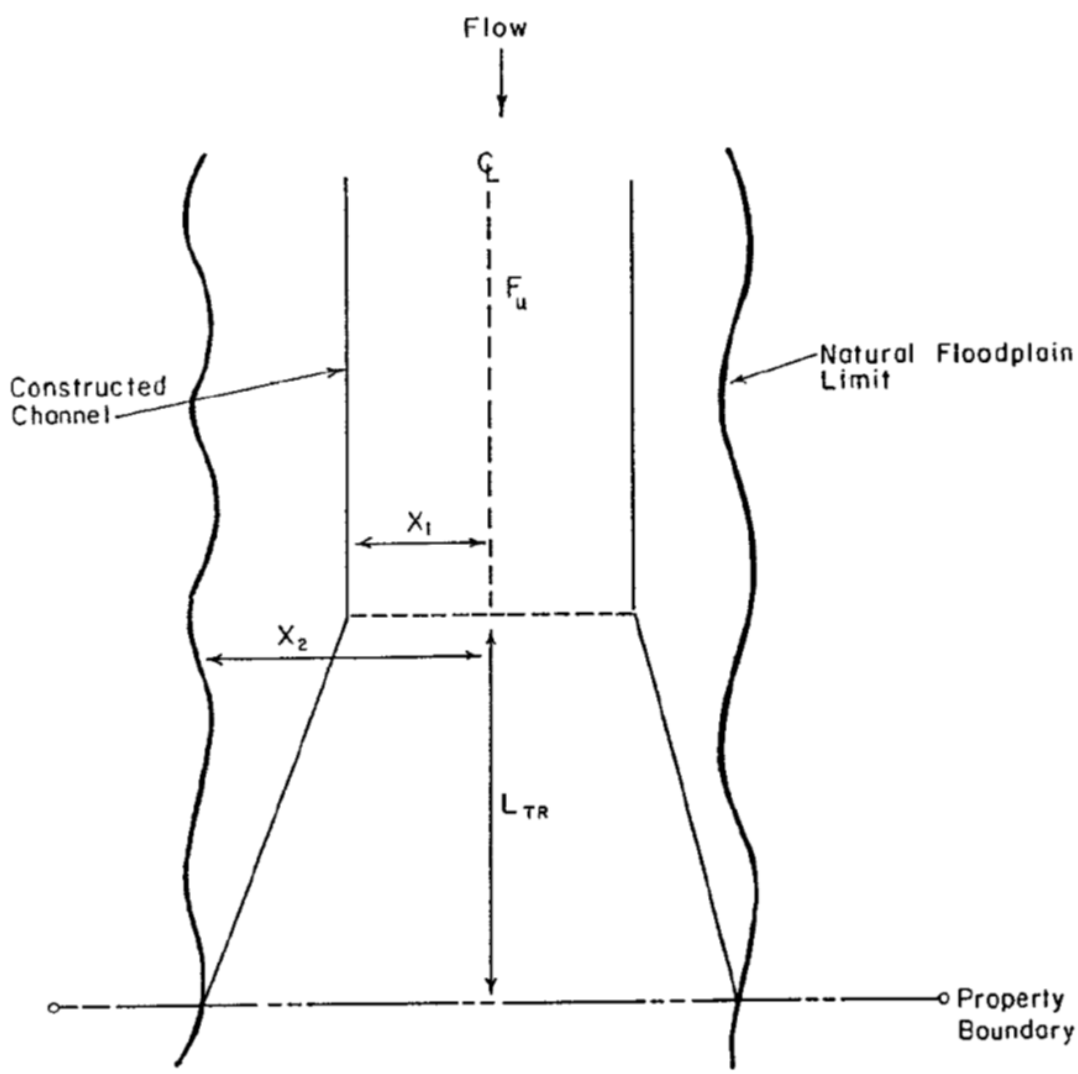

Figure 4-9. Transition Distance Required to Allow Flow to Return to Natural Conditions

4.5.25.3 Internal Channel Transitions

Internal channel transitions must be gradual in order to- minimize flow disturbances. The same formulas presented in the previous sections of this chapter for entrance and exit transitions also shall be used for contractions and expansions of flow within the channel. For transitions which constrict flow under subcritical conditions, use *Equation 4.15* to determine the convergence angle. The maximum transition angle shall not exceed thirty degrees. The length of the transition is computed by using *Equation 4.16*.

Contractions under supercritical flow conditions are computed by using *Equation 4.18a*, *Equation 4.18b*, *Equation 4.18c*, and *Equation 4.18d*. The required length for internal expansions under supercritical flow conditions is computed by using *Equation 4.21*. 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 & Wiggert (1972). Additional freeboard, and possibly additional reinforcement of the channel lining, will be required in order to account for the destructive effects associated with wave formation.

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 Subsection 4.5.14 of this chapter for information on hydraulic jumps). Additional reinforcement of the channel lining may also be required due to energy dissipation associated with the turbulent nature of the flow in the jump. 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 Section 8.10 of the DCM for more detailed information concerning energy dissipators and/or stilling basins.

4.5.26 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, the combining discharges, the 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.

4.5.26.1 General Design Guidelines

General design guidelines for junctions are as follows:

1. Tapered training walls should be constructed between adjoining flows to prevent crisscrossing flow streamlines from creating unwanted turbulence and wave action.

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2. The side-channel wave originating at the junction apex should impinge upon the main-channel wall downstream of the enlargement (see Subsection 4.5.26.3 of this Manual).

3. Junction angles, θ, should be no greater than 12 degrees for subcritical flow, and no greater than 6 degrees for supercritical flow. Angles greater than these are acceptable only if extra bank protection is provided to heights equal to or greater than the maximum wave heights given by Figure 4-10. 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 *Equation 4.22*.

$$L = \frac{3b_2V_2}{V_3 sin\theta} \qquad \textit{(Equation 4.22)}$$

Where,

b_2	=	Bottom width of the main channel downstream of the junction, in feet.
V2	=	Velocity of flow in the main channel downstream of the junction, in feet per second.
V_3	=	Velocity of flow in the tributary or side channel, in feet per second.
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 or the computed wave height, whichever is greater. 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. Ideally, 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.

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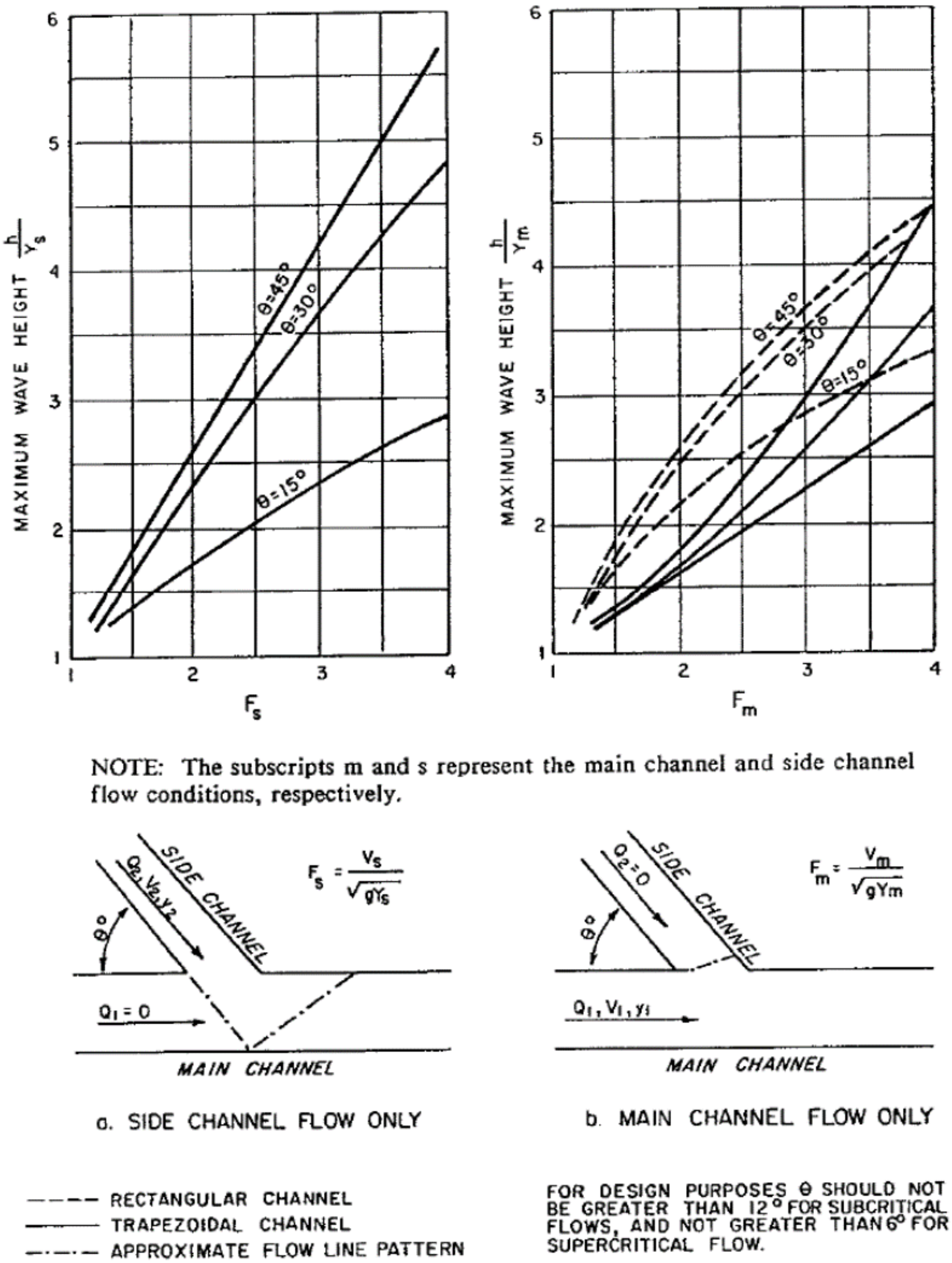

Figure 4-10.Maximum Wave Height at a Channel Junction

4.5.26.2 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 that might occur within the Town. Accordingly, these equations shall be used for designing drainage projects that are to be located within the Town and that incorporate channel junctions in their designs, unless the engineer can justify using other equations. The general form of the momentum equation is expressed as *Equation 4.23*.

$$P_{h2} + M_2 = P_{h1} + M_1 + M_3 cos\theta + P_{hi} + P_{hw} - P_{hf} \qquad \text{(Equation 4.23)}$$

Where,

P_{h1} = Hydrostatic pressure on Section 1, in pounds.

P_{h2} = Hydrostatic pressure on Section 2, in pounds.

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 the moving mass of water leaving the junction at Section 2.

$M_3 cos\theta$ = Axial component of the momentum of the moving mass of water entering the junction at Section 3.

Figures 4-11 and 4-12 show the relationship between the main channel and the tributary channel with respect to the preceding equation.

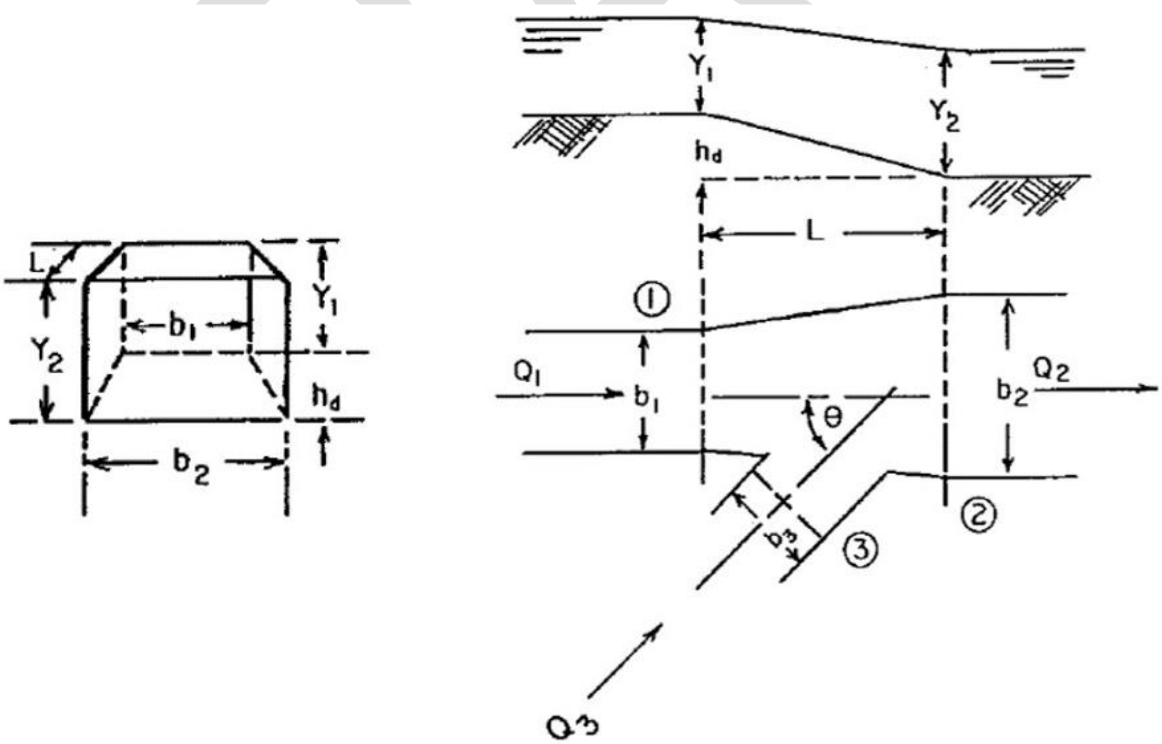

Figure 4-11. Rectangular Channel Junction

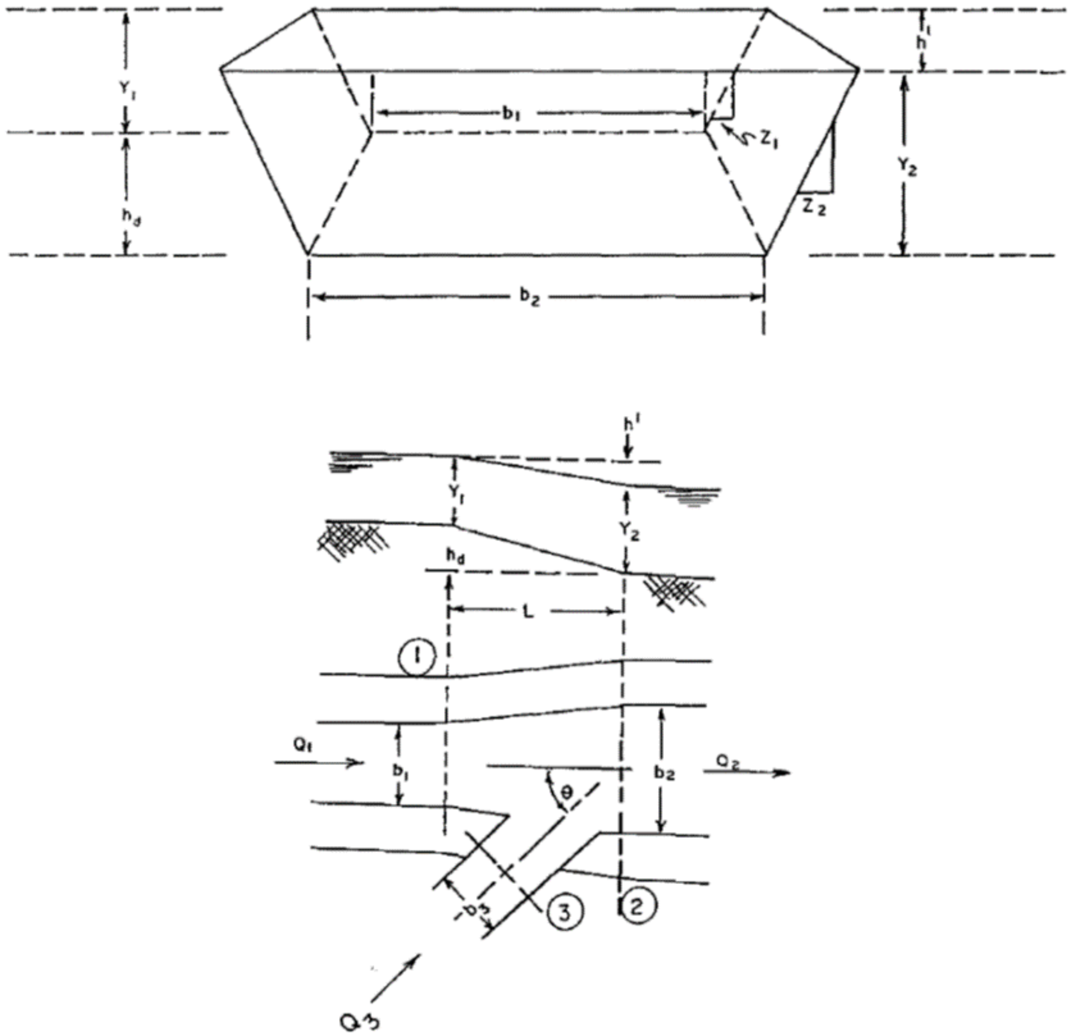

Figure 4-12. Trapezoidal Channel Junction

For a trapezoidal channel, the various equations which follow represent the variables comprising *Equation 4.23*.

$$M_1 = \frac{Q_1^2}{g(b_1+Z_1Y_1)Y_1} = \frac{Q_1^2}{gA_1} \qquad \textit{(Equation 4.24)}$$

$$M_2 = \frac{Q_2^2}{g(b_2+Z_2Y_2)Y_2} = \frac{Q_2^2}{gA_2} \qquad \textit{(Equation 4.25)}$$

$$M_3 = \frac{Q_3^2}{g(b_3+Z_3Y_3)Y_3} = \frac{Q_3^2}{gA_3} \qquad \textit{(Equation 4.26)}$$

$$P_{h1} = \frac{Y_1^2}{6}(3b_1 + 2Z_1Y_1) \qquad \textit{(Equation 4.27)}$$

$$P_{h2} = \frac{Y_2^2}{6}(3b_2 + 2Z_2Y_2) \qquad \textit{(Equation 4.28)}$$

$$P_{hi} = \frac{b_1+b_2}{2} h_d \left[Y_1 + \frac{(Y_2-Y_1)(b_1+2b_2)}{3(b_1+b_2)}\right] \qquad \textit{(Equation 4.29)}$$

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

(Equation 4.30)

$$P_{hf} = \frac{L(S_1+S_2)}{4}[(b_1+Z_1Y_1)Y_1 + (b_2+Z_2Y_2)Y_2] \qquad \textit{(Equation 4.31)}$$

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 (H:V) of channels 1, 2, and 3, respectively, in feet per foot.
S_1, S_2	=	Friction slopes of channels 1 and 2, respectively, in feet per foot.
L	=	Length of channel junction, in feet.
h_d	=	Vertical drop in the channel bottom through a junction, in feet.
h'	=	Vertical drop in the water surface through a junction, in feet.

4.5.26.3 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 centerline, as shown in Figure 4.13. 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 4.13a, is recommended. If Δb_1 is greater than b_3, an offset with respect to the right bank, as shown in Figure 4.13b, is recommended to ensure that the horizontal distance between the parallel alignment of the left banks of the main channels b_1 and b_3 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 (see Figure 4.13). The centerline radius of curvature of the curve is determined by use of *Equation 4.32*.

$$R_c = 4\frac{V_3^2 b_3}{gY_3} + 400 \qquad \textit{(Equation 4.32)}$$

   Where all terms are as previously defined. (See Corps publication EM 1110-2-1601 [USACE, 1994] for source of *Equation 4.32*.)

   This curve will connect the intersecting, straight channel walls, as shown in Figure 4.13, and will represent the revised edge of the bottom of the channel through the confluence area.

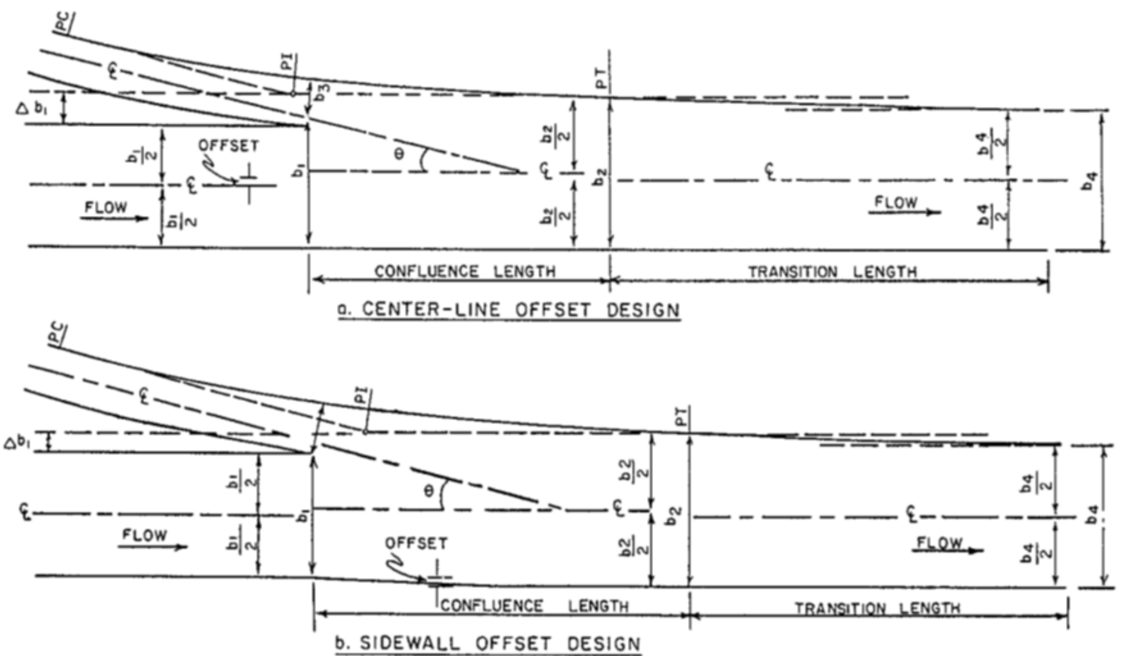

Figure 4-13. Typical Confluence Layouts

Note: For Design Purposes, θ, should be not greater than 12 degrees for subcritical flow conditions, and not greater than 6 degrees for supercritical flow conditions.

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 *Equation 4.33*.

$$L_o = (R_c + b_3)sin\left[cos^{-1}(1 - \frac{b_3 - \Delta b_1}{R_c + b_3}\right] \qquad \textit{(Equation 4.33)}$$

   The total length of the curve, L_{ct}, from PC to PT, can be computed using *Equation 4.34*:

$$L_{ct} = \frac{2\pi R_c \theta}{360} \qquad \textit{(Equation 4.34)}$$

The location of the point of curvature, PC, can be computed using *Equation 4.35*.

$$PC = PI - R\tan\frac{\theta}{2} \qquad \textit{(Equation 4.35)}$$

The location of the point of tangency, PT, can be computed using *Equation 4.36*.

$$PT = PC + L_{ct} \qquad \textit{(Equation 4.36)}$$

Figure 4-13 shows the relationship of these parameters.

5. Compute the confluence length, L_c, by using *Equation 4.37* and *Equation 4.38*, excerpted from the Los Angeles County Flood Control District (1973).

$$L_{c1} = \frac{b_3}{\sin\theta} \qquad \textit{(Equation 4.37)}$$

$$L_{c2} = 5(b_2 - b_1) \qquad \textit{(Equation 4.38)}$$

Compare these lengths to the computed distance L_o (*Equation 4.33*) from the apex to the PT. 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 4.23*), determine the depth of flow and hydraulic conditions at b_2. If either the depth of flow or the elevation of the water-surface 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 Subsection 4.5.25 of this chapter of the DCM.

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 within the region encompassing the Town, 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 confluence angle is equal to or less than 12 degrees (preferably 6 degrees for supercritical flow conditions), and the design procedure described above is followed, these types of waves should not pose a significant problem. However, should a greater confluence angle be dictated by site conditions, extra freeboard will be required in accordance with the procedure described in Subsection 4.5.26.3(3) of this chapter of the DCM, in order to contain waves created by flow impinging onto the opposite bank.

4.5.27 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 to be met than do most other constructed channels.

4.5.27.1 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, in order to prevent the formation of meandering, low-flow thalwegs. Channel slopes should be as steep as reasonably possible so as to help accelerate the water and prevent sediment buildup.

4.5.27.2 Depth

The discharge in a collector channel normally increases with downstream distance along its channel alignment. Collector-channel flows are subject to head losses associated with the impact and turbulence created by contributing flow entering the channel over its banks, in addition to the normal losses created by friction. Therefore, normal-depth procedures and step backwater calculations are not strictly applicable. The correct procedure for analyzing spatially-varied flow of the type that occurs in collector channels is discussed in many hydraulics textbooks under the heading “Side-Channel Spillways” (e.g., see Page 329 of Chow [1959]).

The minimum depth of a collector channel located within Town limits 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 minimum depth criterion 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 the collector channel; or, an equivalent 2-Dimensional hydraulic model may be used (e.g., HEC-RAS 2D, FLO-2D). The method may also be used if there is reason to believe that the guidelines presented above result in an overdesign of a collector channel.

4.5.27.3 Erosion Protection

Erosion protection for a collector channel requires special consideration because of inflow entering from over its sides. Hydrostatic pressure in the soil and seepage behind the bank protection can cause the underlying soils to “pipe,” and thus cause the veneer of bank protection to fail. Another problem is scour caused by side inflows.

In order 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) sides 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 the side inflow that is predicted to occur during the design flood (typically, the 1% AEP 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 the “key-in” are not required if the channel bank is constructed of 8-foot-thick to 10-foot-thick stair-stepped soil cement.

The bottom of the collector channel shall be lined, unless the toedown for the bank protection is buried deep enough to protect against the scour caused by side inflows. The procedures given in Chapter 5 of the DCM are to 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 toedown 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 toedown is needed on the opposite bank. For widths between five and ten times the depth of scour, the toedown on the opposite bank should be computed via a linear interpolation between the side-flow scour depth and the normal toedown depth. A typical collector channel is shown in Figure 4.14.

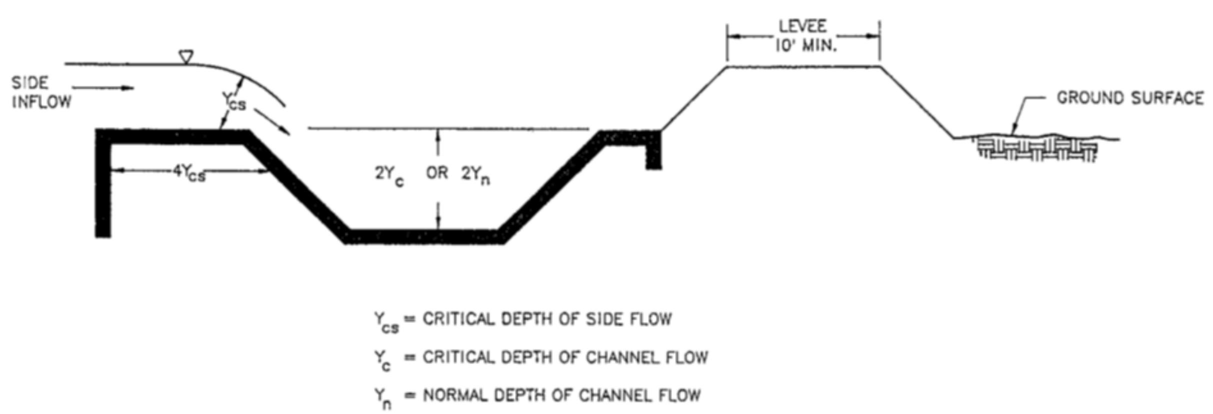

Figure 4-14. Typical Collector-Channel Cross-Section

4.5.27.4 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 5 of this Manual for those procedures that consider the effects of deposition and/or scour of alluvial sediments upon open-channel design.

4.5.27.5 Additional Design Considerations

Gabions are not acceptable for armoring the bottom of channels within the Town. Additionally, the use of “mechanically placed” riprap for armoring the bottom of channels within the Town is discouraged, and will only be permitted upon prior, written approval of the Town.

Material removed by excavation in order 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 (typically, the 1% AEP flood) for subcritical conditions, and should be 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 located contiguous to and along the downstream side of a collector channel should be at least one foot above the highest adjacent 1% AEP WSEL in the collector channel in order to safeguard against possible failure of the collector-channel embankment. This WSEL shall be determined either by the Method of Numerical Integration or by assuming an elevation equal to either (1) two times the normal depth of flow at the peak of the design flood for subcritical flow; or, (2) two times the critical depth of flow at the peak of the design flood for supercritical flow, whichever is greater.

4.6 Design Examples

<u>Example 4.1—Sequent Depth in a Trapezoidal Channel</u>

Given:

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 ft./ft.
Bottom Width (b):	10 ft
Side-Slopes (Z):	1H:1V
Roughness (n):	0.015
Depth (Y_1):	2.3 ft
Froude Number (F_u):	2.2
Hydraulic Depth (Y_h):	1.9 ft

Calculate:

The sequent depth of the hydraulic jump.

Solution:

Equation 4.8 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 4.8*:

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

The result of this substitution is: $304.9 = 240.0$.

Since momentum does not balance, a new sequent depth is chosen. By trial and error, the sequent depth is found to be <u>5.6 feet</u>:

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

The result of this substitution is: $304.9 = 240.0$, which is close enough. Therefore, assume sequent depth to be <u>5.6 feet</u>.

The Engineer should exercise care in using *Equation 4.8*, especially with a calculator or computer-program "root solvers." This is because there are two other solutions (roots) to *Equation 4.8* 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 4-3 can also be used to solve for sequent depth in this example. In order to do this, first compute $t = 10/[1(2.3)] = 4.3$. From Figure 4-3, using $F_u = 2.2$ and $t = 4.3$, $Y_2/Y_1 = 2.4$. Y_2 is then: 2.4(2.3 ft) = <u>5.5 feet</u>.

<u>Example 4.2—The Design of an Open-Channel Junction Under Supercritical Flow Conditions</u>

Given:

A main-channel flow, Q_1, of 2,000 cubic feet per second (cfs) is to be joined by a side-channel flow, Q_3, of 775 cfs. Because the flow is supercritical, the confluence angle, θ, is set to 6 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. Both channels are lined with concrete, and hydraulic conditions in the section located upstream of the channel junction are as follows:

<u>Main Channel</u>	<u>Side Channel</u>
Q_1 = 2,000 cfs	Q_3 = 775 cfs
n = 0.015	n = 0.015
b_1 = 20.0 ft	b_3 = 8.0 ft
Y_1 = 4.0 ft	Y_3 = 4.0 ft
Z = 1H:1V	Z = 1H:1V
S_1 = 0.01 ft/ft	S_3 = 0.008 ft/ft
F_1 = 2.0	F_3 = 1.7
A_1 = 95.8 ft^2	A_3 = 47.8 ft^2
V_1 = 20.9 fps	V_3 = 16.2 fps

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

Q_4 = 2,775 cfs
n = 0.015
b_4 = 28.0 ft
Y_4 = 4.0 ft
Z = 1H:1V
S_4 = 0.01 ft/ft
F_4 = 2.0
A_4 = 127.7 ft^2
V_4 = 21.7 fps

Calculate:

The flow hydraulics in the downstream channel.

Solution:

Calculation Steps:

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

Use centerline offset (Figure 4-13a).

$\Delta b_1 = 8$ feet.

$R_c = 4\frac{V_3^2 b_3}{g y_3} + 400 = 4\frac{(16.2)^2(8)}{g(4)} + 400$ = $\underline{465.2 \text{ feet}}$.

Assume that Station $PI =$ $\underline{100{+}00}$.

Station $PC =$ 100+00 $- R_c tan\left(\frac{\theta}{2}\right)$

Station $PC =$ 100+00 $- 465.2 tan\left(\frac{6}{2}\right) = \underline{99{+}75.62}$.

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

$L_{ct} = 465.2(6)\frac{2\pi}{360} = \underline{48.72 \text{ feet}}$.

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

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

The confluence length is, L_c, is:

$L_c = \frac{8}{sin(6^\circ)} = \underline{76.5 \text{ feet}}$; or,

$L_c = \frac{(28-20)10}{2} = \underline{40 \text{ feet}}$.

Using the largest of these values yields:

$L_c =$ 76.5 feet.

From *Equation 4.24* to *Equation 4.31*:

$M_1 = \frac{2{,}000^2}{[20+1(4)]g(4)} = \underline{1{,}294.0}.$

$M_2 = \frac{2{,}775^2}{[28+1(4)]g(4)} = \underline{1868.4}.$

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

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

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

$P_{hi} = \left[\frac{20+28}{2}\right](0.77)\left[4 + \frac{(4-4)(20+2(28))}{3(20+28)}\right] = \underline{73.9}.$

$P_{hw} = \left(\frac{4+4}{4}\right)\left[\frac{20+28}{2}(4-4) + 0.77[1(4) + 1(4)] + [28 + 1(4)]4 - 4[20 + 1(4)]\right] = \underline{76.3}.$

$P_{hf} = \left(\frac{76.5(0.01+0.01)}{4}\right)\left[[20 + 1(4)]4 + [28 + 1(4)]4\right] = \underline{85.7}.$

Using *Equation 4.23*:

$245.3 + 1868.4 = 181.3 + 1294.0 + 390.2[\cos(6°)] + 73.9 + 76.3 - 85.7$; which yields, 2113.7 = 1927.9.

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

By trial and error, eventually a depth, D_2, of 4.5 feet (±) is obtained. The hydraulic conditions at the end of the junction for $D_2 = 4.5$ feet are as follows:

Q = 2,775 cfs
b_3 = 28.0 ft
Y_3 = 4.5 ft
Z = 1H:1V
F_3 = 1.7
V_3 = 19.0 fps
A_3 = 146.3 ft^2

Additional bank-protection height will be needed in order 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.

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Chapter 5. Scour, Lateral Migration, and Sediment Transport

5.1 Discussion

Fluvial systems, such as alluvial rivers and normally dry washes, exist in a state of dynamic equilibrium, especially within areas such as the desert southwest USA and within the land region encompassing the Town, in particular. Over time, these fluvial systems naturally move either laterally, vertically, or both. While dynamic equilibrium of alluvial systems is often obvious to the trained Hydrologist, Geomorphologist, Professional Engineer, or perceptive observer it is a naturally occurring process, the impact of which is typically lost on most untrained individuals mainly due to the extended timeline over which the majority of these natural processes tend to occur. Oftentimes, an eroding river bank or a normally dry wash that migrates laterally is immediately thought to reflect some type of adverse condition that must be stopped. Sometimes this is the case; however, channel movement, erosion, and deposition are the continuous result of a fluvial system that is trying to adjust to natural or man-made changes and restore or attain dynamic equilibrium.

The primary cause of both horizontal and vertical geometric changes along alluvial rivers and normally dry washes is the disruption of system-wide sediment continuity within the fluvial system. In developing areas, this disruption is most often the result of anthropogenic changes caused by watershed urbanization, which encompasses channelization, floodplain encroachment, surface-water storage (e.g., dams and stormwater detention), floodwater diversions, sand-and-gravel mining, etc. Consequently, the most important issues related to assuring long-term stability of alluvial rivers and normally dry washes are the understanding of and the proper accounting for sediment-related issues in engineering design. Addressing these issues requires more than a quick (and sometimes short-term) fix for a localized problem. This is because the “big-picture” issues require a broad appreciation of the past, consideration of the present, and prediction of future behavior so that an understanding can be gained as to whether the “fix” might unintentionally lead to adjustments to the fluvial system which will create problems elsewhere.

An example of sediment issues is the long-term problem that the Town continues to face caused by deposition of sediment in “at-grade” (dip) roadway crossings of normally dry washes. It is anticipated that over time many of these crossings will be replaced with culverts. Under such circumstances, an important consideration must be that these culverts be designed to be self-cleaning, to the maximum extent practicable, or that at-grade crossings be replaced by bridge crossings.

5.2 Channel Trends

As previously stated in this chapter, it is important to understand the fluvial geomorphology of the wash and its contributing watershed. What the morphology of the wash was in its past, what the morphology of the wash is at the present time, and what the morphology of the wash will

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become are all important steps in the analysis process that are needed in order to identify channel trends and predict long-term channel stability. Accordingly, for all washes within the Town the designer shall assess whether the channel was stable in the past; whether it appears to be stable in the present, or does it appear to be degrading, aggrading, or migrating laterally; and, finally, will it be stable in the future. Visual observation, along with research of historical aerial and topographic records, can provide some evidence and a basis for comparing horizontal and vertical channel changes over time.

There are various analytical techniques available for estimating future channel equilibrium slopes or the magnitude of sediment transport. However, computer-modeling techniques for sediment transport are quite specialized and sophisticated and, as such, are considered to be outside the scope of this DCM. In general, these evaluations should be undertaken by Hydraulic Engineers or Fluvial Geomorphologists that have specialized training and experience in sediment transport. Thus, the techniques contained within the remainder of this chapter will present sediment issues by applying more general levels of assessment.

In general, the designer should follow these practices when performing sediment-transport analysis and design work within the Town:

1. Provide a narrative discussion of visual trends and other factors that may be impacting the stability of the wash.
2. Provide historical aerial photography and topography that presents evidence, or the lack thereof, of channel stability over time.
3. Provide a discussion of the impact that proposed urbanization will have upon system sediment continuity and long-term (future) channel stability.
4. For large washes, a sediment-transport computer model will not be required if it can be demonstrated to the satisfaction of the Town that such a model is not warranted. Otherwise, for any proposed channel modification or bank stabilizing activities on larger washes (> 1 square mile), a sediment-transport computer model such as HEC-6, HEC-RAS (1D or 2D), and FLO-2D, would be required.

   A sediment-transport analysis is typically required when:

   a. Channelization, including bank stabilization, will occur and be located along an otherwise natural watercourse.
   b. Construction or removal of features and/or vegetation that control the natural channel slope will occur.
   c. Excavation of material will occur in the channel bottom.
   d. Known areas of active degradation or aggradation are present.
   e. Bridge and culvert structures are proposed.

5. The Design Engineer shall evaluate the impacts of a proposed project with respect to either sediment deficit or sediment excess for any washes within the project area. A statement shall be provided regarding any impacts or lack thereof. Generally, however, over the long term the Design Engineer should expect that urbanization would lead to system sediment deficit.

5.3 Scour

Scour, as contrasted with general channel aggradation or degradation, relates to localized erosion caused by disturbance to the stream flow. In natural fluvial systems, scour may be induced by a localized change in channel features, such as fallen rocks or trees at channel transitions; while in constructed systems, scour can occur at bridge piers, along the toe of bank stabilization, or other similar locations. There are many documents that deal with scour, both of a local nature and of a national nature. The Town has decided to adopt local scour procedures published by COT (SLA, 1989/1998) and by Pima County (PCRWRD, 2017), for use in this DCM. Unless noted otherwise, the equations presented in this chapter are taken directly from SLA (1989/1998) and PCRWRD (2017).

5.3.1 Maximum Total Scour

The Maximum Total Scour, Z_{max}, is used for long-term design purposes. The scour components that comprise Maximum Total Scour include **A** — Total Single-Event Scour, which includes (1) General Scour, (2) Bedform Scour, (3) Bend Scour, (4) Local Scour, (5) Confluence Scour, and (6) Low-Flow Thalweg Scour, plus **B** — Long-Term Degradation (or Aggradation). These scour components are then summed, and the sum is multiplied by a numerical coefficient which is meant to account for potential increase in unit discharge due to flow irregularities along and within the channel.

For Maximum Total Scour:

When $Z_{MAX} \leq 5Y_{MAX}$, $$Z_{MAX} = Z_{TSE} + Z_{LTD} \qquad \textit{(Equation 5.1)}$$

When $Z_{MAX} > 5Y_{MAX}$, $$Z_{MAX} = 5.0Y_{MAX} \qquad \textit{(Equation 5.2)}$$

(Note that *Equation 5.2* sets the upper limit for Maximum Total Scour at 5.0 times the maximum flow depth, Y_{MAX}, in the channel.)

Where,

Z_{TSE}	=	Total Single-Event scour depth during a 1% AEP flood, in feet.
Z_{LTD}	=	Long-Term Degradation (or Aggradation) depth, in feet.
Y_{MAX}	=	Maximum depth of flow (measured from thalweg elevation), in feet.
Z_{MAX}	=	Upper limit of maximum predicted scour depth, in feet.

The general equation for computing the Single Event Scour Depth, Z_{TSE}, for a 1% AEP flood-peak discharge along either a curved or a straight reach of an alluvial watercourse is:

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$$Z_{TSE} = C_{nuf}(Z_G + Z_A + Z_B + Z_L + Z_C + Z_{LFT}) \qquad \textit{(Equation 5.3)}$$

Where,

Z_{TSE} = As previously defined.

C_{nuf}* = A coefficient to account for nonuniform flow (i.e., flow irregularities), no units (see Table 5.1).

Z_G = General Scour, in feet.

Z_A = Bedform (i.e., anti-dune) Scour, in feet.

Z_B = Bend Scour, in feet.

Z_L = Local Scour, in feet.

Z_C = Confluence Scour, in feet.

Z_{LFT} = Low-Flow Thalweg Scour, in feet.

*This coefficient is meant to account for increased unit discharge due to nonuniform flow and flow irregularities.

Table 5-1. Non-Uniform Scour Coefficients* (C_{nuf})

Channel Geometry	Value for Erodible Bed and Banks	Value for Erodible Bed and Protected Banks
Natural/Irregular	1.55	1.75
Parabolic	1.50	1.70
Trapezoidal	1.33	1.55
Rectangular	1.20	1.33

*As derived from Blodgett (1986)

5.3.2 General Scour

General Scour, Z_G, is the lowering of the streambed across the stream or waterway. This lowering may or may not be uniform across the streambed. That is, the depth of scour may be deeper in some parts of the cross-section. General scour may result from flow contraction and other general scour conditions caused by changes in sediment transport. The equation for computing General Scour is:

$$Z_G = 0.293(q_{P1\%})^{2/3}[(q_{P1\%})^{1/15} - 1.073] \qquad \textit{(Equation 5.4)}$$

(If $Z_G < 0.10$ feet, assume $Z_G = 0.10$ feet.)

Where,

Z_G = General Scour, in feet.

$q_{P1\%}$ = Average peak discharge per unit width of channel during a 1% AEP flood, in cfs/foot.

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5.3.3 Bedform Scour

Bedform Scour, Z_A, represents vertical changes in the streambed elevation due to shear stresses on the streambed causing bed deformation that produce sand waves which move in a longitudinal direction. Dunes and antidunes are typical bedforms. *In the Town, it should be assumed that only antidune bedforms will occur*. The equation for computing bedform scour is:

$$Z_A = 0.0137(V_{1\%})^2 \qquad \textit{(Equation 5.5)}$$

Where,

Z_A	=	Bedform Scour, in feet.
$V_{1\%}$	=	Average channel flow velocity during a 1% AEP flood, in ft./sec.

When computing Z_A, it should be kept in mind that the depth of bedform scour can never exceed one-half the maximum depth of flow (i.e., $Z_A \leq 1/2\, Y_{MAX}$). Thus, if the calculated Z_A exceeds $1/2\, Y_{MAX}$, it should be assumed that $Z_A = 1/2\, Y_{MAX}$.

5.3.4 Bend Scour

Bend Scour, Z_B, is caused by the increase in shear stress that occurs due to flow acceleration and the resulting spiral flow pattern created along the concave (outside) portion of a bend. The equations for computing bend scour are:

For nearly direct impingement, such as right-angle bends, where $\alpha > 60$ degrees [$(R_c/T) \leq 0.5$].

$$Z_B = 0.243(q_{P1\%})^{0.733} \qquad \textit{(Equation 5.6)}$$

For bends where 17.75 degrees $< \alpha < 60$ degrees [$0.5 < (R_c/T) < 10$].

$$Z_B = 0.293 q_{P1\%}^{0.733}\left(2.1\left[\frac{sin^2\left(\frac{\alpha}{2}\right)}{cos\alpha}\right]^{0.2} - 1\right) \qquad \textit{(Equation 5.7)}$$

For bends where $\alpha \leq 17.75$ degrees (i.e., $R_c/T \geq 10$).

$$Z_B = 0.00 \qquad \textit{(Equation 5.8)}$$

The relationship between α and R_c/T is mathematically described below (also, see Figure 5-1).

$$\frac{R_c}{T} = \frac{cos\alpha}{4sin^2\left(\frac{\alpha}{2}\right)} \qquad \textit{(Equation 5.9)}$$

Bend Scour should not be included as a component of the Total Single-Event Scour beyond X_B, which is the distance beyond the end point of the curve. The equation for X_B is:

$$X_B = \left(\frac{0.6}{n}\right) Y_{MAX}^{1.17} \qquad \textit{(Equation 5.10)}$$

Where,

Z_B	=	Bend Scour, in feet.
$q_{P1\%}$	=	Average peak discharge per unit width of channel during a 1% AEP flood, cfs/foot.
R_c	=	Radius of curvature along channel centerline of bend, in feet.
T	=	Top width of the main channel immediately upstream of the bend, in feet.
α	=	The angle formed by projection of the centerline of a channel from its beginning point of curvature to a point that meets the line tangent to the outer bank of channel within the bend (see Figure 5-1).
X_B	=	Distance from the end point of a bend to the point where channel scour is no longer influenced by flow curvature, in feet.
n	=	Channel Manning n-value, dimensionless.
Y_{MAX}	=	Maximum depth of flow in the channel during a 1% AEP flood measured from the thalweg elevation located immediately upstream of the bend, in feet.

Note that Bend Scour is an independent scour component that should be added to the other scour components in order to determine Total Single-Event Scour in areas of channel curvature.

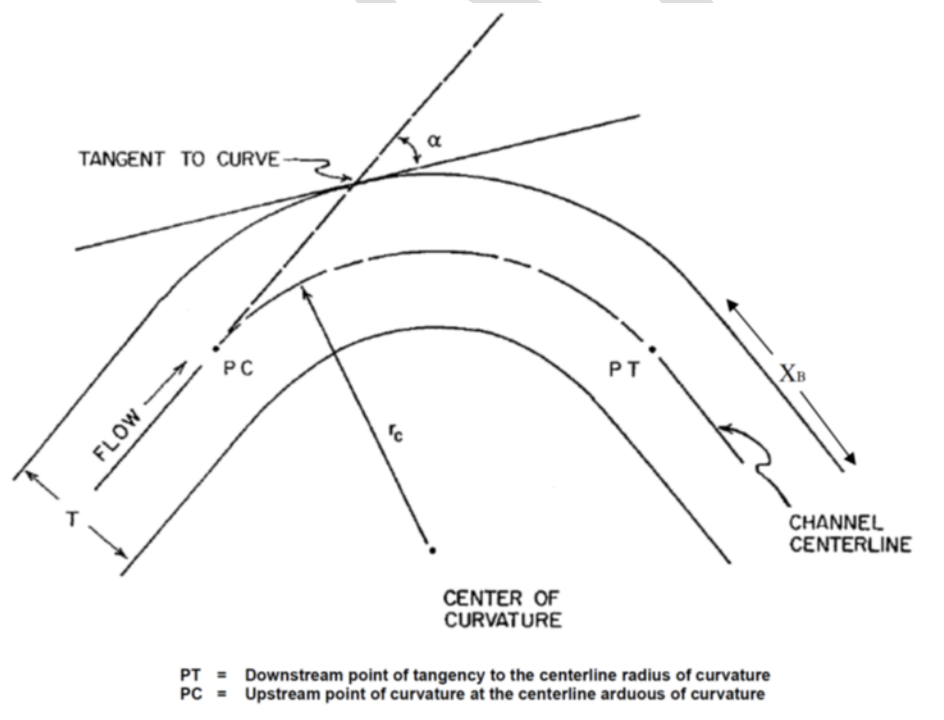

Figure 5-1. Determination of Angle, α, in Bend-Scour Formula

5.3.5 Local Scour

Local Scour, Z_L, is the removal of material from around piers, abutments, spurs, and embankments, caused by an acceleration of flow and resulting vortices induced by obstructions to the flow. Bridge Contraction Scour is a special case of Local Scour. Local Scour can be determined from a summation of the components that are normally associate with Local Scour. These components are:

$$Z_L = Z_{CUL} + Z_{LB} + Z_{LE} + Z_{LD} + Z_{SS} + Z_{MH} \qquad \textit{(Equation 5.11)}$$

Where,

Z_L	=	Local Scour, in feet.
Z_{CUL}	=	Local Scour due to the presence of a culvert, in feet.
Z_{LB}	=	Local Scour due to the presence of bridge piers/abutments, in feet.
Z_{LE}	=	Local Scour due to the presence of encroachments, in feet.
Z_{LD}	=	Local Scour due to the presence of a drop structure (e.g., grade control), in feet.
Z_{SS}	=	Local Scour due to presence of sanitary sewer in scour zone, in feet.
Z_{MH}	=	Local Scour due to presence of a manhole in the scour zone, in feet.

Note that Z_{SS} and Z_{MH} should only be considered when there may be reason to be concerned about the presence of a sanitary sewer system within the flow regime, which is not a common situation in the Town. For guidance on the determination of Z_{SS} and Z_{MH}, the user should refer to PCRWRD (2017).

5.3.5.1 Local Scour Due to Culverts

For Local Scour at outlets of circular culverts flowing full and when D_{50} < 8.0 millimeters (0.315 inches):

$$Z_{SCUL} = 0.5312\left(\frac{Q_{P1\%}^{0.5}}{D_o^{0.25}}\right) \qquad \textit{(Equation 5.12)}$$

For Local Scour at outlets of non-circular or partially full culverts and when D_{50} < 8.0 millimeters (0.315 inches):

$$Z_{SCUL} = 0.3897\left(\frac{Q_{P1\%}^{0.5}}{A^{0.125}}\right) \qquad \textit{(Equation 5.13)}$$

In *Equation 5.12* and *Equation 5.13*, D_{50} = diameter of the culvert, in feet; $Q_{P1\%}$ = 1% AEP flood peak discharge, in cfs; and A = the cross-sectional area of flow in the culvert, in square feet.

Likewise, the length, L_{SCUL}, and the width, W_{SCUL}, of the scour hole created immediately downstream of the outlet of the culvert can be computed from the following equations.

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For a circular culvert flowing full:

$$L_{SCUL} = 3.3667\left(\frac{Q_{P1\%}^{0.62}}{D_o^{0.55}}\right) \quad \textit{(Equation 5.14)}$$

$$W_{SCUL} = 1.2487\left(\frac{Q_{P1\%}^{0.89}}{D_o^{1.225}}\right) \quad \textit{(Equation 5.15)}$$

Note: *Equation 5.14* and *Equation 5.15* apply when D_{50} < 8 mm.

For a non-circular or partially-full culvert:

$$L_{SCUL} = 2.2884\left(\frac{Q_{P1\%}^{0.62}}{A^{0.275}}\right) \quad \textit{(Equation 5.16)}$$

$$W_{SCUL} = 0.6820\left(\frac{Q_{P1\%}^{0.89}}{A^{0.6125}}\right) \quad \textit{(Equation 5.17)}$$

Note: *Equation 5.16* and *Equation 5.17* apply when D_{50} < 8 mm.

Where, both length and width are in feet, and D_{50} equals the median (50%) size of the sediments, in millimeters.

Figure 5-2 of this document (see below) is intended to be applicable to local scour at a drop structure, which is discussed later on as a Local Scour component. However, for purposes of local scour at a culvert outlet, it can also be assumed that the longitudinal profile of the scour hole at the culvert outlet will be identical to the template longitudinal profile of the scour hole depicted in Figure 5-2. Parameters Z_{SCUL} and L_{SCUL} are therefore substituted for parameters Z_{lss} and L_s, shown in Figure 5-2, in order to determine the point, X_{sce}, where maximum scour terminates, which is typically located downstream one-half the length of the calculated culvert outlet scour, L_s. In addition, for design purposes it should be assumed that maximum scour, Z_{SCUL}, occurs everywhere along the streambed between the brink of the culvert outlet and the point X_{sce}.

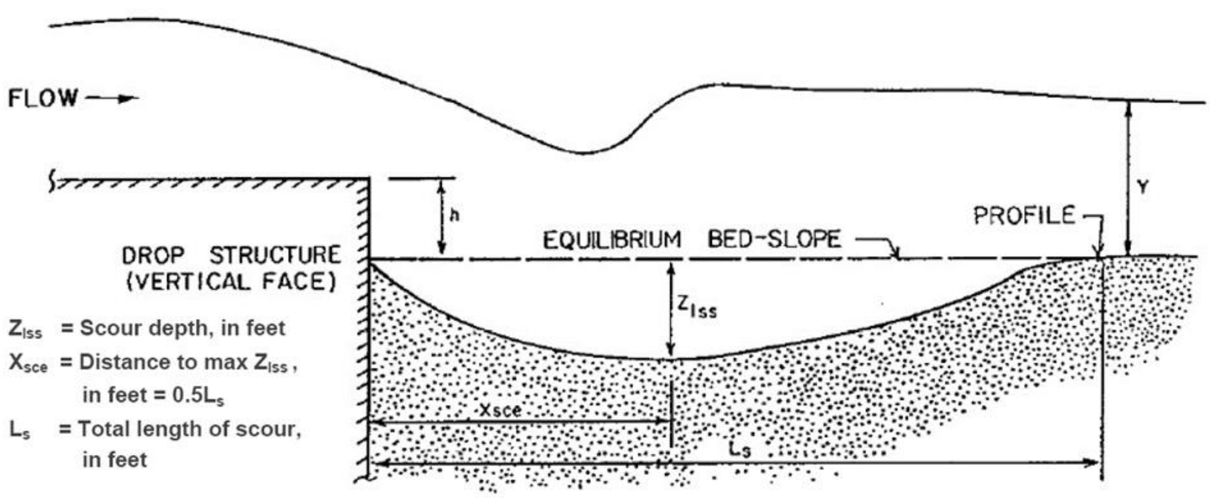

Figure 5-2. Profile Characteristics of Scour Hole Downstream of a Drop Structure

5.3.5.2 Local Scour due to Bridge Piers

Local Scour due to bridge piers is dependent upon the shape of the bridge pier. Due to the likelihood of debris on piers during flood events the following equation, originally derived for square-nosed shaped piers, should be used for local scour due to a bridge pier:

$$Z_{LB} = \left[2.2Y_{1\%}\left(\frac{b_{pe}}{Y_{1\%}}\right)^{0.65} F_{ru}^{0.43}\right] \qquad \textit{(Equation 5.18)}$$

In *Equation 5.18*, the parameters are defined as follows:

For $b_{pd} > 5$ $$b_{pe} = L_{pw}sin(\varphi_p) + b_{pd}cos(\varphi_p) \qquad \textit{(Equation 5.19)}$$

For $b_{pd} \leq 5$ $$b_{pe} = L_{pw}sin(\varphi_p) + 5cos(\varphi_p) \qquad \textit{(Equation 5.20)}$$

Where,

Z_{LB}	=	Local Scour contribution due to bridge piers with a pier shape reduction factor of 1.0 included, in feet.
$Y_{1\%}$	=	Depth of flow during a 1% AEP flood, in feet.
F_{ru}	=	Upstream approach Froude number.
b_{pe}	=	Effective bridge pier width, from *Equation 5.19* or *Equation 5.20*), in feet.
b_{pd}	=	Bridge pier width with debris (= physical width + 2 feet of debris width on each side), in feet.
φ_p	=	Angle of approach flow to pier wall (φ_p = 0° for cylindrical piers), in degrees.
L_{pw}	=	Length of pier wall, in feet.

5.3.5.3 Local Scour due to Encroachments

Local Scour due to encroachments projecting into the flow of a channel (see Figure 5-3), such as, but not limited to, bridge abutments and fill projections, such as overbank levees, can be computed from the following equations. Note that the equation to be utilized is dependent upon the quantity $L_e/Y_{1\%}$. For large values of $L_e/Y_{1\%}$, *Equation 5.22* should be used.

If $\frac{Z_{LE}}{Y_{1\%}F_{ru}^{0.33}} < 4.0$ $$Z_{LE} = 2.15sin(\theta_a)Y_{1\%}\left(\frac{L_e}{Y_{1\%}}\right)^{0.4} F_{ru}^{0.33} \qquad \textit{(Equation 5.21)}$$

If $\frac{Z_{LE}}{Y_{1\%}F_{ru}^{0.33}} \geq 4.0$ $$Z_{LE} = 4YF_{ru}^{0.33} \qquad \textit{(Equation 5.22)}$$

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Where,

Z_{LE}	=	Local scour contribution from encroachments, in feet.
θ_a	=	Slope angle of encroachment face (measured from horizontal), in degrees.
L_e	=	Encroachment length (use caution determining embankment length), in feet.
F_{ru}	=	Upstream Froude number.
$Y_{1\%}$	=	Upstream depth of flow during a 1% AEP flood, in feet.

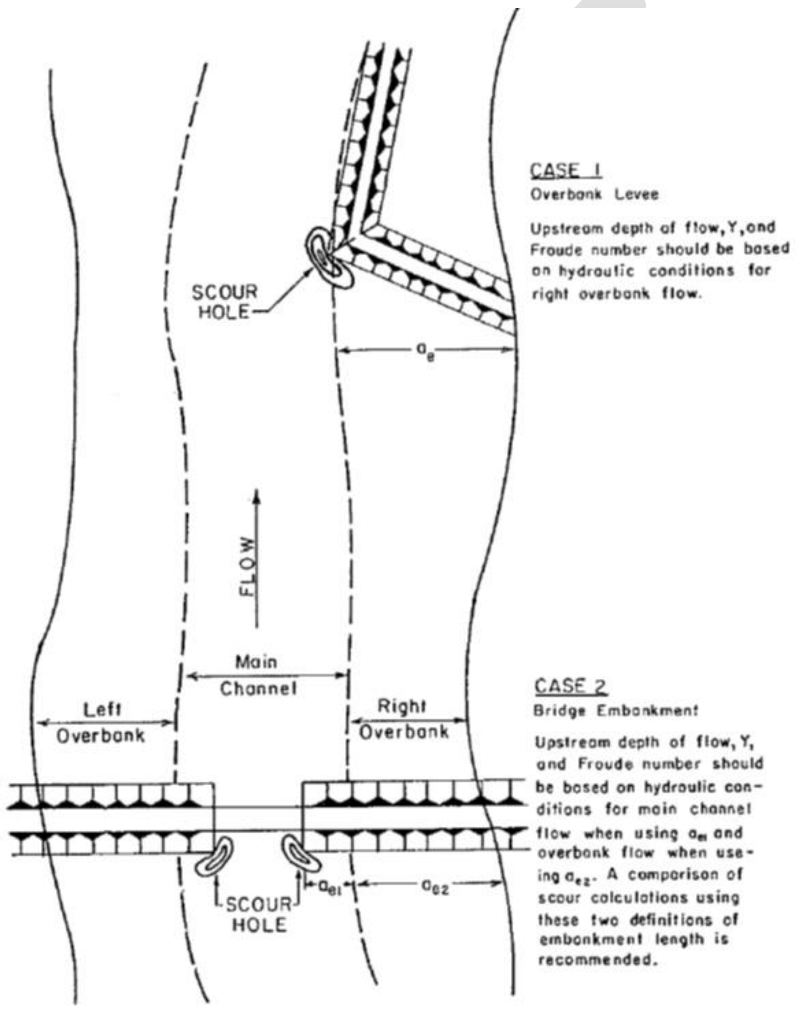

Figure 5-3. Local Scour due to Projection of Encroachment into Flow Profile

5.3.5.4 Local Scour at Drop Structures

Local Scour immediately below channel drops can occur under two conditions. The first condition, say at a high-head grade-control structure, is a special case where the drop is a free, unsubmerged overfall. The second condition is where the drop is submerged, which will likely be the circumstance for most low-head drops comprised of grade-control structures placed across alluvial watercourses in the Town. *Equation 5.23* should be used for the first condition (free overall), while the *Equation 5.24* should be used for the second condition (submerged overfall). Both equations are shown below.

If $\frac{h}{Y_1} \geq 1.0$ $$Z_{LD} = 1.32 q_{P1\%}^{0.54} H_T^{0.225} - TW$$ *(Equation 5.23)*

If $\frac{h}{Y_1} < 1.0$ $$Z_{LD} = 0.581 q_{P1\%}^{0.667} \left(\frac{h}{Y_{1\%}}\right)^{0.411} \left[1 - \left(\frac{h}{Y_{1\%}}\right)\right]^{-0.118}$$ *(Equation 5.24)*

Where,

Z_{LD}	=	Local Scour contribution from a flow drop (i.e., a drop structure, such as a grade control) measured from thalweg downstream of control-point, in feet.
H_T	=	Total drop in head (measured as the difference between upstream and downstream energy grade lines), in feet (normally, use the difference in WSELs, $Y_{1\%}$ - TW).
$Y_{1\%}$	=	Upstream depth of flow during a 1% AEP flood, in feet.
F_{ru}	=	Upstream Froude number, dimensionless.
h	=	Exposed height on downstream side of drop structure, in feet.
TW	=	Tailwater depth (downstream depth of flow) during a 1% AEP flood, in feet.

Note that the exposed height, h, should be based upon the long-term (future) channel degradation predicted to occur on the downstream side of the drop structure.

Figure 5-2, presented previously herein, depicts the longitudinal profile of local scour that occurs immediately below a drop. For design purposes, it should be assumed that $L_s = 12Z_{lss}$, and that $X_{sce} = 6Z_{lss}$. In addition, for design purposes it should also be assumed that maximum scour, Z_{lss}, occurs everywhere along the streambed between the brink of the drop and the point X_{sce}.

Note: If the flow parameter $h/Y_{1\%}$ falls in the range of $0.85 < h/Y_{1\%} < 1.0$ at the brink of a drop structure, then the predicted scour below a channel drop should be computed using both *Equation 5.23* and *Equation 5.24*. Then, the larger of the two values thus computed should be used for design purposes. Figure 5-4 depicts the longitudinal shape of a scour hole located immediately downstream of an overfall.

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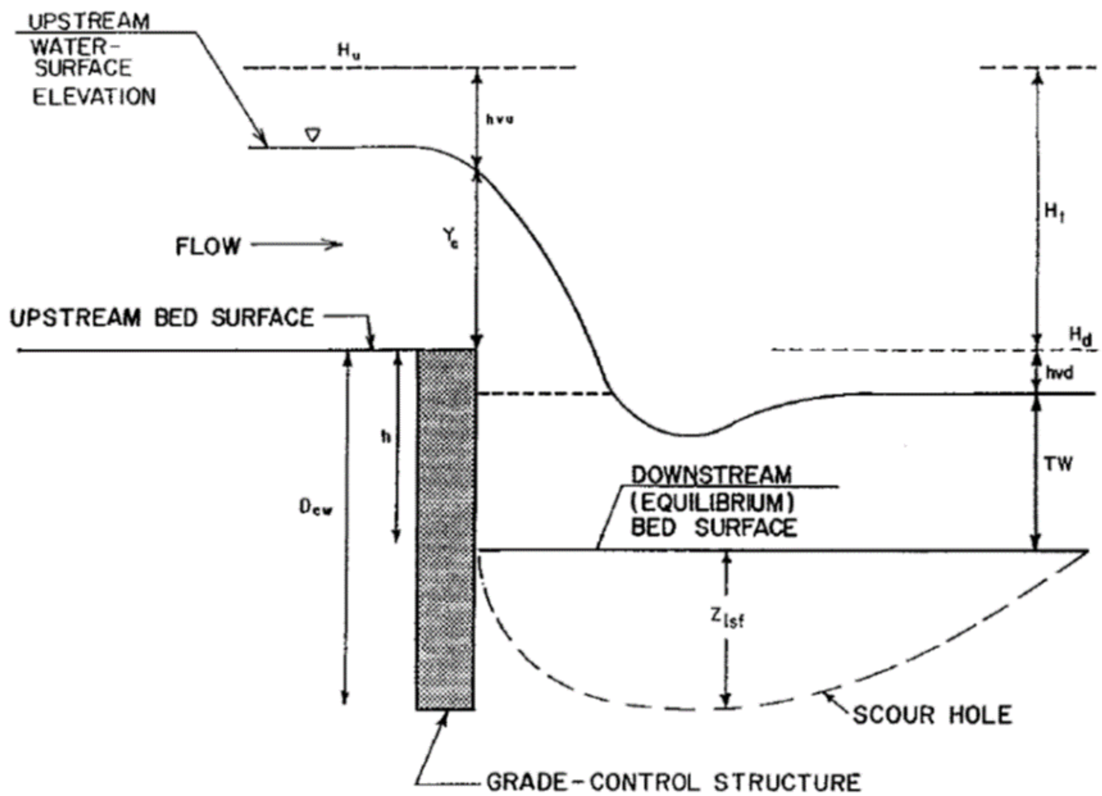

Figure 5-4. Depiction of Scour Hole Downstream of a Grade-Control Structure

5.3.5.5 Local Scour at Sanitary Sewers and Associated Manholes

This DCM does not address impacts of either local scour at Sanitary Storm Sewers, Z_{SS}, or local scour at associated Manholes, Z_{MH}, exposed to the scour zone. However, information regarding these two local-scour parameters is available from the PCRWRD (2017).

5.3.6 Confluence Scour

Confluence Scour, Z_C, is caused by the combining of two or more watercourses, at oblique angles, which leads to flow turbulence and resulting scour at the point of confluence. The equation for computing confluence scour depth is:

$$Z_C = Y_{MC} - Y_{1\%} \qquad \textit{(Equation 5.25)}$$

$$\frac{Y_{MC}}{Y_{MS}} = 2.24 + (0.031)\alpha_c \qquad \textit{(Equation 5.26)}$$

Note that *Equation 5.26* is for non-cohesive sands/gravels (30º < α_c < 90º).

Where,

Z_C = Scour due to a confluence of two or more watercourses, in feet.

Y_{MC}	=	Maximum flow depth in the confluence scour hole during a 1% AEP flood, in feet.
Y_{MS}	=	Average flow depth, from the water surface to the mean scour depth, during a 1% AEP flood, in feet.
$Y_{1\%}$	=	Average flow depth during a 1% AEP flood, in feet.
α_c	=	Angle of confluence of two watercourses, in degrees. (Note: Do not confuse the α_c parameter in *Equation 5.26* with the α parameter depicted in Figure 5-1. They are NOT the same.)

Confluence Scour geometry is characterized by steep slopes dipping downward from the upstream channels into a scour pool, which then feathers out along a gently inclined bed slope which leads to a bar with a pronounced foreset slip face. See Figure 5-5.

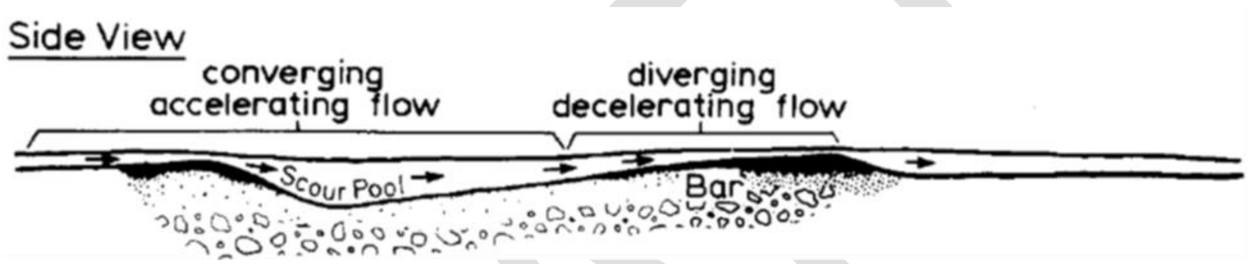

Figure 5-5-5. Profile of Confluence Scour

The approximate location of the Confluence Scour pool cab be defined as being immediately at and downstream from the confluence, as illustrated in Figure 5-6.

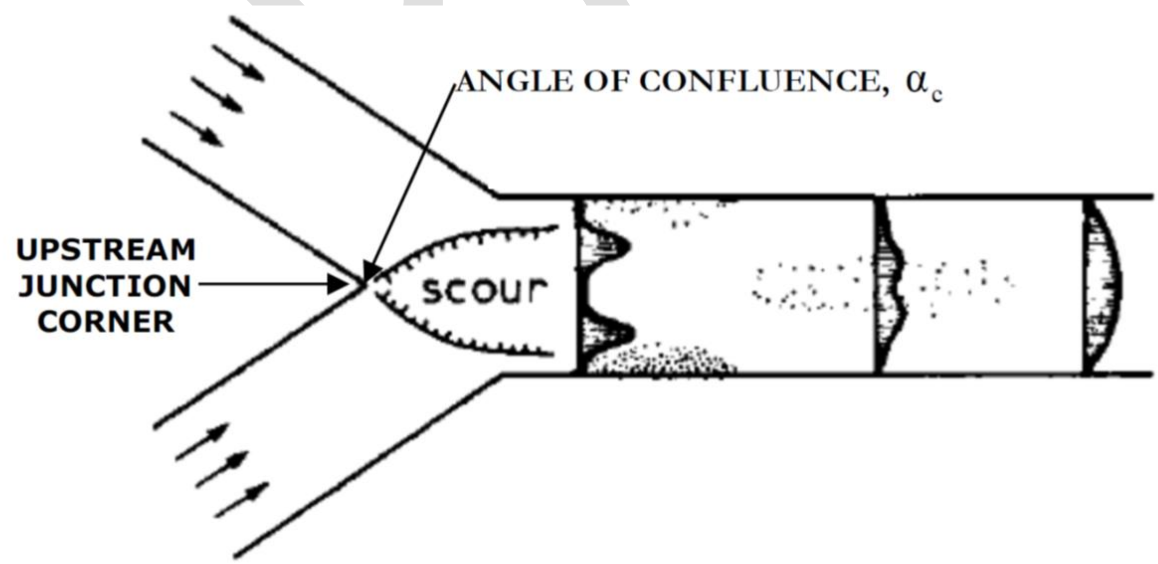

Figure 5-6. Plan View of Confluence Scour

For design purposes, it should be assumed that the Confluence Scour hole extends downstream from the upstream junction corner a distance equal to 2.5 times the bottom width, in feet, of the downstream channel. For design purposes, assume that the Confluence Scour component computed using *Equation 5.25* applies everywhere within a region located 2.5 times the bottom width of the downstream channel, as measured from the upstream junction corner of the confluence.

5.3.7 Low-Flow-Thalweg Scour

Low-Flow Thalweg Scour, Z_{LFT}, is a small lowering, generally from 1 to 2 feet, that occurs within larger channel cross-sections which are over-widened to the extent that they cannot convey dominant discharges (typically, 10% AEP flow events within the Town) at their larger width/depth ratios. The equations for determining Low-Flow Thalweg depth are as follows:

Depending upon the relationship between flow velocity, $V_{1\%}$, flow depth, $Y_{1\%}$, and flow width, $W_{1\%}$, during a 1% AEP flood, the scour depth to account for development of a low-flow thalweg follows the relationships adopted from the COTDSM (SLA, 1989/1998), which are shown below:

$Z_{LFT} = 0.0$ feet When $W_{1\%}/Y_{1\%} \leq 1.15V_{1\%}$ (Equation 5.27)

$Z_{LFT} = 1.0$ feet When $W_{1\%}/Y_{1\%} > 1.15V_{1\%}$ and $A < 30$ square miles (Equation 5.28)

$Z_{LFT} = 2.0$ feet When $W_{1\%}/Y_{1\%} > 1.15V_{1\%}$ and $A \geq 30$ square miles (Equation 5.29)

Where,

A	=	Drainage area of watershed, in square miles.
Z_{LFT}	=	Low-flow thalweg scour depth, in feet.
$W_{1\%}$	=	Flow width of the main channel during a 1% AEP flood, in feet.
$Y_{1\%}$	=	Average flow depth within the main channel during a 1% AEP flood, in feet.
$V_{1\%}$	=	Average velocity within the main channel during a 1% AEP flood, in ft./sec.

(Note that if a low-flow thalweg is present within the project site, the observed thalweg depth should be used in lieu of results generated by any of the preceding equations.)

5.3.8 Local-Scour Geometries

Maximum local scour will occur at the locations defined for the applicable scour components presented in this Chapter of the DCM. In general, the local-scour geometries for the various components can be described as follows (SLA, 1989/1998):

5.3.8.1 Culvert Scour

The longitudinal profile of a scour hole downstream from the outlet of a culvert typically will be as depicted in Figure 5-2 which, although depicting the scour profile below a drop, is also assumed to be the scour profile downstream from a culvert outlet. Using this profile and the equations contained in Subsection 5.3.5(A), the contribution due to culvert scour at a downstream point of interest can be determined, dependent upon the downstream distance from the brink of the culvert outlet.

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5.3.8.2 Bridge Piers and Abutments

The shape of scour holes created by bridge piers or by scour at abutments should be assumed to be more or less consistent with that of an inverted, truncated "cone," with the base of the cone extending away from the pier or the abutment a distance equal to the depth of the computed scour component (i.e., pier scour or abutment scour). Upward from the base, it should be assumed that the sides of the scour hole everywhere slope at an angle of 3H:1V. Therefore, the zone of influence of pier or abutment scour should be assumed to extend a distance of $4Z_{LB}$ from the outside diameter of the pier or from the face of the abutment. Maximum scour occurs 0.0 feet to Z_{LB} feet from the face of the pier or abutment, and then tapers off at a rate of 3H:1V until dissipating at a distance of $4Z_{LB}$ feet from the face of the pier or abutment.

5.3.8.3 Encroachments

Bridge abutments are one form of an encroachment structure for which the local-scour geometry has been described in the preceding paragraph. Another type of encroachment, though, is a directional dike or levee. In general, the local-scour geometry at the tip of a directional dike or levee is to be treated in the same manner as is the local-scour geometry at bridge abutments. In such cases, though, *Equation 5.22* should be applied to determine Z_{LE} if the length of the levee intercepting flow is such that $Z_{LE}/Y_{1\%}F^{0.33} \geq 4.0$. See Figure 5-3. Along the riverside face of the levee, assume that the local toe scour is $2.2Y_{MAX}$, and that this scour depth extends Y_{MAX} feet from the face of the levee, tapering upward at 3H:1V and dissipating at a distance of $8.8Y_{MAX}$ feet from the face of the levee.

5.3.8.4 Grade-Control Structures

Local-scour geometry immediately downstream from a grade-control structure is generally as depicted in Figure 5-2 and Figure 5-4, from which the contribution due to grade-control-induced scour at the point of interest can be determined, dependent upon the distance downstream from the brink of the grade-control structure. Example 5.1 provides an example of how to determine the spacing of grade-control structures.

<u>Example 5.1—Spacing of Grade-Control Structures</u>

Given:

A channel in a highly urbanized watershed is to be built to contain the 1% AEP flood discharge. The sides of the channel are to be of comprised of shotcrete, and the bottom is to be comprised of natural alluvium. Channel characteristics are as follows:

Bottom Width:	20 feet.
Design Slope:	0.015
Side-Slopes:	1H:1V
Manning n:	0.035

Hydraulic characteristics are as follows:

$Q_{P1\%}$ = 700 cfs	$Q_{P10\%}$ = 350 cfs
$Y_{1\%}$ = 3.1 ft.	$Y_{10\%}$ = 2.1 ft.
$V_{1\%}$ = 9.7 ft./sec.	$V_{10\%}$ = 7.7 ft./sec.
$q_{P1\%}$ = 35 cfs/ft.	$q_{P10\%}$ = 17.5 cfs/ft.

Calculate:

The required spacing between grade-control structures.

Solution:

Because the watershed is highly urbanized, *Equation 5.34*, which follows, is used to compute the predicted (future) equilibrium slope of the channel. The result of applying *Equation 5.34* yields:

$S_{eq} = ([0.875]^{-1.1}[1-0.4]^{0.7})(0.015) = (0.81)(0.015) = 0.0121$ feet/foot.

Now, it is desired that a 2-foot drop height eventually be allowed to develop on the immediate downstream side of each grade-control structure located along the longitudinal extent of the channel. Accordingly, the spacing between the structures can be determined by using *Equation 5.30*, which follows:

$$L_r = \frac{H}{S_{ib} - S_{eq}} \qquad \textit{(Equation 5.30)}$$

Where,

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

Thus, $L_r = \frac{2}{0.015-0.0121} = 690$ feet (i.e., the required spacing between grade-control structures).

The depth of scour immediately downstream of each 2-foot drop will be dependent upon whether the grade-control structure will be submerged or will be subjected to free overall (see previous discussion in this chapter regarding the use of *Equation 5.23* and *Equation 5.24*).

5.3.9 Long-Term Degradation

Long-Term Degradation (LTD), Z_{LTD}, is a general and progressive lowering of the channel bed caused by system-wide sediment discontinuity (typically sediment deficit), and the streambed erosion that occurs over a relatively long channel reach length and over a long period of time. Estimating LTD along an alluvial watercourse can be an extremely difficult task to accomplish with reasonable accuracy. This DCM uses a procedure which is a refinement to the LTD methodology presented in Section 6.9 of the COTDSM (SLA, 1989/1998), which was originally published more than 30 years ago. Since that time, new techniques have been developed for determining LTD—

particularly with regard to the passage of time required in order to reach the amount of LTD predicted. Under certain circumstances, application of a time factor can significantly reduce the amount of predicted LTD to use in the design process. Generally, the procedure in this document assesses the long-term changes, based upon a specified dominant discharge over a specified project design life, that are predicted for streambed slope as the alluvial watercourse approaches either an armoring or an equilibrium-slope (i.e., dynamic-equilibrium) condition.

The following three equations are recommended for computing estimates of LTD along an alluvial watercourse. The first two equations, which are based upon the computed estimate of the needed time to achieve a stable slope (TSS), should be used when a downstream control (such as a roadway or grade-control structure) exists, or if the armoring particle size, D_a, $> D_{90}$, the particle size for which only 10 percent of the sediment sizes in the reach are larger, by weight.

For $T_{SS} < P_o$

$$Z_{LTD} = \frac{8}{13}\left(S_n - S_{eq}\right)L_{dc} \qquad \textit{(Equation 5.31)}$$

If $T_{SS} > P_o$, use the following equation:

For $T_{SS} > P_o$

$$Z_{LTD} = C_{LTD}\left[\frac{W_{10\%}^{0.3}}{Q_{10\%}^{0.376}} A P_o\left(S_n - S_{eq}\right)\right]^{1/2} \qquad \textit{(Equation 5.32)}$$

For primarily *natural* conditions within the upstream contributing watershed (i.e., less than 10% imperviousness cover), use a LTD coefficient of C_{LTD} = 4.55.

For primarily *rural to suburban conditions* within the upstream contributing watershed (i.e., from 10% to 30% imperviousness cover), use a LTD coefficient of C_{LTD} = 7.73.

For essentially *moderately urban* to *highly urban* conditions within the upstream contributing watershed (i.e., where imperviousness cover typically is more than 30%), use a LTD coefficient of C_{LTD} = 13.99.

However, if D_a is less than the D_{90} particle size of the reach (D_a < D_{90}), then the following equation should be used to determine the limit of LTD, given the presence of armoring:

$$Z_{LTD} = \frac{(0.6562 D_a)}{P_c} \qquad \textit{(Equation 5.33)}$$

Where, in the preceding three (3) equations:

Z_{LTD}	=	Long-Term Degradation, in feet.
A	=	Drainage area of watershed, in square miles.
D_a	=	Armoring particle size, in millimeters.
P_c	=	*Percent* (%) of the material which is coarser than the armoring size.
$Q_{P10\%}$	=	10% AEP peak discharge, in cfs.
S_n	=	Natural channel slope, in feet per foot.
S_{eq}	=	Equilibrium channel slope, in feet per foot.

L_{dc} = Estimated distance to downstream control; in feet.

P_o = Estimated time period over which streambed degradation will occur (i.e., a design life of typically 100 years), in years.

T_{SS} = Estimated time to achieve a stable slope (typically less than 100 years), in years.

$W_{10\%}$ = Width of channel conveying the 10% AEP peak discharge (i.e., the so-called "dominant discharge"), in feet.

In order to select the proper equation to use for the calculation of the LTD component to use in the Total Maximum Scour calculation, the user should follow the procedure described below to calculate an appropriate amount for the LTD component.

First, determine the equilibrium slope, S_{eq}, using the following equation:

$$S_{eq} = \left(\left[\frac{Q_{Pu,10\%}}{Q_{Pn,10\%}}\right]^{-1.1}[1 - R_s]^{0.7}\right)S_n \qquad \textit{(Equation 5.34)}$$

Where,

S_{eq} = Equilibrium channel slope, in feet per foot.

S_n = Natural channel slope, in feet per foot.

$Q_{Pu,10\%}$ = 10% AEP flood peak discharge for *urbanized* conditions, in cfs.

$Q_{Pn,10\%}$ = 10% AEP flood peak discharge for *natural* conditions, in cfs.

R_s = Sediment reduction factor for upstream sediment supply (i.e., typically the ratio of predicted long-term impervious area to total area. Some examples of the causes of R_s are (1) watershed urbanization, (2) sand-and-gravel mining, (3) stormwater detention and/or retention facilities. The value of R_s varies from 0.0 to 1.0 (i.e., a decimal fraction).

Typically, 0.15 would represent a reasonable average value for R_s for rural to suburban watershed conditions; 0.4 would represent a reasonable average value for R_s for moderately-urban watershed conditions, and 0.7 would represent a reasonable average value for R_s under highly-urban watershed conditions.

Also, typical values for the parameter $\left[\frac{Q_{Pu,10\%}}{Q_{Pn,10\%}}\right]^{-1.1}$ are 0.875 for suburban to moderately-urban watershed conditions and 0.833 for highly-urban watershed conditions.

Next, determine the fluvial system's controlling factor (i.e., either streambed armoring or stable slope), as follows:

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5.3.9.1 Streambed Armoring

First, determine if LTD, Z_{LTD}, is controlled by streambed armoring. In order to do this, the size of armoring material, D_a in mm, must be calculated. The armor size can be determined using the following equation:

$$D_a = 0.2659 \left[\frac{V_{10\%}^{3.5}}{q_{10\%}^{0.5}}\right] \qquad \textit{(Equation 5.35)}$$

D_a is assumed to be representative of the larger particle sizes observed in the streambed. It should also be assumed that the D_a particle size is consistent with the D_{90} to D_{95} particle size ranges in the streambed, with D_{90} being designated as the "default" streambed armoring size. In the absence of any sediment data, within the Town it should be assumed that D_{50} is 1 mm in size, and that D_{90} is 4 mm in size. However, whenever possible a sediment analysis should be conducted in the event that there is evidence of possible D_a particle sizes which would better represent the potential armoring sediments encountered within the study reach. If D_a is consistent with the D_{90} to D_{95} particle sizes in the study reach, then *Equation 5.33* shall be used to determine the limit of LTD due to armoring:

$$Z_{LTD} = \frac{(0.6562 D_a)}{P_c} \qquad \textit{(Equation 5.33)}$$

Where,

Z_{LTD} = Limit of LTD due to armoring, in feet.

D_a = Size of the armoring material, in mm.

P_c = *Percent* (%) of the material which is coarser than the armoring size.

If a downstream control exists, or if D_a > D_{90} to D_{95} within the study reach, then streambed armoring would not control. *This will typically be the case for watercourses located within the Town*. Under such circumstances, the procedure in Subsection 5.3.9.2 should be applied:

5.3.9.2 Stable Slope

Determine long-term degradation, Z_{LTD}, controlled by stable slope using the following approach:

If the time to achieve the stable slope, T_{SS}, is less than the design life, P_o, of the project, that is if $T_{SS} < P_o$, use Equation 5.31:

For $T_{SS} < P_o$

$$Z_{LTD} = \frac{8}{13}\left(S_n - S_{eq}\right)L_{dc} \qquad \textit{(Equation 5.31)}$$

On the other hand, if $T_{SS} > P_o$, use Equation 5.32.

For $T_{SS} > P_o$

$$Z_{LTD} = C_{LTD}\left[\frac{W_{10\%}^{0.3}}{Q_{10\%}^{0.376}} A P_o \left(S_n - S_{eq}\right)\right]^{1/2} \qquad \textit{(Equation 5.32)}$$

For primarily *natural* conditions within the upstream contributing watershed (i.e., less than 10% imperviousness cover), use a LTD coefficient of C_{LTD} = 4.55.

For primarily *rural* to *suburban* conditions within the upstream contributing watershed (i.e., from 10% to 30% imperviousness cover), use a LTD coefficient of C_{LTD} = 7.73.

For essentially *moderately urban* to *highly urban* conditions within the upstream contributing watershed (i.e., where imperviousness cover typically is more than 30%), use a long-term degradation coefficient of C_{LTD} = 13.99.

The general equation to use for computing the time required to achieve a stable slope (T_{SS}) is:

$$T_{SS} = \frac{C_{TSS}(S_n - S_{eq}) L_{dc}^2 Q_{10\%}^{0.376}}{W_{10\%}^{0.3} A} \qquad \textit{(Equation 5.36)}$$

Note:

For primarily *natural* conditions within the upstream contributing watershed (i.e., less than 10% imperviousness cover), use a long-term stable-slope time coefficient of C_{TSS} = 0.0183.

For primarily *rural* to *suburban* conditions within the upstream contributing watershed (i.e., from 10% to 30% imperviousness cover), use a long-term stable-slope time coefficient of C_{TSS} = 0.0063.

For essentially *moderately urban* to *highly urban* conditions within the upstream contributing watershed (i.e., where imperviousness cover is more than 30%), use a long-term stable-slope time coefficient of C_{TSS} = 0.0019.

The definitions for the hydraulic parameters contained in the preceding equations for long-term degradation, Z_{LTD}, are the same as previously defined for *Equations 5.31* through *Equation 5.33*.

Note that long-term degradation can be limited by downstream channel controls, as well as by streambed armoring (if applicable), and can be influenced by several other factors as well—all of which can be extremely difficult to predict. Nevertheless, if applied properly, the preceding relationships should provide a reasonable means of assessing long-term-degradation trends that exist along alluvial watercourses which traverse less urbanized to highly urbanized watersheds located within the Town.

5.3.10 Scour Summary

The designer should always account for scour under the following circumstances:

1. At, upstream of, and downstream of in-channel drop structures or grade-control structures.
2. At, upstream of, and downstream of proposed bridges.
3. At, upstream of, and downstream of critical areas of channel expansions or contractions.
4. Whenever a hydraulic obstruction is placed in a wash (e.g., manholes, pipelines, etc.).
5. Downstream of culverts, drainage channels, or other outlets, if stabilization is not provided.

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6. Along the side-slopes and toes of channel banks, whether natural or constructed, whenever the channel bottom remains in an unprotected state.

5.4 Lateral Migration

Lateral movement of a watercourse can be unexpected and devastating. For example, in the October 1983 flood, the Rillito River and the Santa Cruz River each moved hundreds of feet laterally in places, impacting property and leading to the loss of several structures, including buildings. The designer shall identify the limits of lateral migration in the design.

The Town will only accept the evaluation of lateral-migration potential from a qualified Arizona Registered Professional Engineer. Guidelines for this type of analysis can be found in several documents. The COTDSM (SLA, 1989/1998) is one reference, as well as the ADWR's *State Standard 5-96, Watercourse System Sediment Balance* (ADWR, 1996). Section 5.5 provides guidelines for determining "default" Erosion-Hazard Setback zones.

5.5 Erosion-Hazard Setbacks

Erosion-hazard setbacks shall be incorporated into the project design based upon the results of the lateral-migration analysis. The designer shall consider bank erosion and use the following setback equations, adopted from Chapter VII, Section 7.6, of the COTDSM (SLA, 1989/1998).

For watercourses which have contributing watershed areas less than 30 square miles in size or times of concentration less than 3 hours during a 1% AEP flood, use:

For $\frac{R_c}{T} \geq 10$ $\qquad SB \geq 1.0(Q_{P1\%})^{0.5}$ *(Equation 5.37)*

For $5 < \frac{R_c}{T} < 10$ $\qquad SB \geq 1.7(Q_{P1\%})^{0.5}$ *(Equation 5.38)*

For $\frac{R_c}{T} \leq 5$ $\qquad SB \geq 2.5(Q_{P1\%})^{0.5}$ *(Equation 5.39)*

Where,

SB	=	Erosion-hazard setback limit, in feet.
$Q_{P1\%}$	=	The 1% AEP flood, in cfs.
R_c	=	Radius of curvature of channel centerline, in feet.
T	=	Top width of the main channel, in feet.

For watersheds with contributing watershed areas larger than 30 square miles in size, setbacks shall be prescribed by the Town.

Other methods may be applied to determine setbacks, but only after prior written acceptance by the Town. When applying allowable-velocity/tractive-stress/tractive-power approaches, appropriate geotechnical data, including sieve analyses to determine D_{50}, D_{65}, D_{75}, etc., shall supplement erosion-hazard setback determinations. A particular procedure can be found in

PCRFCD Technical Policy, TECH-020, titled *Engineering Analysis Requirements for Determining an Alternative Safe Erosion Hazard Setback Limit* (2011).

The following erosion-hazard criteria shall apply:

1. Within the calculated zone of lateral migration; the construction of all structures, including non-habitable buildings, roads, and infrastructure are prohibited unless adequately protected and stabilized to resist erosion. (This criterion does not prohibit stream crossings for roadways and utilities, but lateral migration is to be considered by the designer.)
2. Within any area identified as a lateral-migration zone, the construction of habitable buildings is prohibited unless it can be demonstrated that some natural feature such as rock, caliche, or a constructed feature limits lateral migration, thus producing a lesser setback.
3. Unless an alternate erosion-hazard setback limit is presented to and subsequently accepted by the Town, under no circumstances shall the erosion-hazard setback be less than 50 feet from an unprotected bank of any watercourse subject to *Equation 5.37* that has a 1% AEP flood peak greater than 2,500 cfs; or for any watercourse subject to *Equation 5.38* that has a 1% AEP flood peak greater than 865 cfs; or for any watercourse subject to *Equation 5.39* that has a 1% AEP flood peak greater than 400 cfs. For any other watercourse, the erosion-hazard setback shall not be less than 25 feet from an unprotected bank.

5.6 Bed-Material Sediment-Transport

In the past, bed-material sediment-transport within and along alluvial watercourses has been estimated by numerous investigators over many, many decades. Unfortunately, most methods of determining bed-material sediment transport have been developed for watercourses with mild to moderate longitudinal slopes with flows containing primarily very fine to fine sand sizes. In the Town, most watercourse slopes are in the range of 1% to 2%, with some in the range of 2% to 3%. For steep-slope channels, bed-material sediment-transport cannot be evaluated using “traditional” methodologies. In order to account for the expected high bed-material sediment transport that occurs within and along the steep-sloped alluvial watercourses located within the Town, *Equation 5.40* (Zeller, 2019) shall be used to compute the peak bed-material sediment-transport capacity that occurs during individual flood events whenever site-specific bed-material sediment size data is available.

$$Q_{sp} = 0.0886(S_c^{0.885} Q_{pe}^{1.44}) \left(\frac{G^{0.45}}{D_{50}^{0.61}}\right) W_e^{-0.44} \qquad \textit{(Equation 5.40)}$$

When applying *Equation 5.40*, the Engineer should use site-specific bed-material sediment data obtained from a sufficient number of sieve analyses in order to properly characterize the bed-material sediment composition within the study reach of the watercourse. However, lacking the availability of site-specific bed-material sediment size data, it can be assumed that within the Town the default value to use for D_{50} = 1 mm and the default value to use for G = 3.0 (The default values for these two sediment parameters are based upon an analysis of sediment samples

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obtained from the streambeds of various watercourses located within the Town). Substituting D_{50} = 1 mm and G = 3.0 into *Equation 5.40* yields *Equation 5.41* (Zeller, 2019):

$$Q_{sp} = 0.1453(S_c^{0.885} Q_{pe}^{1.44}) W_e^{-0.44} \qquad \textit{(Equation 5.41)}$$

Where,

Q_{sp}	=	Bed-material sediment-transport rate (unbulked), in cfs, at the flood-peak discharge.
S_c	=	Channel slope, in ft./ft.
Q_{pe}	=	Effective flood-peak discharge, in cfs (may include overbank flows, where applicable).
W_e	=	Effective flow width at peak, in feet (may include overbank flows, where applicable).
D_{50}	=	Median sediment size, in millimeters.
G	=	Gradation Coefficient ($= 1/2\left[\frac{D_{50}}{D_{16}}\right] + 1/2\left[\frac{D_{84}}{D_{50}}\right]$)

(Note: If a different D_{50} size applies, based upon site-specific sediment data, *Equation 5.40* should be used.)

In both *Equation 5.40* and *Equation 5.41*, the effective peak discharge is typically meant to represent the flow discharge that is normally contained within and along the main flow path during a flood event (i.e., within the primary flow channel); but it may also separately include additional overbank flow, especially when slopes are considered steep and overland velocities are high enough to produce *significant* bed-material sediment transport in the overbanks (i.e., velocities in excess of the permissible velocity for a given D_{50} sediment size; which, for sediment-laden flow, is typically V_{ob} > 3.0 ft/sec. when D_{50} = 1.0 mm). Thus, for overbank or sheetflow areas, $Q_{pe} = Q_{ob/s}$ (i.e., the effective flow discharge in the overbanks or sheetflow area). In such instances, where velocities exceed permissible values, determination of total bed-material sediment transport within the flood plain will likely require three separate bed-material sediment-transport calculations — one for the channel, one for the left overbank, and one for the right overbank.

Once the total bed-material sediment-transport capacity at the peak, Q_{sp}, has been determined, the bulked volume (i.e., when accounting for porosity) of the bed-material sediment predicted to be transported during a specific flood can be determined from the following equation:

$$V_s = 1.5 Q_{sp} T_r \qquad \textit{(Equation 5.42)}$$

Where,

V_s	=	Bulked bed-material sediment volume, in cubic feet (based on an assumed bulked weight of 100 pounds/cubic foot [i.e., a Bulking Factor of ≈ 1.65]).
T_r	=	Rise time of the flood hydrograph, *in seconds*.

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Equation 5.42 was derived by discretizing the Pima County Synthetic Flood Hydrograph over specified time steps; then calculating and summing sediment-transport volume per time step.

The approximate rise time of the flood hydrograph, T_r, *in seconds*, can be determined from:

$$T_r = 2860\frac{C_w P_n A}{Q_p} \qquad \textit{(Equation 5.43)}$$

Where,

T_r	=	Rise time of flood hydrograph, *in seconds*.
C_w	=	Watershed-weighted runoff coefficient for the design AEP flood, dimensionless.
P_n	=	The n-hour rainfall depth for the design AEP flood, in inches. (Note: P_n should be greater than watershed time of concentration, with a maximum duration of 3 hours.)
A	=	Drainage area of watershed, in acres.
Q_p	=	Peak discharge of the design AEP flood, in cfs (typically the 1% AEP peak discharge).

<u>Example 5.2—Calculate Peak Sediment-Transport Capacity</u>

Given:

An upstream, moderately-urbanized watershed has a drainage area, A, of 640 acres (i.e., 1 square mile). The 1% AEP flood peak is computed to be 1,230 cfs. The channel bed slope of the approaching 40-foot-wide watercourse is 0.01 ft./ft.

Calculate:

The estimated bed-material sediment-transport capacity and the bulked sediment volume of a design flood generated from a 1% AEP flood event.

Solution:

Using *Equation 5.41* (i.e., the default equation to use for calculating bed-material sediment transport when site-specific bed-material sediment sizes are unavailable), the bed-material sediment-transport rate at the peak flood discharge is calculated to be:

$Q_{sp} = (0.1453)(0.01)^{0.885}(1230)^{1.44}(40)^{-0.44}$ = 13.7 cfs.

Note that, in this instance, for a flood peak of 1,230 cfs, a Q_{sp} = 13.7 cfs represents a peak bed-material sediment concentration, by weight, of approximately 2.87%, or 28,700 parts per million (ppm).

Now, assuming that the typical bulking factor for active watercourse sediments located within the Town yields a bulked bed-material sediment weight = 100 lbs./ft.^{3}, using *Equation 5.42* and *Equation 5.43* the total volume of bulked bed-material sediment that will occur during a 1% AEP 3-hour flood with a peak discharge of 1,230 cfs is calculated as follows:

$P_{3\text{-}hr}$	=	3.19 inches, which represents the upper 90% confidence interval point precipitation for a 1% AEP storm (from NOAA Atlas 14).
$C_{w1\%}$	=	0.653 for the 1% AEP flood (from Table 3-4 of Chapter 3 of the DCM).
A	=	640 acres.

Then,

$$V_s = 1.5\left(Q_{sp}\right)\left(2860\,\frac{C_w P_n A}{Q_p}\right) = (1.5)(13.7)(2860)(0.653)(3.19)(640)/(1230) = 63{,}703$$

cubic feet; or, alternatively, $V_s \approx 1.46$ acre-feet of bulked bed-material sediment. During this specific flood hydrograph, the calculated result represents, overall, an average bed-material sediment concentration throughout the design flood, by weight, of about 2.07%, or 20,700 ppm.

Note that the procedures and results presented in Section 5.6 of this DCM are based upon *Equation 5.40*, in general; and, more specifically, *Equation 5.41* within the Town. These are theoretically-developed relationships for determining the bed-material sediment-transport *capacity* of the design flow, rather than relationships based upon empirical knowledge of the bed-material sediment *supply* emanating from upstream sources—which knowledge is usually unavailable. Because the determination of sediment *supply*, whether bed-material load and/or wash load, is problematic even for small flow events, and is most often a field-measured quantity rather than theoretically calculated, it is extremely difficult to measure bed-material sediment *supply* during large flood events that occur on steep-sloped alluvial watercourses, like those watercourses commonly found within the Town. This reality means that the use of sediment-transport *capacity* relationships for estimating total bed-material load may sometimes lead to an overestimation of the actual bed-material sediment *supply* transported from upstream reaches during flood events. The possibility for overestimation of the bed-material sediment supply is highest when geomorphic features, whether natural or man-made, are located within the upstream reaches—features that might limit the incoming supply (e.g., in-line stormwater detention facilities, sand-and-gravel extraction areas, etc.).

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Zeller, M.E. (2019). *A Simplified Sediment-Transport Formula*.

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Chapter 6. Storm Drains

6.1 Introduction

The Town accepts the use of storm drains only in locations where sediment-transport concerns are either low or nonexistent (e.g., parking lots and commercial areas). Surface-drainage systems can be limiting in many situations, and properly designed storm drains can offer alternative design solutions to achieve a better overall drainage system. However, should sediment transport be an issue, which ordinarily is the case for many areas within Town limits, then storm drains are not necessarily the mechanism needed to achieve an overall better drainage solution—especially given issues with “plugging” of the system due to sediment inflow.

A storm-drain system typically consists of a series of inlets designed to intercept street flow and convey it in an underground conduit to some logical outlet, such as a natural watercourse. Curbs, gutters, and transverse street slopes all function together to collect the stormwater along either one or both sides of a street, where it can drain into the inlets. In order to understand how the complete storm-drain system operates, it is first necessary to understand how the individual components function. For design criteria and details regarding the design of storm-drain systems under pressure flow (i.e., criteria and details such as capturing street and gutter flow, determining pavement-inlet capacities, computing conduit flow-profiles [accounting for friction losses and “minor” losses]), etc., refer to Chapter X of the COTDSM (SLA, 1989/1998).

The storm-drain procedures presented herein generally follow Chapter X of the COTDSM, unless noted otherwise. In the event of conflict between the contents of the DCM and the contents of Chapter X of the COTDSM, the contents of the DCM shall take priority.

6.2 General Guidance

The following general guidance is provided for the design of storm drains:

1. A storm drain is any closed conduit that exceeds 300 feet in length, or any conduit that contains more than one manhole or more than two catch-basin inlets; or is a conduit system that, in the opinion of the Town, functions as a storm drain.
2. The minimum acceptable diameter of storm-drain conduit is 24 inches. The definition of a storm drain is provided in Item 1, above.
3. Reinforced Concrete Pipe (RCP) is required for all storm drains constructed under public streets. All privately owned storm-drain conduit shall consist of either (i) RCP; (ii) Corrugated Metal Pipe (CMP) with a smooth interior; (iii) High-Density Polyethylene Pipe (HDPP) with a smooth interior; or (iv) conduit of another acceptable and pre-approved material, as determined by the Town, and having a smooth interior. It is emphasized that conduits with smooth interiors are essential components for the construction of storm drains in order to minimize hydraulic roughness within the conduit system, thus maximizing conveyance of both stormwater and any bed-material sediments, if applicable. Although the Town will not

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mandate the use of RCP under private streets unless soils, load limitations, etc., dictate otherwise, conduits with smooth interiors are required unless it can be demonstrated that no significant bed-material sediments will be present in the drainage system. Use of RCP under private streets is encouraged if the private owner desires for the Town to take over maintenance. It is noted, though, that the Town will not accept or take over private streets for maintenance unless all new RCP storm drains are used and all existing non-RCP storm drains associated with the project site are replaced by RCP.

4. Spiral-Ribbed Pipe (SRP) may be used in other locations if approved in advance, in writing, by the Town. However, as noted in Item 3 above, this type of conduit, particularly when proposed in high sediment-transport areas of the Town, will only be permitted if it can be demonstrated that any sediment conveyed through the conduit would be as efficiently conveyed as would sediment through a conduit with a smooth interior.
5. The Town C1FHP found in Chapter 3 of this DCM shall be used for storm-drain design, unless justification for the use of another procedure is submitted and approved in advance, in writing, by the Town.
6. All onsite runoff, from whatever source, shall be taken into account when designing a storm-drain system, especially when the runoff might affect the area for which the storm drain is designed to service.
7. Storm drains on arterial streets must be designed to keep at least one lane of traffic free from runoff during a 10% AEP flow event. On lesser category streets, storm drains must keep the 10% AEP flow event between the curbs along the street.
8. Storm drains shall be designed as pressure-flow systems during the 1% AEP event. Lesser storm events may flow under open-channel conditions. Use of open-channel storm drain systems for the 1% AEP event are allowed; but approval must be obtained in advance, and in writing, from the Town.
9. The minimum allowable slope of a storm-drain system shall be 0.3 percent (0.003 ft./ft.).
10. The minimum right-of-way or easement width for a storm-drain system shall be the conduit diameter plus 10 feet on each side of the conduit.
11. A pressure analysis and computation of the Hydraulic Grade Line (HGL) and Energy Grade Line (EGL) is required for all proposed pressure-flow storm drain systems. Pressure analysis can be performed either manually, using spreadsheets, or through use of computer design software (e.g., StormCAD) for more complex systems. Pressure analysis of the HGL and EGL computations shall be provided with a calculation summary and system schematic/mapping clearly showing the HGL and EGL for each pipe segment and for the various system components. Computer outputs without explanation will not be accepted. See Section 6.3.

12. Either a backwater or a forewater analysis, as appropriate and applicable, is required for all proposed open-channel storm-drain systems, where the definition of a storm drain is provided under Item 1, above.
13. The HGL for the 1% AEP design event shall not conflict with street drainage standards, which are discussed in Chapter 9 (i.e., the HGL shall remain at least 6 inches below the finished street surface).
14. The minimal design velocity within a storm-drain system should be 4 ft./sec., based upon the hydraulics of a 20% AEP storm event.
15. The designer shall submit plan-and-profile sheets for storm drains, including the plotted HGL. The maximum system HGL through manholes shall be, at least, a minimum of one foot below the rim elevation.
16. All storm drains shall be maintained according to an approved maintenance plan described in the project Drainage Report provided by the Project Engineer and accepted by the Town.
17. In the Town, the use of plunge basins for dissipating energy (velocity) at storm-drain outlets are highly discouraged, and should not be used. Use of plunge basins shall only be considered in extraordinary cases after the items described below have been addressed and after written permission from the Town has been obtained. Plunge basins are typically excluded from consideration because historical experience with such basins has shown that they do not function as intended over the long term. These plunge basins tend to "plug" with sediment and remain filled with stormwater, often leading to an attractive nuisance and a public-health issue within the Town.

    If a plunge basin should be proposed, a report sealed by an Arizona-Registered Professional Engineer shall be submitted to the Town that satisfies the following items:

    a. The inclusion of a statement *guaranteeing* that the Town will not be responsible for or incur additional maintenance and associated costs for sediment removal or repair of downstream systems.
    b. Evidence provided that not using a plunge basin is poor engineering design, and creates a flood and/or an erosion hazard to existing properties that otherwise overrides Town maintenance concerns.
    c. Demonstrating that the plunge basin is self-cleaning and self-draining.
    d. Demonstrating that the plunge basin will drain within a maximum of six hours.
    e. Providing evidence that not using a plunge basin presents a significant financial hardship.
    f. Demonstrating that physical limitations preclude use of other energy-dissipation designs.
    g. Providing other criteria as requested and defined by the Town.

The designer should note that any new storm drains proposed to be constructed within in the Town will discharge into the Town's municipal storm-sewer system, which includes all constructed and natural washes, and must therefore meet the requirements of the Town Storm Water Management and Discharge Control Ordinance, Section 15-24-14 (2020). In addition, the designer should be aware that "clean water" emanating from urban storm-drain systems which discharge into natural wash channels likely may lead to localized long-term channel degradation and/or streambank migration, unless appropriate mitigation measures are implemented along the downstream reaches of any non-stabilized natural or altered/man-made watercourses.

Furthermore, the designer should note that any new storm drains, in particular ones that utilize a natural wash as an outfall, may have additional special environmental permitting considerations. Moreover, the designer also should be aware that constructing an outfall that discharges into WUS may require a permit under Section 404 of the Clean Water Act.

6.3 Hydraulic Grade Line (HGL)

As stated previously herein, it is a requirement of the Town to compute the design HGL for pressure flow when proposing installation of a storm-drain system.

The HGL is computed by starting with the control tailwater elevation located at the storm-drain outlet, and subsequently performing an HGL calculation in the upstream direction. Friction and minor losses are compared 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 HGL is equal to the total energy grade line, minus velocity head at any point along the storm drain. The design of each storm-drain system shall be accompanied by an HGL Calculation Sheet, an example of which is provided as Figure 6-1.

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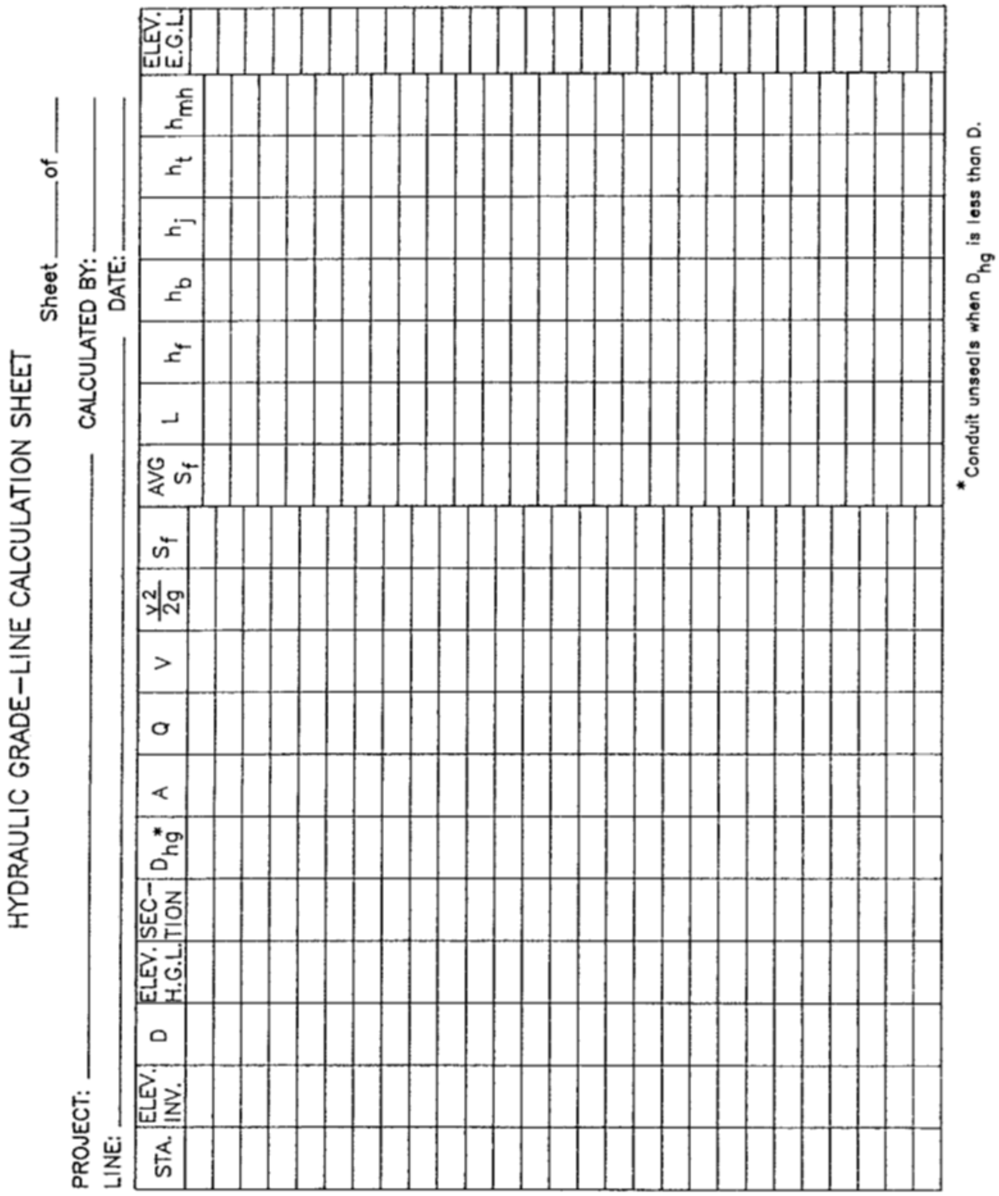

HYDRAULIC GRADE—LINE CALCULATION SHEET

Sheet ______ of ______

PROJECT: ______ CALCULATED BY: ______

LINE: ______ DATE: ______

STA.	ELEV. INV.	D	ELEV. H.G.L.	SEC-TION	D_{hg}*	A	Q	V	$\frac{V^2}{2g}$	S_f	AVG S_f	L	h_f	h_b	h_j	h_t	h_{mh}	ELEV. E.G.L.

*Conduit unseals when D_{hg} is less than D.

Figure 6-1. Example Hydraulic-Grade-Line Calculation Sheet for Pressure Flow

6.4 Criteria

The general criteria for storm drain design within the Town should adhere to the COTDSM (SLA, 1989/1998), except where noted otherwise within this chapter. Table 6-1 summarizes and directs the user to specific report sections and applicable equations in the COTDSM for use in designing the various storm-drain components. Chapter X of the COTDSM is provided as an appendix to Chapter 6 of this DCM as a convenience to the user.

Table 6-1. Storm Drain Design Criteria Guidance

Component	Parameter	COTDSM Section	COTDSM Equations	Comment
Design Discharge and Hydrology	Peak Flow	N/A	N/A	Use of C1FHP, as Discussed in Chapter 3 and Chapter 9
Street and Gutter Flow	Capacity	10.5	10.1 – 10.9	See Chapter 9 for Specific Criteria; Use Section 10.5 of COTDSM for Hydraulic Calculations
Pavement Inlets	Capacity of a Grate Inlet in a Sag	10.6.1	10.10 – 10.13	Per COTDSM
	Capacity of a Curb Inlet in a Sag	10.6.2	10.14 – 10.16	Per COTDSM
	Capacity of a Combination Inlet in a Sag	10.6.3	N/A	Per COTDSM
	Capacity of a Slotted Inlet in a Sag	10.6.4	10.17	Per COTDSM
	Capacity of a Grate Inlet on a Continuous Grade	10.6.5	10.19 - 10.21	Per COTDSM
	Capacity of a Curb Inlet on a Continuous Grade	10.6.6	10.22 – 10.26	Per COTDSM
	Capacity of a Combination Inlet on a Continuous Grade	10.6.7	N/A	Per COTDSM
	Capacity of a Slotted Inlet on a Continuous Grade	10.6.8	N/A	Per COTDSM
	Clogging	10.6.9	N/A	Per COTDSM
Inlet Design Procedure	Inlet Locations	10.7	N/A	Per COTDSM
Storm-Drain Calculations	Normal-Depth Calculations	10.8.1	10.27	Per COTDSM
	Pressure-Flow Calculations: Computation of Hydraulic Grade Line	10.8.2	10.28	Per COTDSM
	Friction Losses	10.8.3	10.29 – 10.30	Per COTDSM
	Minor Losses	10.8.4	10.31	Per COTDSM
	Bend Losses	10.8.5	10.32 – 10.33	Per COTDSM
	Junction Losses	10.8.6	10.34 – 10.36	Per COTDSM
	Transition Losses	10.8.7	10.37 – 10.38	Per COTDSM
	Manhole Losses	10.8.8	10.39	Per COTDSM
	Entrance and Outlet Losses	10.8.9	10.40 – 10.41	Per COTDSM
Storm Drain Design Procedure	Preliminary Design	10.9.1		Per COTDSM or as Specified in Section 6.2, this Chapter
	Final Pipe Sizing: Hydraulic Grade-Line Calculations	10.9.2		Per COTDSM

6.5 Component Preferences

The principal, or so-called “major,” hydraulic components in a storm-drain drainage system consist of inlets, conduits, junctions, manholes, and outfalls. Each of these system components influence the system, although inlets and conduits typically affect system performance the most.

The Town preferences for the various storm-drain components are provided in Table 6-2.

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Table 6-2. Component Preferences

Component	Detail Type
Catch Basins - Curb Inlets	PAG Standard Detail 308, Catch Basin Type 3 and Standard Specification Section 503
Catch Basins - Drop Inlets	PAG Standard Detail 309, Catch Basin Type 4 and Standard Specification Section 503
Catch Basins - Combination Inlets	PAG Standard Detail 307, Catch Basin Type 1 and Standard Specification Section 503
Culverts (including inlet and outlet headwalls)	ADOT Standard Details 6.01 thru 6.36 and PAG Standard Specification Section 501 and 502
Sidewalk Scuppers	PAG Standard Details 205 and 205.5
Curb Cuts	PAG Standard Details 209/210, Curb Transition
Manholes	PAG Standard Details 300, 301, 302, 303, and 304
Conduits	PAG Standard Specification Section 501
Outfalls	Consultant-designed component(s) that are required in order to reduce outlet velocities to non-erosive values that are acceptable to the town*

*Stated as such because neither PAG nor ADOT address "Outfalls" in their standard details or standard specifications.
Source: PAG (2015).

6.6 Storm-Drain Design Checklist

The following checklist is provided as a guide regarding the minimum items necessary for appropriate storm-drain design. It is the obligation of the designer to ensure that any additional information that is necessary for the specific storm-drain design be identified and provided to the Town, and that all such information be technically accurate and correct. In this regard, the following items should be included within, or on, any submitted reports and/or plans.

1. A map of the project site showing the storm-drain alignment in relation to streets, drainageways, and land uses.
2. An offsite watershed map, at a scale no smaller than 1" = 200', with at least 2-foot contour intervals superimposed upon aerial topography. All points of concentration and watershed definitions/delineations shall be clearly shown on these maps.
3. All offsite hydrologic data sheets and supporting information for offsite drainage calculations.
4. Site information for offsite drainage concentration points. This should include hydraulic information and calculations at the points where offsite drainage is to be intercepted. Two main points to consider are:
    a. Is the proposed storm-drain inlet adequate to capture all of the offsite flow it is designed to intercept?
    b. Is there enough information provided to determine what impact flow into the inlet will have upon the downstream storm-drain system?
5. Proposed street cross-sections, grades, and compositions. These should be in the form of plan-and-profile sheets, with street cross-sections shown.

6. Calculations showing street capacities at the allowable design limit of flow spread along the entire length of the storm drain. Points where drainage must be entirely (i.e., 100 percent) removed from the street should also be located.

7. Inlet calculations should be performed according to the procedure outlined in Section 10.8 of the COTDSM (SLA, 1989/1998). All hydrologic data sheets used to conduct the calculations should be included, as well as a description of all assumptions, all supporting calculations, and all proposed inlet types and sizes. Detailed watershed maps should also be included showing drainage areas, land uses, and topography. These maps should be of a more detailed scale than 1" = 200'. A scale no smaller than 1" = 100' shall be employed. A scale of 1" = 40' is preferable, where appropriate.

8. All hydrologic data sheets and supporting calculations for discharges used in conduit sizing. Additional watershed maps should be provided, if necessary, in order to show larger watershed delineations.

9. Since the storm drain shall be designed for pressure flow, the intermediate calculations for conduit sizing need not be included. However, final storm-drain calculations must be included. The submittal of the final storm-drain design shall include HGL calculations. There should be a storm-drain profile drawn to scale and showing, at a minimum, the storm-drain soffit and invert, ground surface, HGL, energy grade line, outlet-control elevation, manholes, junctions, transitions, bends, and inlets. A plan-view map should be included which shows the storm drain, adjacent streets, and proposed inlets and manholes. The calculation of minor losses should be provided, and should include a clearly labeled diagram of the structure, or structures, involved. Calculations and supporting ground information are required showing how the controlling WSEL was determined. In this regard, an HGL Calculation Sheet shall be included.

10. Proprietary software may be used for storm-drain design, however the computational output must clearly specify the hydraulic parameters used to represent the system, as well as boundary conditions (e.g., tailwater elevations). Furthermore, the summary output must be in a format that closely follows Figure 6.1.

11. There should be a clear, concise, text description of the design process, and the assumptions upon which the analysis was based. The drainage report should follow the Town guidelines for the submittals of drainage reports, as presented in Chapter 2 of this DCM.

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References

Oro Valley Town Code Chapter 15 Water Code (2020). https://orovalley.town.codes/TC/15

Pima Association of Governments (PAG). (2015). *Standard Specifications and Details for Public Improvements*.

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Chapter 7. Culverts and Bridges

7.1 Criteria

Culverts and bridges are important elements of modern urban drainage, primarily for the reason that their intended purpose is to provide continuous transportation access in times of high drainage flows. Consequently, this DCM provides an outline for the hydraulic design of such structures. Nevertheless, it is the designer's responsibility to utilize appropriate techniques to ensure the structural integrity of the facility under both dry conditions and wet conditions (e.g., uplift forces or localized scour mitigation). Analysis and design of structures shall generally follow PAG *Standards and Specifications and Details for Public Improvements* (2015). The design criteria outlined in the following sections shall apply to the design of culverts and bridges with Town limits.

7.1.1 General Requirements

1. Determine if the proposed culvert or bridge crossing is located within a regulatory floodplain; and, if so, the designer shall incorporate the standards of the Town Floodplain and Erosion Hazard Management Ordinance.
2. At-grade wash crossings (i.e., so-called "dip" crossings) of arterial and collector streets are discouraged in the Town, and will only be allowed under special circumstances. Written permission from the Town is required for authorizing the use of at-grade crossings. Refer to Chapter 9, Section 9.4, for additional guidance.
3. The designer shall show ponding or backwater limits for existing and proposed conditions for the 1% AEP flood event as well as the design event, if other than the 1% AEP flood event. The Town may require easements or other forms of compensating mitigation where proposed conditions would increase offsite ponding limits.
4. Culverts shall be designed to receive and all floodwaters conveyed, and all bed-material sediments transported, during a 1% AEP flood.
5. The minimum acceptable height of box culverts and arch culverts is five (5) feet, regardless of design discharge, in order to provide headroom for possible maintenance. A lessor height may be allowed with prior written approval from the Town, if it can be successfully shown that the planned culvert will receive and convey both floodwaters and bed-material sediments without causing backwater in the channel or the filling of the culvert barrel with sediments. All such structures shall be appropriately sized to substantially eliminate sediment deposition within and immediately upstream of the culvert.
6. Physical and legal access to the inlet and outlet areas of the culverts shall be provided for both public and private maintenance purposes.

7. Drop inlets into culverts are discouraged within the Town, and will only be allowed under special circumstances. If culvert inverts are to be placed below the existing wash flowline, thus necessitating a drop inlet, then an inclined inlet chute will be required that is made of concrete and has a typical slope (maximum) of 10% (i.e., 10H:1V).
8. Critical depths of flow at culvert entrances must equal, or be less than, approach flow depths.

7.1.2 Culverts

1. A primary concern with culverts in the Town is sedimentation, particularly at their entrances. Oftentimes, bed-material sediment that is transported with the approaching flow fills a culvert, either partially or completely, which in turn prevents the culvert from functioning properly by inhibiting the conveyance of stormwater underneath the roadway. Consequently, where bed-material sediment-transport is significant, the following procedures shall be used to minimize the potential for sedimentation to occur at the entrance to, and inside of, culverts:
   a. Maintain a headwater to culvert diameter (HW/D) ratio, or headwater to culvert height (HW/H) ratio, that is significantly smaller than 1.0. The Town requires the design to be such that the HW/D ratio or HW/H ratio is no greater than 0.667.
   b. Use a culvert(s) that is as wide, or nearly as wide, as the design flow-width of the approach channel. However, an overly-wide culvert system shall not be used.
   c. When and where practicable, install the culvert(s) at the same gradient as the approach channel. If a portion of the culvert is to be installed below the natural channel grade line, unless floodwaters are free of bed-material sediments it must be assumed that only the portion of the cross-sectional area of the culvert which is at and above the natural bed elevation will be effective in conveying both the floodwaters and bed-material sediments.
   d. Avoid angular deviation by installing culverts so that they are aligned parallel to the approach channel.
   e. Avoid any design that reduces flow velocities immediately upstream of the entrance of the planned culvert in order to maintain incoming bed-material sediment-transport rates.
2. The minimum acceptable diameter of a pipe culvert shall be twenty-four (24) inches along natural washes and sandbed channels, unless approved in writing by the Town. In this regard, a minimum-diameter pipe culvert must be able to accept and convey incoming sediment, and satisfy the requirements of Subsection 7.1.1.8, above, and not create any rise in headwater.
3. Headwalls are required for all culverts thirty-six (36) inches in diameter, or greater. Acceptable end treatments for culverts less than thirty-six (36) inches in diameter are required, and shall be approved by the Town.
4. Where little to no bed-material sediment-transport exists, headwater depth shall not exceed 1.5 times the pipe diameter or rise of a culvert, and in no case shall exceed one foot in height

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above the existing 1% AEP WSEL. Where bed-material sediment-transport does exist, headwater depth shall be at least 1 foot below the soffit of the pipe or culvert; and shall not exceed 0.667 (2/3) times the pipe diameter or rise of the culvert, excluding the arched portion of a concrete culvert. In all cases, additional ponding on adjacent properties shall not exceed 0.1 feet in depth, as stipulated in the Town Floodplain and Erosion-Hazard Management Ordinance.

5. A backwater analysis is required for all culvert crossings, using recognized standards such as those found in the third edition of the Federal Highway Administration (FHWA) document, *Hydraulic Design of Highway Culverts* (FHWA, 2012).
6. Reinforced Concrete Pipe (RCP) or Reinforced Concrete Culvert (RCC) shall be used for all pipe culverts constructed under public streets. RCP or RCC is strongly recommended for use under private streets as well. For all other locations, concrete pipe, corrugated metal pipes *with a smooth interior*, or other similar materials, are recommended on all projects within the Town limits. Due to the historical accumulation of bed-material sediments, corrugated metal pipe is not recommended unless it can be demonstrated to the satisfaction of the Town that there would be no adverse impact, such as abrasion and/or corrosion, created by bed-material sediment transport.
7. The Town ordinarily accepts a minimum depth of cover for culverts of 2.0 feet. If a smaller depth of cover is desired, the Design Engineer shall provide manufacturers specifications that indicate that a smaller depth of cover is acceptable. A smaller depth of cover is subject to acceptance by the Town.
8. The minimal design velocity in a culvert shall be one that is sufficient to convey the majority of bed-material sediments through the culvert barrel. This criterion can be waived if it can be demonstrated that the probability of bed-material sediment is minimal (e.g., in large paved parking lots that are protected from offsite flows).
9. Protection is required at culvert outlets, per the outlet-velocity constraints in Table 7-1.

Table 7-1. Culvert Outlet Protection

Outlet Velocity	Outlet Protection
< 10 fps	Riprap is required, with the minimum extent of riprap able to cover the calculated scour hole.
> 10 fps, but < 15 fps	Rock-faced concrete, with an acceptable low-flow outlet that covers, at a minimum, the calculated scour hole.
> 15 fps	An acceptable energy dissipator. Approval from the Town is required prior to design of such a structure.

10. In general, the construction of outlet plunge basins is discouraged in the Town, and will only be allowed under special circumstances, and only after the items described below have been addressed and after prior written permission from the Town has been obtained. A report sealed by an Arizona-Registered Professional Engineer shall be submitted to the Town, demonstrating/ensuring the following items are addressed:

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a. That the Town will not incur additional maintenance and associated costs for sediment removal or repair of the downstream fluvial system.

b. That in a specific instance not using a plunge basin is poor engineering design, and creates a flood and/or erosion hazard to existing properties.

c. That a plunge basin, if utilized, will be self-cleaning and self-draining.

d. That the plunge basin will drain within a maximum duration of six (6) hours.

e. That not using a plunge basin will present a significant financial hardship.

f. That physical limitations preclude the use of another type of energy-dissipation design.

g. That other criteria will be provided, as requested and defined by the Town.

11. As an acceptable alternative to standard concrete plunge basins, the designer may choose to use loose-riprap blankets for outlet erosion protection. Loose-riprap blankets avoid several of the negative aspects of concrete. When choosing to use a loose-riprap blanket, the Design Engineer shall place a minimum 1-foot-wide concrete header at the downstream toe of the blanket in order to prevent loss of stone. Two reference sources for loose riprap blankets are (1) *Technical Manual: Outlet Works Energy Dissipators Best Practices for Design, Construction, Problem Identification and Evaluation, Inspection, Maintenance, Renovation, and Repair* (FEMA, 2010); and (2) *Erosion and Riprap Requirements at Culverts and Storm-Drain Outlets* (Bohan, 1970).

12. Culverts of sufficient size shall be provided to pass both floodwaters and bed-material sediments underneath streets without any flow diversion(s) to adjacent properties.

13. The applicable guidelines contained within the preceding 12 items apply equally to Reinforced Concrete Box Culvert (RCBC) and Reinforced Concrete Arch Culvert (RCAC) systems.

7.1.3 Estimating Sediment Impacts at Culverts

Within the Town, due to the presence of steep channel slopes it is generally assumed that culvert systems will operate under inlet-control. Evidence must be provided to the Town when the case is otherwise (e.g., when outlet-control occurs).

The following equations are intended to be used to determine the capability of either pipe culverts or box culverts to transport the approaching bed-material sediments past their culvert entrances and through their culvert barrels. However, in order to determine the capability of a culvert system to pass the bed-material sediment beyond its entrance, for inlet-control conditions it shall be assumed that critical depth occurs at the entrance for the discharge involved unless it can be clearly demonstrated that the flow passing into and through the culvert is acting entirely as open-channel flow. The culvert may be placed on a slope greater than critical, but in such instances it shall be assumed that critical slope will occur at the culvert entrance during the design discharge; meaning that critical depth and critical velocity will also occur, unless true open-channel flow occurs at the entrance and everywhere along and within the culvert system (i.e., supercritical flow occurs). However, if the tailwater at the outlet exceeds the soffit of the culvert, potentially causing outlet control, then a hydraulic grade line shall be calculated through the culvert, and the resulting friction slope shall be used in the following applicable equations, in lieu of the culvert slope.

In order to determine the appropriate size of a culvert entrance so that the culvert will be capable of passing the approaching bed-material sediment, the following procedure shall be employed.

When $0.3 < \frac{Y_c}{D_o} < 0.9$, a formula (Chow, 1959) for determining the approximate critical depth, Y_c, at the brink of the entrance of a circular pipe with diameter D_o is:

$$Y_c = 0.325\left(\frac{Q_p}{ND_o}\right)^{2/3} + 0.083D_o \qquad \textit{(Equation 7.1)}$$

Where,

Y_c	=	Critical depth of flow at the brink of the entrance of the circular pipe, in feet.
D_o	=	Diameter of the circular pipe, in feet.
Q_p	=	Peak discharge in approach channel, in cfs.
N	=	Number of circular pipes.

Now, in order to prevent sedimentation at the entrance of a pipe culvert when sediment-laden flow approaches, the critical depth, Y_c, at the pipe entrance shall not exceed $0.667D_o$. Therefore, setting $Y_c = 0.667D_o$ and rearranging *Equation 7.1* yields,

$$D_o = 0.7035\left(\frac{Q_p}{N}\right)^{0.4} \qquad \textit{(Equation 7.2)}$$

Critical depth at the brink of the entrance of a box culvert can be determined from,

$$Y_c = 0.315\left[\left(\frac{Q_p}{bN}\right)^2\right]^{1/3} \qquad \textit{(Equation 7.3)}$$

Where,

Y_c	=	Critical depth of flow at the brink of the entrance of the box culvert, in feet.
b	=	Bottom width of the box culvert, in feet.
Q_p	=	Peak discharge in approach channel, in cfs.
N	=	Number of box culverts.

As with the circular pipe, when sediment-laden flow approaches a box culvert, the critical depth, Y_c, shall not exceed $0.667H$, where H = the height of the box, in feet.

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Therefore, setting $0.667H = 0.315\left[\left(\frac{Q_p}{bN}\right)^2\right]^{1/3}$ yields,

$$b = 0.3245\left(\frac{Q_p}{NH^{3/2}}\right) \qquad \textit{(Equation 7.4)}$$

Note: Where a RCAC system is proposed to be used in lieu of a RCBC, the designer shall use only that vertical portion of the sidewall height that is located below the arched portion of the RCAC for purposes of determining the effective width-height dimensions necessary to accommodate bed-material sediment transport.

<u>Example 7.1—Smooth-Barrel Pipe-Culvert System</u>

Given:

A sediment-laden flow of 400 cfs approaches a proposed roadway crossing. The approach channel has an effective flow width of 20 feet and a slope of 0.02 ft./ft. The approach depth and flow velocity are assumed to be at critical flow; thus, approach flow depth equals critical depth = 2.32 feet, and flow velocity = 8.64 ft./sec.

Calculate:

Calculate the size of the circular pipe system required to convey both the flow and the bed-material sediment past the culvert entrances the through the culvert barrels.

Solution:

In order to pass approaching bed-material sediments through the entrance, the pipe diameters must be equal to or greater than 1.5 Y_c at the culvert entrances. Furthermore, the critical depth at the pipe entrances must be equal to or less than the approach flow depth (i.e., $Y_{cp} \leq$ 2.32 feet). In order to satisfy these criteria, initially select eight circular smooth-barrel pipes for placement underneath the proposed roadway.

Using *Equation 7.2*, $D_o = 0.7035\left(\frac{400}{8}\right)^{0.4} = 3.36$ feet, and therefore critical depth at the pipe entrances will be approximately 2.24 feet; which is less than 2.32 feet, and thus acceptable. Round the pipe diameter up, and use eight, 3.5-foot-diameter (i.e., 42-inch-diameter) smooth-barrel circular pipes. Including the spacing for eight pipes will require a total width of about 44 feet. Thus, this pipe system will be significantly wider than the 20-foot effective width of the approach channel, and therefore an alternate system should be considered for passing the flow (e.g., box culverts).

Continuing, at a critical depth of 2.24 feet the flow area in each pipe = 6.4 sq. ft. (i.e., the total flow area of all eight pipes equals 51.2 sq. ft.). Thus, the entrance velocity at each pipe = 7.81 ft./sec. This velocity is less than the approach flow velocity of 8.64 ft./sec., and therefore would likely not transport all bed-material sediment past the culvert entrances, although the additional culvert height of 1.26 feet that is available might accommodate some sediment deposition that would occur at the entrance to the culvert system.

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Based upon these results, an eight-barrel circular pipe-culvert system for this flow does not work well, and it is therefore recommended that a box-culvert system be evaluated as an alternative (see Example 7.2).

Example 7.2—Box-Culvert System

Given:

A sediment-laden flow of 400 cfs approaches a proposed roadway crossing. The approach channel has an effective width of 20 feet and a slope of 0.02 ft./ft. The approach depth and flow velocity are assumed to be at critical flow; thus, approach flow equals critical depth = 2.32 feet, and flow velocity = 8.64 ft./sec.

Calculate:

Calculate the size of the box-culvert system required to convey both the flow and the bed-material sediment past the culvert entrances.

Solution:

Select a two-cell box-culvert system to place underneath the proposed roadway. The height of each box culvert must be equal to 1.5 Y_c at the entrance to the box-culvert system. Critical depth at the box-culvert entrances must be equal to or less than the critical depth of the approach flow = 2.32 feet. In this case, use 2.32 feet. Therefore, H = 1.5 Y_c = 3.48 feet. However, per Town requirements, for maintenance purposes a minimum box-culvert height of 5.0 feet is selected.

Using *Equation 7.4*, with a calculated height of 3.48 feet $b = 0.3245\left(\frac{400}{2(3.48)^{1.5}}\right) = 10$ feet. Therefore, use b = 10 feet. Spacing for a two-cell 10-foot-wide by 5-foot-high box-culvert system will require a total width of 23 feet. In this case, the box-culvert system will be only slightly wider than the 20-foot effective width of the approach-channel flow, and therefore will be acceptable.

At a critical depth of 2.32 feet at the culvert entrances, the entrance flow area at each box-culvert is 23.2 sq. ft. (total flow area of the two box culverts = 46.4 sq. ft.). Therefore, the entrance velocity is 8.62 ft./sec. This velocity is essentially the same as the approach flow velocity of 8.64 ft./sec., and should therefore transport the majority of the bed-material sediment past the culvert entrance, with an additional height of 2.68 feet available to accommodate any sediment deposition that might occur at the entrances to the two cells of the box-culvert system.

It is mentioned again that, in order to provide ease of maintenance of a box-culvert system, the Town requires that the minimum height of each box shall not be less than 5.0 feet.

Once culvert entrance sizes have been determined, the procedure provided below should be used to verify the capacity of a culvert barrel to pass the approaching bed-material sediments all the way through the culvert and into the downstream reaches of the channel.

The following equation, which is adapted from Graf & Acaroglu (1968) for use with this DCM, estimates bed-material sediment-transport capacity in a *smooth-barrel* culvert:

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$$Q_s = [(7325)D_{50}^{-1.02}S_o^{2.52}R^{1.52}A]N \qquad \textit{(Equation 7.5)}$$

Where,

Q_s = Bed-material sediment transport in a smooth-barrel pipe or culvert, in cfs.

D_{50} = Mean sediment diameter, in *millimeters*.

S_o = Slope of smooth-barrel culvert, in feet/foot.

R = Hydraulic Radius of smooth-barrel culvert, in feet.

A = Cross-sectional area of smooth-barrel culvert, in square feet.

N = Number of smooth-barrel culverts (pipe or box).

When applying *Equation 7.5*, the Engineer should use site-specific bed-material sediment data obtained from a sufficient number of sieve analyses in order to properly characterize the bed-material sediment composition within the study reach of the watercourse. However, lacking the availability of site-specific bed-material sediment size data, it can be assumed that a typical mean sediment diameter for the region encompassing the Town is D_{50} = 1 millimeter. Thus, *Equation 7.5* can be simplified, as follows:

$$Q_s = [(7325)S_o^{2.52}R^{1.52}A]N \qquad \textit{(Equation 7.6)}$$

Where the parameters are as defined in *Equation 7.5*.

However, should D_{50} equal a value other than 1 millimeter, based upon site-specific sediment size data, then the calculation of Q_s shall be based upon the use of *Equation 7.5*.

In order to assure that the bed-material sediment-transport capacity in the smooth-barrel culvert equals or exceeds the bed-material sediment-transport capacity of the approaching channel (i.e., conveyance capacity equals or exceeds sediment supply [$Q_{sp} \geq Q_s$]), *Equation 5.41*, contained in Chapter 5 of this DCM, must be equated to *Equation 7.6*. Therefore,

$$(0.1453)\left(S_c^{0.885}Q_{pe}^{1.44}W_e^{-0.44}\right) = [(7325)(S_o^{2.52}R^{1.52}A)](N).$$

Where the parameters are as defined in *Equation 7.5 as well as* in *Equation 5.41*, found in Chapter 5 of this DCM.

For a circular smooth-barrel culvert, $R = D_o/4$ and $A = \pi D_o^2/4$, where D_o = diameter of the culvert, in feet. Substituting these two identities into the preceding equality and solving for D_o yields:

$$D_o = (0.0899)\left(\frac{S_c^{0.2514}}{S_o^{0.7159}}\right)Q_{pe}^{0.4091}W_e^{-0.125}N^{-0.2841} \qquad \textit{(Equation 7.7)}$$

Equation 7.7 applies to a circular smooth-barrel culvert.

(Note: If D_{50} and G are other than 1 millimeter and 3, respectively, the numerical coefficient in *Equation 7.7* will change. Hence, *Equation 5.40* must be evaluated for a different D_{50} and G, and then be set equal to *Equation 7.5*, rather than setting *Equation 5.41* equal to *Equation 7.6*.).

Example 7.3—Bed-Material Sediment-Transport Capacity of a Smooth-Barrel Pipe Culvert

Given:

A sediment-laden flow of 400 cfs approaches a proposed roadway crossing. The approach channel has an effective width of 20 feet and a slope of 0.02 ft./ft. The approach depth and flow velocity are assumed to be at critical flow; thus, approach flow depth equals critical depth = 2.32 feet, and the approach flow velocity = 8.64 ft./sec.

Calculate:

Calculate whether the size of the circular smooth-barrel pipe system determined in Example 7.1 is large enough to transport the approaching bed-material sediments through the barrels and into the downstream channel.

Solution:

For the eight-barrel circular pipe-culvert system in Example 7.1, set the interior culvert slopes to 0.02 ft./ft. (i.e., the same as the slope of the approach channel). In order to determine the minimum required culvert diameter to convey the approaching bed-material sediments through the smooth barrel of each pipe, use *Equation 7.7*.

$D_o = 0.0899\left(\frac{(0.02)^{0.2514}}{(0.02)^{0.7159}}\right)(400)^{0.4091}(20)^{-0.125}(8)^{-0.2841} = 2.44$. Use D_o = 2.5 feet (30 inches).

Since D_o = 2.5 feet is less than the adopted D_o of 3.5 feet in Example 7.1, instead assume D_o = 3.5 feet. Comparing the results of *Equation 7.6* to the results of *Equation 5.41* for D_o = 3.5 feet yields Q_{sp} = 24.08 cfs for *Equation 7.6* and Q_{sp} = 6.81 cfs for *Equation 5.41*. Thus, the calculations and the comparison indicate that a system of eight smooth-barrel pipes, each with a diameter of 3.5 feet, can easily transport the approaching bed-material sediments through their barrels and into the downstream reach of the channel; that is, *assuming all of the bed-material sediment can pass the culvert entrances*.

Equation 7.6 can also be used to algebraically determine the minimum slope that would be required in order to transport at least 6.81 cfs of bed-material sediment through the eight, 3.5-foot-diameter smooth-barrel pipe culverts. The minimum calculated slope = 0.008 ft/ft. This means that placing the eight pipe culverts on any slope less than 0.008 ft./ft. would potentially lead to the deposition of bed load in the culvert barrels. However, in order to account for nonuniform flow and additional energy losses associated with high rates of bed-material sediment transport, it is required that the minimum slope of any pipe culvert be placed on a slope not less than 1.3 times the minimum calculated slope; which, in this particular case, would be equal to a slope of 1.3 x 0.008 ft./ft. = 0.0104 ft./ft. Note that the critical slope for a flow of 400 cfs given this particular pipe-culvert system = 0.0111 ft./ft., which assumes that n = 0.020 for the culvert due to the resultant low flow velocity that causes bed-load sediment to be in contact with the culvert inverts as the flow moves through each of the pipe-culvert barrels.

For a square or rectangular culvert (i.e., a box) $R = bH/(2[b+H])$ and $A = bH$, where b = culvert width, in feet, and H = culvert height, also in feet. Substituting these two identities into *Equation 5.41* and *Equation 7.6*, then setting them equal to each other and solving, yields:

$$\frac{(bH)^{2.52}}{(b+H)^{1.52}} = 5.69x10^{-05}\left(\frac{S_c^{0.885}}{S_o^{2.52}}\right)Q_{pe}^{1.44}\left(\frac{W_e^{-0.44}}{N}\right) \qquad \textit{(Equation 7.8)}$$

Equation 7.8 requires an iterative solution. In order to assist with the solution, a regression analysis on bH yields:

$$bH \approx 2.0X^{0.5555} \qquad \textit{(Equation 7.9)}$$

Where $X = 5.69x10^{-05}\left(\frac{S_c^{0.885}}{S_o^{2.52}}\right)Q_{pe}^{1.44}\left(\frac{W_e^{-0.44}}{N}\right)$.

Once bH has been determined, the width, b, can be determined by selecting an appropriate culvert width that yields a sediment-transport rate that matches or exceeds the approach-channel sediment-transport capacity.

<u>Example 7.4—Box Culvert Sediment-Transport Capacity</u>

Given:

A sediment-laden flow of 400 cfs approaches a proposed roadway crossing. The approach channel has an effective width of 20 feet and a slope of 0.02 ft./ft. The approach depth and flow velocity are assumed to be at critical flow; thus, approach flow depth equals critical depth = 2.32 feet, and flow velocity = 8.64 ft./sec.

Calculate:

Calculate whether the size of the box-culvert system determined in Example 7.2 can transport approaching bed-material sediment entirely through each barrel.

Solution:

For the two box culverts in Example 7.2, set the culvert slope at 0.02 ft./ft. (i.e., the same as the slope of the approach channel). In order to determine the required culvert width and height needed to convey bed-material sediment through the box, use *Equation 7.8* and *Equation 7.9*.

$\frac{(bH)^{2.52}}{(b+H)^{1.52}} = 5.69x10^{-05}\left(\frac{(0.02)^{0.885}}{(0.02)^{2.52}}\right)(400)^{1.44}\left(\frac{(20)^{-0.44}}{2}\right) = 25.5$. Thus,

$bH \approx 2.0(25.5)^{0.5555} = 12.1$.

Set H = 3.5 feet (i.e., ≈ 1.5 times critical depth). Therefore, b = 12.1/3.5 ≈ 3.5 feet. However, for maintenance purposes the minimum height of a box culvert in the Town shall not be less than 5 feet. For geometric conformity, use b = 5 feet. Therefore, the preceding calculation indicates that two, 5-foot-wide by 5-foot high box culverts should be of sufficient size to transport 6.81 cfs of approaching bed-material sediment through their culvert barrels. Applying *Equation 7.6*, the bed-material sediment-transport capacity within the barrels of two 5-foot-wide by 5-foot-high box culverts on a 0.02 percent slope is calculated to be 26.90 cfs, which is significantly more than the approaching bed-material sediment-transport capacity of 6.81 cfs. However, based upon entrance conditions, the box-culvert sizes calculated in Example 7.2 that are necessary in order to transport both the peak flow and the approaching bed-material sediments past the culvert entrances requires the installation of two, much larger, 10-foot-wide by 5-foot-high box culverts. Once past the culvert entrances, the two 10-foot-wide by 5-foot-high box culverts are capable of transporting 83.29 cfs of bed-material sediment within and through the culvert barrels. This example demonstrates that, for *inlet-control conditions, entrance conditions are what "control" the bed-material sediment-transport capacity of a box-culvert system, rather than the sizes of the barrels of the culverts*.

Accordingly, using bH = 50 square feet (i.e., a 10-foot-wide by 5-foot-high RCBC), and comparing the results of *Equation 7.6* to the results of *Equation 5.41*, yields Q_{sp} = 83.29 cfs for *Equation 7.6* and Q_{sp} = 6.81 cfs (i.e., the approaching bed-material sediment being transported) for *Equation 5.41*. Thus, once the flow passes the culvert entrances, a two-cell 10-foot-wide by 5-foot-high box-culvert system would be more than capable of transporting the approaching bed-material sediment through its two culvert barrels and into the downstream reaches of the channel.

Using *Equation 7.6* to determine the minimum slope that would be required in order to transport 6.81 cfs of bed-material sediment through the two-cell 10-foot-wide by 5-foot-high culverts yields a minimum calculated slope = 0.0074 ft./ft. This means that placing the two box culverts on any slope less than 0.0074 ft./ft. would potentially lead to the deposition of bed load in the box-culvert barrels. However, as with Example 7.3, in order to account for nonuniform flow and additional energy losses associated with high rates of bed-material sediment transport, it is required that the minimum slope of any box-culvert system be placed on a slope not less than 1.3 times the minimum calculated slope; which, in this particular case, would be equal to a slope of 1.3 x 0.0074 ft./ft. = 0.0096 ft./ft. Note that the critical slope for a flow of 400 cfs, given this particular box-culvert system, is Y_c = 0.0073 ft./ft., which assumes that n = 0.020 for each culvert due to the resultant low flow velocity in each culvert that causes bed-load sediment to be in contact with the culvert inverts as the flow moves through each of the box-culvert barrels.

<u>Example 7.5—Wide Floodplain Flow Approaching Culverts</u>

Given:

A sediment-laden flow of 2,200 cfs approaches a proposed roadway crossing in a wide, compound flood plain. The approaching flood plain has a main flow path (channel) with an effective width of 25 feet, and a right-overbank flood plain with an effective width of 475 feet, for a total effective flow width of 500 feet. The slope of the approaching compound flood plain is 0.028 ft./ft. Design a box-culvert system to safely convey both the flow and the bed-material sediments.

Calculate:

The size of a box-culvert system necessary in order to safely convey both the flow and the bed-material sediments.

Solution:

Assume that the slopes of the box-culvert barrels also will be placed on a slope of 0.028 ft./ft. The approach depths and flow velocities are assumed to be at critical flow. Therefore, the approach flow has a channel discharge of 970 cfs and a right-overbank discharge of 1,230 cfs. The critical flow depths are 3.6 feet in the channel and 0.6 feet in the right overbank; and the critical flow velocities are 10.77 ft./sec. and 4.4 ft./sec. in the channel and right overbank, respectively.

In order to assure that there will be adequate bed-material sediment-transport capacity at the box-culvert entrances, the velocity at the entrance to each box culvert must be equal to or greater than the approach velocity of the main flow path (channel), which is 10.77 ft./sec. At the culvert entrance, it is assumed that critical depth will occur. Therefore, at critical velocity, V_c, the unit discharge in each box culvert must be:

$$q = 0.0311 V_c^3 \qquad \textit{(Equation 7.10)}$$

Substituting V_c = 10.77 ft./sec. into *Equation 7.10* and solving for q yields q = 38.85 cfs/ft. Since the total flow = 2,200 cfs, the combined width of the box culverts should be about = 2200/38.85 = 56.63 feet. Therefore, seven 8-foot-wide box culverts (i.e., total flow width = 56 feet) are selected to assure that $V_c \geq 10.77$ ft./sec.

The height of each box must be equal to or greater than 1.5 Y_c at the entrance to the box-culvert system. For each 8-foot-wide box culvert, it is preferable that the critical depth at each entrance should be approximately equal to, or less than, the critical depth of the approach flow in the main flow path (channel). In this case, for each 8-foot-wide box culvert Y_c = 3.7 feet (close enough to 3.6 feet, which = Y_c in the approach channel) and H = 1.5 Y_c = 5.55 feet. Using *Equation 7.4* with b = 8 feet and H = 5.55 feet, and solving for N yields, $8 = 0.3245\left(\frac{2200}{N(5.55)^{1.5}}\right)$; and thus N = 6.8. Again, this confirms the use of seven 8-foot-wide box culverts.

As noted in the paragraph above, for a 7-cell 8-foot-wide box-culvert system, the calculated critical depth at each culvert entrance = 3.7 feet (again, this depth is only slightly greater than the approach critical depth of 3.6 feet *in the channel*); and H = 1.5 Y_c = 5.55 feet. Since the height, H, exceeds 5.5 feet, for design purposes select the next standard box-culvert height of 6.0 feet.

When placed side-by-side, spacing for the seven box culverts, each 8 feet wide and 6 feet high, will require a total width of 64 feet. Thus, the box-culvert system will be 2.56 times wider than the approaching main flow path, but will be about 7.8 times narrower than the 500-foot-wide effective width of the approach flow in the channel plus the wider right-overbank flood plain. In order to accommodate this condition, three of the culverts should be placed in direct alignment with the 25-foot-wide approach channel, and the remaining four culverts should be placed in the right-overbank flood plain, preferably adjacent to the 3 box culverts for the approach channel.

At a critical depth of 3.7 feet, the entrance flow area at each *channel* box culvert is 29.6 sq. ft. (total flow area of three culverts = 88.8 sq. ft.). Therefore, the entrance velocity at each of the approach-channel culverts is 10.92 ft./sec. This velocity is greater than the approach velocity of 10.77 ft./sec. *in the channel*, and thus the 970 cfs channel flow should be capable of transporting bed-material sediments past the culvert entrances with an additional height of 2.3 feet available in order to accommodate any deposition of bed-material sediments that might occur in the three 8-foot-wide by 6-foot-high box culverts that are in direct alignment with the main flow channel.

In the right-overbank flood plain, for flow to pass through the entrances of the four 8-foot-wide by 6-foot-high box culverts located within the right-overbank, the effective floodplain flow width of 475 feet will contract as it approaches the roadway. However, as the flow depth increases from 0.6 feet to a Y_c of 3.7 feet at the box-culvert entrances, deposition will likely occur in the upstream floodplain; but the preceding calculation demonstrates that the right-overbank bed-material sediments which do arrive at the box culverts would be transported past the culvert entrances.

Using *Equation 7.6*, with a slope of 0.028 ft./ft. the bed-material sediment-transport-capacity calculations yield Q_{sp} = 682.2 cfs for bed-material sediment-transport through the seven 8-foot-wide by 6-foot-high culvert barrels. Using *Equation 5.41*, Q_{sp} = 41.25 cfs for the total bed-material sediments transported in the approaching flood plain (i.e., 29.78 cfs *in the channel*, and 11.47 cfs in the wide, right overbank). Therefore, once the flow passes the culvert entrances, a seven-cell 8-foot-wide by 6-foot-high box-culvert system should be more than capable of transporting the approaching bed-material sediments through its culvert barrels and into the downstream channel.

Using *Equation 7.6* in order to determine the minimum slope that would be required in order to transport 41.25 cfs of approaching bed-material sediments through the seven-cell 8-foot-wide by 6-foot-high culverts yields a minimum calculated slope = 0.0092 ft./ft. This means that placing the seven box culverts on any slope less than 0.0092 ft./ft. would lead to deposition of bed load in the box-culvert barrels. However, as with the previous examples herein, to account for nonuniform flow and additional energy losses associated with high sediment transport, it is required that the minimum slope of any box-culvert system be placed on a slope not less than 1.3 times the minimum calculated slope; which, in this particular case, would be equal to a slope of 1.3 x 0.0092 ft./ft. = 0.0120 ft./ft. Note that the critical slope for the flow of 970 cfs is in the 3 main-channel box culverts = 0.0090 ft./ft., and the critical slope for the flow of 1,230 cfs in the 4 right-overbank box culverts is = 0.0089 ft./ft.; which, in both cases, assumes that n = 0.020 for the culvert due to the resultant low flow velocity that causes bed-load sediment to be in contact with the culvert inverts as the flow moves through each of the box-culvert barrels.

Comparison of the results of the preceding five examples demonstrates that for sediment-laden flows inlet-control conditions dictate that the *"control point" for bed-material sediment transport*

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occurs at the culvert entrance, providing that the slope of the culvert barrel is sufficient enough to convey the bed-material sediment, once it passes the culvert entrance. Thus, when assessing impacts of sediment-laden flows at culverts, and for the appropriate sizing of a culvert system so that it can convey both the flow and the approaching bed-material sediments, entrance geometry is of paramount importance. Once bed-material sediments pass a properly-sized culvert entrance, the capability of a smooth-barrel culvert to transport the approaching bed-material sediments all the way through its barrel usually exists, provided that the interior culvert slope is placed on a steep enough gradient to guarantee adequate bed-material sediment-transport capacity. The constraint of the “control point” being at the culvert entrance will normally be the case for culvert systems subject to inlet-control conditions within the Town, unless true open-channel flow occurs (e.g., supercritical flow within a fully-lined concrete channel); which avoids the occurrence of critical depth at the culvert entrance and everywhere along and within the barrel(s) of the culvert system.

Regarding the promotion of open-channel flow through a box culvert, when and where feasible consideration shall be given to construction of a smooth-lined, sloping, drop-chute inlet immediately upstream of the culvert entrance that is of sufficient length and incline to sufficiently reduce the approach flow depth such that the flow remains well below critical depth and below the culvert soffit; thus entering the culvert barrel at a velocity in excess of the upstream approach-channel flow velocity (i.e., before the drop-chute), accelerating both the flow and the bed-material sediment past the culvert entrance and through the culvert barrel as everywhere supercritical flow. A recommended incline for such sloping drop-chute inlets is 10 feet of longitudinal change for every 1 foot of vertical change (i.e., a 10-percent chute-slope).

The minimum acceptable height for box culverts, concrete arches, or Integra-Arches is 4 feet in order to accommodate maintenance considerations, unless a smaller culvert is approved by the Town. For arch culverts, the 5-foot height shall be measured at the vertical endwalls of the structure, and below the arched portion of the culvert.

7.1.4 Bridges

Due to the significant bed-material sediment-transport issues that exist within the Town, whenever feasible the use of a bridge to pass flows underneath urban infrastructure is preferable to pipe culverts and/or box culverts. Bridges that completely span their respective watercourses minimize disturbances to the geomorphology of the fluvial system, since they can safely pass both floodwaters and bed-material sediments. Whenever a bridge is proposed for construction, the following conditions shall apply:

1. A backwater analysis is required for all proposed bridge structures.
2. The low-chord elevation of any bridge shall be set higher than the hydraulic grade line for the design event in order to account for sedimentation, wave action, and floating surface debris. At a minimum, the low-chord shall be set equal to 1.5 Y_{max}, where Y_{max} equals the maximum depth of flow in the approach channel measured from the thalweg elevation, in feet, during

the design event. Otherwise, if more restrictive, the minimum low-chord elevation shall be established as directed by FEMA criteria (FEMA, 2019).

3. Local scour analysis for piers and abutments is required for all bridge structures. Refer to Section 5.3 of this DCM for guidance regarding local scour.
4. The designer shall determine whether the channel in the vicinity of the proposed bridge is stable, is aggrading, or is degrading; and shall make appropriate adjustments in the design based upon the anticipated design life of the bridge. See Section 5.3.9 of this DCM for guidance on this item.

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Chapter 8. Bank Stabilization

8.1 Criteria

As part of natural channel processes, a channel bank may become unstable and prone to erosion or bank failure. In such instances, the preferred method of treatment within the Town is to establish a minimum setback distance from the adjacent channel bank. The term "setback" is defined as the distance from the bank in question to a line within which no habitable structures may be placed, whether on a full-time or part-time basis. Other treatments are possible, but there are many occasions when these alternatives are not possible. In such cases, the designer should first document the basis of channel instability and then describe the impact that stabilization might have on the fluvial system. While many different factors may lead to instability, often times bank-protection solutions that treat the visible erosion ignore, on a system-wide basis, what is actually causing the erosion problem. For example, while intermittent or so-called "piece-meal" bank stabilization may address local erosion issues, it may inadvertently create bank instability along the opposite side of a channel or other reaches of the fluvial system.

Channel downcutting, while a natural process, is accelerated by systematic changes in a watershed, changes which are usually due to anthropogenic activities. As the channel becomes incised, bank stability is threatened. This leads to further erosion and bank failure, often with accompanying lateral migration of the channel banks. Examples of the agents that may influence erosion include:

1. A migrating headcut from a channel excavation (e.g., sand-and-gravel mining), or the loss of a channel invert control.
2. A channelization project that attempts to straighten, widen, or narrow, a channel or increase the flow velocity through a stream reach.
3. Changes in watershed hydrology and system-wide sediment balance, which lead to more frequent runoff and reduced bed-material sediment supply (e.g., an urbanizing watershed).
4. The trapping of bed-material sediments in reservoirs or other catchments (e.g., stormwater detention facilities).

Also, the natural meandering process of a stream will induce lateral erosion. Over widening a channel often exacerbates lateral erosion, especially at meander bends.

8.2 Bank-Stabilization Considerations

Bank stabilization can be accomplished by various methods, including the use of bio-engineering techniques. The following items should be considered, regardless of the method employed:

1. For all cases, "natural looking" bank-stabilization techniques and designs are the preferred methods that should be utilized, unless an acceptable alternative is approved by the Town. Where an alternative or modification is requested, all substantiating information shall be

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submitted to the Stormwater Utility Manager for initial review. If the Stormwater Utility Manager concurs with the information submitted, it then would be transmitted to the Town for acceptance.

Natural looking bank stabilization shall be defined as:

a. Mechanically-placed or hand-placed rock riprap placed over a geosynthetic filter fabric or an engineered granular filter blanket. The geosynthetic filter fabric shall be completely covered either by the armoring or by embedment.

b. Mechanically-placed or hand-placed rock riprap set in concrete (i.e., rock-faced concrete). The color of the riprap and concrete shall match the natural color of the base channel.

c. Wire-enclosed rock, sometimes referred to as Gabions and/or Reno Mattresses, is an acceptable bank-stabilization material for short-term mitigation purposes (i.e., 20 years, or less).

d. In highly visible areas, plain gunite bank protection shall not be considered “natural looking” unless it is textured and colored to simulate the existing ground.

e. Prior to their implementation, the use of other mechanical, tile, or masonry-block systems shall be submitted to the Town for approval.

2. Both hand-placed and mechanically-placed riprap are acceptable. The minimum blanket thickness for mechanically-placed and hand-placed riprap shall be $2D_{50}$.

3. Stabilization should include adequate wingwall tiebacks at the upstream and downstream limits of the bank stabilization in order to mitigate lateral migration, and adequate toedown depth should be provided to mitigate the impacts of local scour and degradation. The wingwall tieback should be oriented perpendicular to the direction of flow both upstream and downstream. Upstream, the wingwall tieback should have a length that is at least 5 times the channel bank height, but not less than 15 feet; and downstream, the wingwall tieback should not have a length less than 10 feet.

4. Riprap should be sized using the equations found in Section 8.3 of this DCM. When sizing riprap, it is recommended that the channel-bank side-slopes not be steeper than 3H:1V. For direction regarding the sizing of riprap for other bank side-slopes, see the last paragraph of Section 8.3 of this DCM.

5. Treatments that are porous (e.g., riprap, gabions, bio-engineered systems) shall include the use of an engineered granular filter blanket or a geosynthetic filter fabric.

6. For bank-stabilization purposes, design velocities and depths should be checked at all transitions; changes in channel bed slope, or at culverts and bridges.

7. The designer should account for the draining of overland flows into the channel system, especially those that overtop channel banks. Flows that threaten the bank stabilization may require a separate drainage system.

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8. Gunite-type bank stabilization is typically used for the treatment of erosion problems, and therefore it is recommended that it be primarily limited to small drainage swales (generally less than 5 to 10 feet wide, 1 to 2 feet deep, and with velocities less than 8 feet per second); but its use on larger channels may be acceptable with prior written approval from the Town.
9. Soil-cement application. The placement of soil-cement for bank-stabilization purposes should consist of 8-foot-wide to 10-foot-wide stair-stepped lifts; with each lift being 6 inches to 9 inches in thickness, and typically placed on 1H:1V to 2H:1V side-slopes. A soil-cement design should be accompanied with proper strength tests exhibiting that the proposed soil-cement mixture is able to meet the required compressive strength after 7 days. A good source document for the design and construction of soil-cement bank-stabilization is: *Soil-Cement Guide for Water Resources Applications* (Richards & Hadley, 2006).
10. Geotechnical investigations are to be included with all proposed bank-stabilization designs, and are particularly applicable in situations where cut-and-fill slopes exceed 2H:1V; and where cut-and-fill slopes exceed a vertical height of 8 feet. The vertical height of slopes shall not exceed 12 feet.
11. Bank stabilization on only one side of a channel is discouraged, due to concerns with bank erosion and/or lateral migration along the unstabilized bank. If proposed, evidence must be provided to the Town that stabilization on only one bank will not impact the unstabilized bank.

8.3 Riprap Performance Elements

Riprap design requires evaluation of a number of performance elements, such as:

1. <u>Quality of stone</u>. Stone for riprap blankets should be hard, angular, weather-resistant, free from clay or shale, free of seams, cracks, and other structural defects, and have a specific gravity (typical) of not less than 2.65.
2. <u>Dimensions of stone</u>. The longest dimension of the stone should not exceed 1.5 times the mean dimension of the stone, with the least dimension of the stone not less than one-third the greatest dimension of the stone. The minimum median rock size (D_{50}) permissible for use in the Town is 6 inches.
3. <u>Thickness of riprap blanket and composition</u>. The riprap veneer should have a minimum blanket thickness of 12 inches, and should be underlain with an engineered granular filter blanket or an approved geosynthetic filter fabric.
4. <u>Channel Hydraulics</u>. When using riprap for bank stabilization, it is recommended that the average velocity of flow in the channel not exceed a specified maximum, dependent upon channel curvature (see Section 8.6, below).
5. <u>Site conditions</u>. Areas on which bank protection is to be constructed shall be cleared, grubbed, and excavated or backfilled in a manner which shall produce a ground surface in

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reasonable conformance with the lines and grades shown on the project plans, or as established by the Design Engineer. Placement of riprap through water shall not be permitted.

6. Watercourse geometry. The cross-section and line and grade of the riprapped channel shall more or less conform to the cross-section and line and grade of the pre-riprapped channel.

8.4 Riprap Sizing

Along straight channel reaches, sizing for mechanically-placed or hand-placed riprap is to be based upon the following equation for calculating the D_{50} particle size, *in inches*:

$$D_{50} = 0.15V^2 \qquad \textit{(Equation 8.1)}$$

Along reaches with in-channel low-flow-thalweg entrenchment, 1 foot to 2 feet in depth, sizing for mechanically-placed or hand-placed riprap is to be based upon the following equation for calculating the D_{50} particle size, *in inches*:

$$D_{50} = 0.24V^2 \qquad \textit{(Equation 8.2)}$$

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 component 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 Oro Valley 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 (see COTDSM, Section 6.6.3 [SLA, 1989/1998]).

Along channel reaches where curvature exists (i.e., bends), sizing for mechanically-placed or hand-placed riprap should be based upon the following equation for calculating the D_{50} particle size, *in inches*:

$$D_{50} = 1.14V^2 \left[\frac{sin^2(\frac{\alpha}{2})}{cos(\alpha)}\right]^{0.4343} \qquad \textit{(Equation 8.3)}$$

Where, in the preceding equations,

V = the average flow velocity in the channel, in ft./sec.; and

α = angle formed by projection of the channel centerline from the beginning point of curvature to a point meeting the line tangent to the outer bank (see Figure 5-1 of this DCM).

Equation 8.1 should be used for all straight reaches with no in-channel low-flow-thalweg meandering, and whenever $\alpha \leq 11$ degrees. *Equation 8.2* should be used for all straight channel reaches where in-channel low-flow-thalweg entrenchment exists between the primary channel banks. *Equation 8.3* should be used along all channel reaches where curvature exists (i.e., along bends whenever $\alpha \geq 11$ degrees); except that the maximum value for α should not exceed 60 degrees. Note however, if in-channel low-flow-thalweg entrenchment exists along a channel reach where mild curvature also exists (i.e., the bend-angle $\alpha < 18$ degrees), the minimum median rock size for loose riprap should not be less than $D_{50} = 0.24V^2$, where D_{50} is in inches.

Equation 8.1, *Equation 8.2*, and *Equation 8.3* are each based on an assumed unit weight of 165.36 lbs./ft.^{3} for rock, and on 3H:1V channel-bank side-slopes. If conditions are otherwise, then the "default" D_{50} size for riprap should be decreased or increased dependent upon whether the unit weight of rock is greater than or less than 165.36 lbs./ft.^{3}, and dependent upon whether the channel side-slopes are steeper than or milder than 3H:1V (note: channel side-slopes should not be less than 4H:1V or greater than 2H:1V). For example, if the unit weight of the rock were 156 lbs./ft.^{3}, and the channel side-slopes were steepened to 2H:1V, the results using *Equation 8.1*, *Equation 8.2*, and *Equation 8.3* would each have to be multiplied by a factor of 1.32 in order to establish the appropriate D_{50} rock size. The general equation for applying a D_{50} size correction, D_{50}^*, to account for differences in unit weight and side-slopes is:

$$D_{50}^* = \frac{D_{50}}{\left(\frac{\gamma_S}{165.36}\right)^{2.0565}\left(\frac{\cos\varphi}{0.9487}\right)^{2.667}} \qquad \textit{(Equation 8.4)}$$

Where,

D_{50}^*	=	the median rock size, in inches,
γ_s	=	the unit weight of the riprap, in lbs./cu. ft., and
$\cos\varphi$	=	the angle of the embankment side-slopes, in degrees.

The maximum allowable side-slope for loose riprap embankments is 2H:1V. The minimum D_{50} rock size permissible for use in the Town is 6 inches.

The recommended side-slope for a loose-riprap embankment is 3H:1V. The maximum allowable side-slope for a loose-riprap embankment is 2H:1V, unless a site-specific geotechnical report is more restrictive, and recommends a milder slope than 2H:1V. Note that, as stated previously, *Equation 8.1*, *Equation 8.2*, and *Equation 8.3* must be adjusted using *Equation 8.4*, when applicable, in order to account for a steeper side-slope than 3H:1V.

Equation 8.1 through *Equation 8.4* were derived based upon data published in the National Cooperative Highway Research Program document titled *NCHRP Report 568, Riprap Design Criteria, Recommended Specifications, and Quality Control, Transportation Research Board* (Lagasse et al., 2006).

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8.5 Riprap Gradation and Blanket Thickness

The gradation of the riprap in the mechanically-placed or hand-placed protective blanket should be as follows, as directed by the Town:

Table 8-1. Recommended Riprap Gradation*

Percent Finer	Sieve Opening (% of D_{50})
100	$2.00D_{50}$
70	$1.60D_{50}$
50	$1.00D_{50}$
20	$0.50D_{50}$

The minimum thickness of a mechanically-placed or hand-placed protective riprap blanket should be $2D_{50}$, and the riprap blanket should not be less than 12 inches in thickness for practical placement.

The stones should be reasonably well-graded throughout the riprap-blanket thickness, and shall meet or exceed the specifications contained in Section 913-2.01 of the updated 2015 PAG *Standard Specifications and Details for Public Improvements* (PAG, 2015).

8.6 Velocity Considerations

Equation 8.1, *Equation 8.2*, and *Equation 8.3* each suggest that the size requirements for mechanically-placed or hand-placed riprap will typically require D_{50} sizes that are likely to be too large for ready availability and/or will be too large for economical installation within the Town when average channel flow velocities exceed 8.5 ft./sec. using *Equation 8.2*; or when average channel flow velocities exceed 11.0 ft./sec. using *Equation 8.1*. When using *Equation 8.3*, even smaller average channel flow velocities (i.e., < 8.5 ft./sec) should not be exceeded when the bank-angle α exceeds 20 degrees. Accordingly, the designer should consider using a means of bank stabilization other than mechanically-placed or hand-placed riprap (e.g., rock-faced concrete, soil cement, concrete, etc.) when computed average channel flow velocities are considered to be "high" for the hydraulic and geomorphic conditions that exist within the channel reach under investigation.

8.7 Wire-Enclosed Rock

Wire-enclosed-rock revetments (e.g., gabions) consist of rectangular wire-mesh baskets filled with rock. The most common types of wire-enclosed revetments are mattresses and stacked blocks constructed from commercially available wire units. It is recommended that the design of wire-enclosed-rock revetments follow the procedures given in Sec. 7.2 of the Federal Highway Administration's *Hydraulic Engineering Circular No. 15, Third Edition, Design of Roadside Channels with Flexible Linings* (FHWA, 2005). Further, the materials used to construct wire-enclosed-rock revetments shall meet or exceed the specifications contained in Section 913 of the updated 2015 PAG *Standard Specifications and Details for Public Improvements* (PAG, 2015).

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The minimum rock size in a gabion basket must be larger than the opening size of the wire matrix that comprises the gabion. Also, when wire-enclosed-rock revetments are stacked, filter fabric or granular backfill is needed in order to prevent leaching or piping of soil through the revetments.

8.8 Rock-Faced Concrete

Revetment comprised of rock-faced concrete consists of concrete bank protection with rocks set into its top surface to form a ridged, monolithic armor with a more natural appearance. This type of armoring can be placed within and along the channel banks and streambed, or along just the channel banks alone. The hydraulic design of rock-faced concrete channels is described in Chapter 4 of the DCM; and the design of scour protection, such as toe downs and grade-control structures, is described in Chapter 5 of the DCM. The materials used to construct bank protection comprised of rock-faced concrete shall meet or exceed specifications in Section 912 and Section 913 of the updated 2015 *PAG Standard Specifications and Details for Public Improvements* (PAG, 2015).

8.9 Concrete Bank Protection

Concrete is often used for bank protection when flow velocities are high, or when it is necessary to maximize usable land area on a site. Concrete lining of the entire channel (i.e., both channel banks and bottom) is usually required for very high flow velocities and steep channel gradients. However, an earthen bottom and concrete-lined banks, with appropriately spaced grade-control structures, is a stabilization measure that can be employed within the Town along channels having milder gradients. Typical construction details should be provided for both partial and full channel lining. Details for wingwall tiebacks should be in accordance with the guidelines found in Section 8.5.4 of the COTDSM. Also, see Section 8.5.4 of the COTDSM for key-in details (SLA, 1989/1998).

8.10 Energy Dissipators

The term Energy Dissipator encompasses a wide variety of hydraulic structures that are intended to dissipate the kinetic energy of flowing water. It usually becomes necessary to dissipate this energy when flow velocities become high enough to likely cause excessive erosion or damage to channels and hydraulic structures. Unacceptably high flow velocities generally occur at locations where the energy slope of the flow becomes very steep. Examples include drop structures, spillways, drop-inlets, constrictions at culverts and bridges, etc. The type of energy dissipator that is appropriate for a particular installation will depend upon a number of factors, including hydraulics, economics, potential damage to structures, surrounding improvements, and the environment. The Design Engineer should refer to Chapter IX, Section 9.4, of the COTDSM (SLA, 1989/1998) for additional guidance regarding the use of energy dissipators.

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The following sections provide some suggested guidelines for use in the analysis and design of various types of energy-dissipator structures.

8.10.1 Culvert Outlets

Energy dissipators are frequently needed at culvert outlets because culverts typically concentrate flow and increase flow velocities. Concentrated, high-velocity flow is erosive, and scour holes will form at culvert outlets unless protective measures are taken. The potential size of a scour hole can be determined according to the procedures described in Chapter 5 of this DCM and in Chapter VI of the COTDSM (SLA, 1989/1998). Should energy dissipation be necessary at a culvert outlet, there are several designs that could be effectively used. Detailed procedures for the design of such energy-dissipating devices can be found in publications of the USBR (Peterka, 1984) and the FHWA (2006).

8.10.2 Channel Outlets

Where a narrow, lined channel ends at an unlined or natural channel, energy dissipation may be needed in order to prevent erosion typically caused by high-velocity concentrated flow. As with culvert outlets, there are many types of energy dissipators that can be used. Again, detailed procedures for the design of such energy-dissipating devices can be found in publications of the USBR (Peterka, 1984) and the FHWA (2006).

8.10.3 Channel Drops

Channel drops occur at locations along a channel where the bed of a channel makes an abrupt drop over a very short distance. Headcuts are examples of channel drops in unlined channels. Protection against the energy dissipation of low channel drops often can be provided by merely increasing the toedown along the bank protection and at the drop structure, as is done with most grade-control structures. Higher drops typically require energy-dissipation structures.

Energy-dissipation structures at channel drops normally take the form of chutes with baffle blocks or stilling basins (Peterka, 1984; FHWA, 2006). Examples of energy dissipators used at channel drops can be found within the metropolitan Pima County region on the Kinnison Wash at Lakeside Park; on the Pantano Wash at Broadway Boulevard; and on the Airport Wash, west of Interstate 19.

8.10.4 Seepage Forces

Seepage forces occur whenever there is interflow through the bed and banks of a channel formed in permeable alluvium. The interflow occurs through the interface between the water and the channel wall, and the severity of this interflow depends upon both the difference in pressure across the interface and the permeability of the bed material. Under such circumstances, seepage forces act to reduce the effective weight and stability of the bed and bank materials. Historically,

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in the region encompassing the Town the primary cause of bank protection has been due to excessive seepage forces.

Seepage forces may create an upward hydrostatic pressure on structures (uplift). The magnitude and distribution of seepage forces in a foundation, as well as the amount of underseepage for a given coefficient of permeability, can be obtained utilizing a coefficient from a flow net. The Weighted-Creep Theory, as developed by Lane (1935), is suggested as a means for designing hydraulic structures on pervious foundations so that they will be safe from uplift pressures and piping. Lane's theory defines the weighted-creep ratio as:

$$C_{rw} = \frac{\Sigma L_H + 3\Sigma L_V}{3H_s} \qquad \textit{(Equation 8.5)}$$

Where,

C_{rw}	=	Weighted creep ratio.
L_H	=	Horizontal, or flat, contact distance (less than 45°), in feet.
L_V	=	Vertical, or steep, contact distance (more than 45°), in feet.
H_s	=	Head on structure (headwater - tailwater), in feet.
Σ	=	Summation of individual L_H and L_V values, dimensionless.

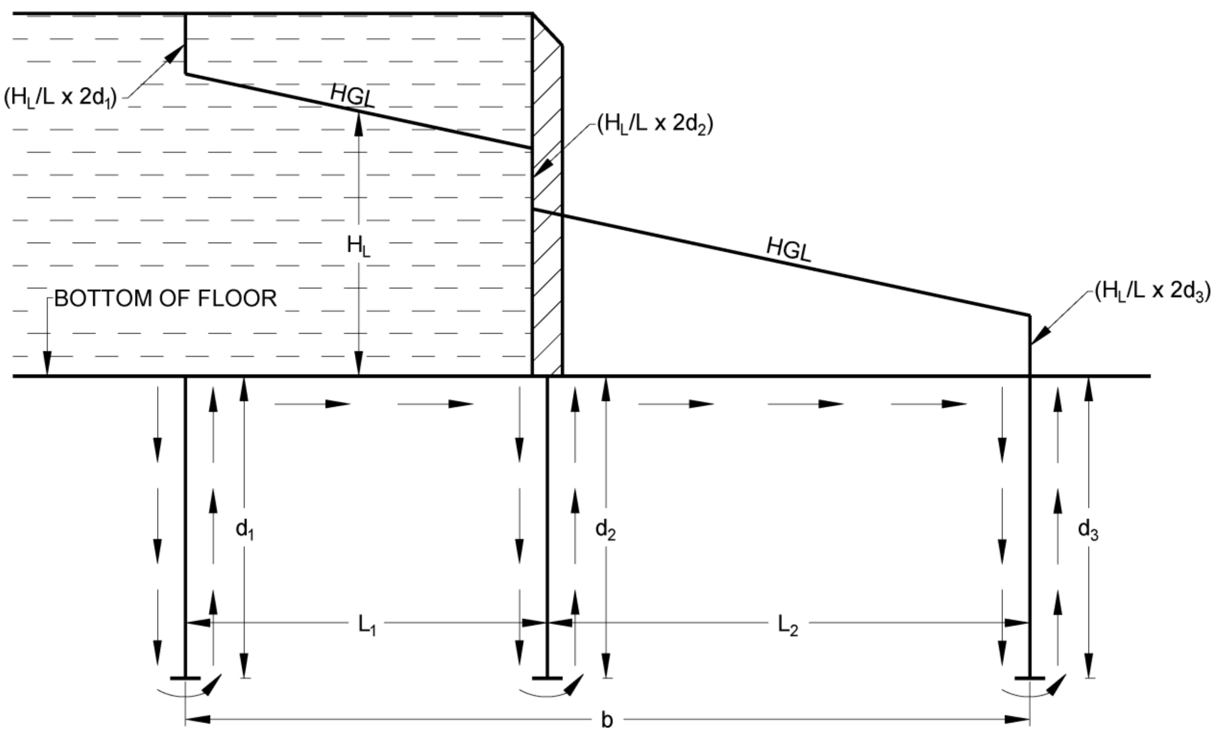

Figure 8-1. Creep Length (along Arrows) for Hydraulic Structure on Pervious Foundation

From Figure 8-1, the Total Creep Length (TCL), in feet, is given by:

$$TCL = (d_1 + d_1) + \frac{1}{3}L_1 + (d_2 + d_2) + \frac{1}{3}L_2 + (d_3 + d_3);$$

$$TCL = \frac{1}{3}(L_1 + L_2) + 2(d_1 + d_2 + d_3)\text{; therefore,}$$

$$TCL = \frac{1}{3}b + 2(d_1 + d_2 + d_3) \qquad \textit{(Equation 8.6)}$$

Thus, in *Equation 8.5*, $\Sigma L_H = b$ and $\Sigma L_V = 2(d_1 + d_2 + d_3)$.

Table 8-2 lists Lane's recommended weighted-creep ratios for various foundation materials.

Table 8-2. Weighted-Creep Ratios

Material	C_{rw}
Very fine sand and silt	8.5
Fine sand	7.0
Medium sand	6.0
Coarse sand	5.0
Fine gravel	4.0
Medium gravel	3.5
Coarse gravel, including cobbles	3.0
Boulders, with some cobbles and gravel	2.5
Soft clay	3.0
Medium clay	2.0
Hard clay	1.8
Very hard clay or hardpan	1.6

Piping under the foundation of a structure occurs when the upward seepage force at the downstream toe of the structure exceeds the submerged weight of material. The underlying soil is “flooded out,” and the erosion progresses backwards along the seepage flowline until a "pipe" is formed, allowing rapid flow under the foundation and subsequent failure of the structure.

Cutoff walls, aprons, and drains are generally installed to control the amount of seepage under the structure, and to limit the intensity of the uplift so that the stability of the structure will not be threatened. In addition to the weighted-creep ratio presented here, the Engineer should also perform other structural calculations, as appropriate, to ensure stability of proposed structures.

8.11 Standard Specifications

Standard Specifications for bank-stabilization techniques should be as specified in the updated 2015 PAG *Standards Specifications and Details for Public Improvements* (PAG, 2015). In particular,

Section 913 — Bank Protection.

Section 920 — Soil-Cement for Bank Protection, Channel Lining and Grade-Control Structures.

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Chapter 9. Subdivision (Local) Street Drainage

9.1 Introduction

This section is intended as serve as a supplement to existing street standards that can be found in the Town's *Subdivision Street Standards* (2004), the Town's *Zoning Code* (2019), and the Town's *Floodplain and Erosion Hazard Management Ordinance* (2020). In the case where this DCM or other standards conflict with a Town Code or Ordinance, the Town Code or Ordinance shall have supremacy, and shall apply.

9.2 Criteria

The criteria in Chapter 9 are intended to be used for evaluating the allowable encroachment of stormwater runoff within public and private streets, driveways, and Parking Area Access Lanes (PAAL) located within the Town.

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Table 9-1. Street Drainage Criteria (for Flow within and Parallel to the Street Cross-Section)

Policy	Parameter	Standard	Comment
Street **WITH**: Curb and Gutter	Peak Discharge for the 1% AEP Flood	Street Drainage, Curb-to-Curb, Shall NOT Exceed 50 cfs. Maximum Allowable Depth Shall Not Exceed 6 Inches above the Pavement Surface at the Curb Line. All Runoff Is to Be Contained within the Curbs.	Refer to Town Subdivision Street Standards, as Applicable
	Peak Discharge for the 10% AEP Flood	The Runoff Must Be Contained within Street Curbs. For Collector and Arterial Streets, at Least One 12-Foot-Wide "Dry" Driving Lane Must Be Maintained in Each Direction.	Refer to Town Subdivision Street Standards, as Applicable
Street **WITHOUT**: Curb and Gutter	Peak Discharge for the 1% AEP Flood	Street Drainage Shall NOT Exceed 50 cfs. Maximum allowable depth Shall Not Exceed 6 Inches above the Pavement Surface. Runoff Is to Be Confined to the Road Right-of-Way or to Dedicated Drainage Easements.	Refer to Town Subdivision Street Standards, as Applicable
	Peak Discharge for the 10% AEP Flood	WSEL below the Roadway Pavement Subgrade for Runoff Contained within Roadside Channels/Ditches.	Refer to Town Subdivision Street Standards, as Applicable
Street **WITH**: Storm-Drain System	Peak Discharge for the 1% AEP Flood	Catch Basins, Scuppers, etc., Are to Be Provided, as Necessary, to Remove Runoff so Flow Does Not Exceed the Maximum Allowable Depth of 6 Inches.	Refer to Town Subdivision Street Standards, as Applicable
	Peak Discharge for the 10% AEP Flood	Dry-Lane Accessibility – Roadside Channels Are to Be Supplemented with Pipes to Accommodate Flows that Exceed the 10% AEP Runoff Street Capacity.	Refer to Town Subdivision Street Standards, as Applicable
Dip Crossings	Depth, Velocity, and Duration of Flow	Per Subsection 9.4.3 of this DCM.	See Subsection 9.4.3

9.3 Street Drainage

Streets, driveways, and PAALs should not to be used as drainage corridors, especially for the conveyance of offsite flows. However, it is understood that nuisance flows associated with runoff and adjacent developments may drain into the streets, driveways and PAALs, for short-term conveyance to stormwater facilities. Streets with inverted crowns are not permitted, unless approved by the Town.

9.4 All-Weather Crossings, Combined Crossings, and At-Grade Dip Sections

9.4.1 All-Weather Crossings

At locations where all-weather crossings are required, flows greater than 50 cfs running transverse to the street or driveway shall be contained in a culvert, or a combined crossing, capable of conveying stormwater runoff from a 1% AEP flood event. During a 1% AEP flood event, stormwater runoff overtopping the road shall not exceed 6 inches in depth. All-weather crossings of streets and driveways will be required to adhere to the stipulations set within the Town *Floodplain and Erosion Hazard Management Ordinance* (2020).

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At least one all-weather vehicular access shall be provided for every habitable structure located within the Town. This includes, but is not limited to, all commercial, industrial, municipal, institutional (school, church, organization, etc.) buildings, as well as all residential structures (single and multi-family dwellings). In all situations where private vehicular access crosses a FEMA regulatory floodplain located between the points where the private access leaves a paved, publicly maintained roadway and the end of the private access, the owner of the property requiring the private vehicular access shall:

1. Construct at least one private vehicular access in such a manner that it is permanent and is located over terrain that can be traversed by conventional motor vehicles during a base (i.e., 1% AEP) flood.
2. For all other points of access, execute and record a covenant, running with the land and enforceable by the Town, which contains the following:
    a. An acknowledgment that the private vehicular access may be impassable to conventional motor vehicles and emergency vehicles in times of flooding.
    b. A hold-harmless provision, holding the Town harmless from and against all injuries and damages resulting from traversing or attempting to traverse the private vehicular access during times of flooding.
    c. A provision which either:
        i. Requires the covenant or successors and assigns to erect and maintain a sign(s) in a location(s) and of a size(s) acceptable to the Town, stating "DO NOT ENTER WHEN FLOODED"; or,
        ii. Causes the covenant or successors and assigns to assume responsibility to notify users of the private vehicular access that it may be impassable in times of flooding, and agree to indemnify and defend the Town, its officers, employees, servants and agents, against all claims for injuries to persons or damages to property due to the construction, installation, location, operation, safeguarding, maintenance, repair, and condition of the private vehicular access.
3. Maintenance and repair of driveway culverts
    a. Non-public (i.e., private) driveway culverts located within the public Right-of-Way (ROW) are the responsibility of the owners. Driveway culverts are to be maintained by the builder/user of the driveway. Note that the Town reserves the right to ask for removal or modification of any driveway culvert, if conditions are warranted.
    b. Existing driveway culverts may be replaced only with culverts that are designed to meet flow-capacity standards as specified in this DCM.
    c. Existing driveway culverts may be replaced with 12-inch-diameter culverts, or larger, provided that Town flow-capacity criterion has been met, which includes assessment of

the capacity of the pipe to adequately transport all incoming bed-material sediments unless flow is coming almost entirely from sediment-free impervious surfaces, such as commercial areas, paved streets, and parking lots.

9.4.2 Combined Crossings

A combined crossing conveys drainage/runoff via a combination of culvert(s) and a sag vertical curve located at a street/channel intersection. The combined crossing is intended to pass minor, more frequent flows entirely beneath the street; while passing major, less-frequent, flows over and beneath the roadway surface, (i.e., combined culvert(s)/dip crossing). The combined crossing shall be designed such that maximum depth of flow over the dip-crossing pavement surface (i.e., the "over-the-roadway" flow) does not exceed 6 inches for the 1% AEP flood event. The design and construction of combined crossings requires prior written approval by the Town.

9.4.3 At-Grade (Dip) Sections

Out of concern for sight visibility and maintenance, at-grade (dip) crossings are to be avoided whenever possible on public streets located within the Town, and are discouraged on private streets. At-grade crossings on public and private streets will only be allowed where the combination of the depth of flow, Y, and the velocity head, $\frac{V^2}{2g}$ (i.e., $Y + \frac{V^2}{2g}$), does not exceed the numerical value of 1.30 for a duration in excess of 30 minutes. Even in such cases, prior written approval from the Town is required. All at-grade crossings shall be designed to be self-cleaning, which means that the minimum pavement cross-slope at the at-grade crossing should be four percent (4%), sloping from the upstream to the downstream edge of pavement. In general, requests for an at-grade crossing shall address and justify, in writing, each and all of the following items, and shall be sealed by an Arizona-Registered Professional Civil Engineer.

1. Demonstrate that flows are infrequent and insignificant—generally less than 10 cfs during a 50% AEP flood event, and less than 50 cfs in a 1% AEP flood event, with a maximum flow depth of 6 inches in the at-grade crossing that does not exceed a duration of 30 minutes.
2. Demonstrate that the proposed at-grade crossing is not limiting all-weather access to habitable structures or critical facilities.
3. Demonstrate that, in the future, the Town will not incur additional street maintenance and costs for sediment removal or repair of pavements at the at-grade crossing. If this requirement cannot be adequately demonstrated, a covenant shall be recorded requiring that such additional street maintenance and costs be borne by the original owner(s).
4. Demonstrate that hazardous conditions downstream and within the at-grade crossing will not result either during or immediately following a flow event.
5. Demonstrate that appropriate signage will be placed at the at-grade crossing.

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6. Demonstrate that not using an at-grade-crossing is poor engineering design, creates a dry-weather hazard, and creates a flood and/or erosion hazard to existing properties.
7. All at-grade crossings shall be designed to be self-cleaning. A pavement cross-slope at the at-grade roadway section shall be constructed with a minimum cross slope of 4%, sloping from the upstream to the downstream edge of pavement, unless it can be demonstrated that traffic-safety controls would dictate otherwise. If an at-grade crossing is approved by the Town, an upstream sediment trap or other means of keeping the dip crossing clear of sediment/debris may be acceptable in lieu of a 4% pavement cross-slope. However, in order to waive the 4% pavement cross-slope requirement, it must be clearly demonstrated that an upstream sediment trap is a workable solution. This is so because unless constantly maintained an upstream sediment trap likely will fill (perhaps rapidly during a large flow event, unless constructed to be very large); and thus subsequent incoming sediments will simply pass through the sediment "trap" and be deposited in the dip section. Also, if larger amounts of incoming sediments are captured in the sediment trap, a "clear-water" condition would likely be created immediately downstream of the dip section, thus exacerbating scour/erosion at the downstream edge of the roadway and within the watercourse segmented located immediately downstream. These types of sediment issues must be addressed, and approved in writing by the Town, at the time of project design of an at-grade crossing.
8. Other criteria, as specified by the Town.

Dip sections in which the transverse flow is less than or equal to 50 cfs in a 1% AEP flood event shall be fitted with at least 6-inch-wide by 12-inch-deep concrete headers. Where flows of greater than 50 cfs in the 1% AEP flood event occur, cutoff walls shall be installed. Cutoff walls are required in order to maintain the integrity of the roadway pavement during flooding. The required depth of a cutoff wall shall be designed to be 1-foot deeper than the calculated scour depth determined by use of the local scour equations contained in this DCM. However, in no case shall the cutoff wall be less than 2 feet in depth upstream and 4 feet in depth downstream of the dip crossing. Sliding and overturning moments may need to be analyzed for dip sections on all-weather access streets that are protected by cutoff walls. Cutoff walls shall be placed 6 feet outward from the upstream and downstream roadway edges of the pavement line, and shall be placed to the edge of the roadway shoulder on the downstream side. The pavement in the at-grade crossing shall be widened to the upstream and downstream cutoff walls in order to create an adequate vehicular recovery zone. Concrete headers and cutoff walls must be extended, at a minimum, to the limits of the 1% AEP floodplain limits, unless directed otherwise by the Town.

9.5 Curb Openings

Drainage conveyed within streets, driveways, and PAALs while being delivered to channels or outlets may be disposed of through curb openings, preferably not greater than 10-feet in length (bottom width). Curb openings in excess of five feet in bottom width shall be fitted with a bollard

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centered within the opening. All curb openings shall be fitted with outlet aprons and approved scour protection. Should the hydraulic design require greater conveyance than can be provided by a curb opening, approval from the Town is required for other structure options (e.g., grate inlets). Along and within sidewalk areas, no at-grade curb openings are allowed without approval of the Town. The designer shall consider pedestrian needs and other related requirements by providing scuppers under sidewalks, where necessary.

9.6 Curb Inlets and Grate Inlets

Guidelines for design of curb inlets and grate inlets that collect and discharge street drainage into outfall conveyance systems for safe disposal can be found in Chapter X of the COTDSM, which is appended to this DCM for the convenience of the user.

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References

Oro Valley Town (2004). *Town of Oro Valley Subdivision Street Standards and Policies Manual*. 2nd Edition.

Oro Valley Town Code Chapter 17 Floodplain and Erosion Hazard Management (2020). https://orovalley.town.codes/TC/17

Oro Valley Zoning Code (2019). https://orovalley.town.codes/ZC

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Chapter 10. Safety Factors

In accordance with the standards, accepted methodologies, and design references contained in this DCM, specific factors-of-safety are to be applied to the design of stormwater systems as discussed in the following sections:

10.1 Pressure Flow in Culverts:

Cross-drainage culverts shall not be designed to operate under pressure flow, particularly at culvert entrances. This requirement shall not be applicable to storm-drain systems that are free of bed-material, sediment-laden flows.

Cross-drainage culvert sizing shall be based upon conveyance of both water and bed-material sediment using the procedures contained in Chapter 7 of this DCM. The procedure presented in Chapter 7 of this DCM incorporates assessment of sedimentation potential at the culvert entrance, as well as through the culvert barrel, and requires adjustment to sizing of the culvert on the basis of the results of such an assessment.

When and where feasible, consideration shall be given to the construction of a smooth-lined, sloping, drop-chute inlet immediately upstream of the culvert entrance. The drop-chute shall be of sufficient length and slope to reduce approach flow depth in order for the flow to remain below the culvert soffit, thus accelerating both the flow and the bed-material sediments past the culvert entrance and through the culvert barrel as non-backwater, supercritical flow. A preferred incline for such a sloping drop-chute inlet is 10H:1V (i.e., a 10-percent chute-slope). Within the Town, all drop-chute slopes shall have inclines between 15H:1V, minimum; and 5H:1V, maximum.

For all other conditions, *in the absence of sediment-laden flow*, at a minimum the culvert size should be increased by one culvert size, or increased by one additional barrel, when the flow depth (d), in feet, is greater than or equal to 0.8 times the culvert diameter (D), in feet; or, when flow depth (d) is greater than or equal to the culvert height (H), in feet, for non-circular culverts (e.g., arch or elliptical culverts).

10.2 Catch Basin and Other Inlet Factors of Safety

As specified in Chapter X, Section 10.6.9, of the COTDSM (SLA, 1989/1998), and in this DCM.

10.3 Clogging Factor of Safety

As specified in Chapter X, Section 10.6.9, of the COTDSM (SLA, 1989/1998), and in this DCM.

10.4 Open Channels

The total channel depth shall include the required freeboard, plus calculated depth of bed-material sediment deposition (if aggradation is predicted). Design freeboard for channel banks,

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along with the design freeboard for bridges over alluvial channels, is to be based upon the methodology contained in this DCM.

10.5 Stormwater Detention Facilities

In order to account for possible piggyback storm events, the containment volumes of stormwater detention facilities should be sized to be equal to 1.5 times the calculated runoff from a 1% AEP storm (i.e., a so-called 100-year design storm), as well as the calculated bulked volume of bed-material sediment deposition from a 10% AEP storm (i.e., 10-year storm).

10.6 Clogging Factors for Storm-Drain Systems

The following guidelines, taken from the COTDSM (SLA, 1989/1998) should be followed to provide an appropriate factor of safety against the clogging of the various appurtenances that comprise a storm-drain system.

1. GRATES AND SLOTTED DRAINS
    a. Sump Conditions:
        i. Orifice Flow: required area = 2.0 x calculated area, in sq. ft.
        ii. Weir Flow: required perimeter = 2.0 x wetted perimeter, in feet.
    b. Continuous-Grade Conditions
        i. Required length of opening = 2.0 x calculated length, in feet.
2. CURB INLETS
    a. Sump Conditions
        i. Required length of opening = 1.50 x calculated length, in feet.
    b. Continuous-Grade Conditions
        i. Required length of opening = 1.25 x calculated length, in feet.
3. COMBINATION GRATE AND CURB INLET
    a. Sump Conditions
        i. Orifice Flow:

            Required area = 2.0 x calculated area for grate, in sq. ft.

            Required length = 1.25 x calculated length for curb inlet, in feet.
        ii. Weir Flow:

            Required perimeter = 1.0 x calculated perimeter for grate, in feet.

            Required length = 1.25 x calculated length for curb inlet, in feet.

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b. Continuous-Grade Conditions

i. Required length of opening = 1.0 x calculated length for grate, in feet.

Required length = 1.25 x calculated length for curb inlet, in feet.

4. ARIZONA DEPARTMENT OF TRANSPORTATION (ADOT) STANDARD TYPE-3 CATCH BASINS

a. Continuous-Grade Conditions

i. Curb-inlet length upstream of catch basin = 1.25 x calculated length, in feet.

ii. Required length of grate = 1.0 x calculated length, in feet.

These general guidelines for clogging should be applied in cases where more-detailed information about clogging for a specific grate type is unavailable. Grates that are longer than needed in order to intercept 100 percent of frontal flow will have greater debris-handling efficiencies than will shorter grates.

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References

Simons, Li & Associates, Inc. (SLA). (1998). *Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona*. (Original work published 1989.)

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11. STORMWATER QUALITY

11.1 Introduction

The Town of Oro Valley manages the quality of stormwater as mandated by the Federal Clean Water Act (33 U.S.C. 1251 et seq.), National Pollutant Discharge Elimination System Regulations (40 CFR Part 122), and State regulations for stormwater discharge (ARS Title 49, Chapter 2, Article 3.1) to help ensure that all surface water is free from pollutants in amounts or combinations that:

- Settle to form bottom deposits that inhibit or prohibit the habitation, growth, or propagation of aquatic life or that impair recreation uses (bottom deposits standard);
- Cause objectionable odor in the area in which the surface water is located;
- Are toxic to humans, animals, plants or other organisms, (toxics standard);
- Change the color of the surface water from the natural background levels of color; or,
- Are free from toxic pollutants and/or pollutants in toxic amounts.

In addition, the Town must also comply with the regulatory requirements of the Arizona Pollutant Discharge Elimination System (AZPDES) General Permit AZG2016-002, and the Construction General Permit AZG2020-001, or as otherwise amended.

In part, these rules require "owners/operators of construction activities, new or redeveloped land, and industrial and commercial facilities to minimize the discharge of pollutants to the Municipal Separate Storm Sewer System (MS4) through the installation, implementation, and maintenance of stormwater control measures." The Town enforces these rules by first reviewing Grading Plans, Site Plans, Stormwater Pollution Prevention Plans (SWPPPs), Erosion and Sediment Control Plans, and related construction documents, and then afterwards by conducting regular site inspections during and after construction.

A typical management approachused by the Town and many municipalities, is to reduce the discharge of objectionable pollutants by requiring all new commercial developments to capture and filter, or otherwise mitigate, the first one-half inch of runoff coming from paved parking areas, often referred to as the "first flush." This is done because the first flush often contains a higher pollutant load compared to the same water volume captured at later periods of the same storm.

In the context of this manual, *First-Flush* means a collection system approved by the Town that is employed to capture and isolate the first one-half (1/2) inch runoff from the commercial development site.

All new commercial sites with vehicular parking and access lanes (PAALs) must be designed to provide a combination of onsite structures and long-term housekeeping procedures to prevent pollutant discharges. Among the options available to the owner and their engineer are:

- Onsite retention storage and infiltration (such as water harvesting and shallow detention basins);
- Stormwater filtration and treatment;
- Good housekeeping and periodic pavement sweeping; or,
- Combination thereof.

11.2 Criteria for Preventing Construction Site Pollution Discharges

In compliance with ADEQ's Construction General Permit AZG2020-001 CGP, a Stormwater Pollution Prevention Plan, or SWPPP, shall be submitted with all grading-permit applications for new construction activities that will disturb one or more acres of land, or will disturb less than one acre, but is part of a common plan of development or sale that will ultimately disturb one acre or more. SWPPPs are not required for routine maintenance performed to maintain the original line and grade, hydraulic capacity, or original purpose of the construction site (without acreage limitations). However, corrective action or site modifications larger than one acre do require SWPPPs.

The Stormwater Pollution Prevention Plan is to be created using the SWPPP template available from ADEQ and formatted to fit 8½" x 11" letter-size paper, and is to conform to the requirements in Section 6.3 (SWPPP Contents) of ADEQ's Construction General Permit. Hereinafter, this narrative document is referred to as a SWPPP Notebook, and is to generally include a description of the planned construction activities, potential sources of pollution, best management practices to be implemented, good housekeeping practices, inspection procedures, and recordkeeping.

The SWPPP Notebook is to be accompanied by Erosion and Sediment Control Plan (also referred to by ADEQ as a site map) and is to conform to the requirements in Section 6.3.5 (Site Map) of ADEQ's Construction General Permit. In addition to showing BMPs to be used during construction, and to assist the Town in performing requisite post-construction and annual compliance inspection, the Erosion and Sediment Control Plan shall include a separate sheet highlighting post-construction BMPs, such as permanent erosion control, hillslope vegetation, and first-flush filtration devices.

To facilitate review, and to help the operator ensure the SWPPP contains all required components, the SWPPP Notebook and Erosion and Sediment Control Plan submitted to the Town shall be accompanied by a copy of the ADEQ checklist completed by the engineer of record.

A copy of the Notice of Intent (NOI) submitted by the owner/applicant to ADEQ shall be given to the Stormwater Utility prior to commencing construction or requesting inspections by the Town.

Drainage reports for the project will include a brief reference, or acknowledgement, of the SWPPP, as well as include any engineering calculations needed in order to specify a particular erosion-control method, such as rock riprap, first-flush interception devices, water harvesting and detention. The drainage report shall also include a long-term, post-construction maintenance plan for site final stabilization and water-quality controls, described in Section 11.3.5.

Projects disturbing less than the one-acre regulatory size limit and which are not part of a common plan of development do not need to submit a SWPPP Notebook or an Erosion and Sediment Control Plan. However, the grading plan or site plan must clearly show what Best Management Practices (BMPs) will be used and where, as well as the total disturbance area. The plan must also show installation details of BMPs to be used.

11.3 Criteria for Preventing Post-Construction Pollution Discharges

All new or substantially improved commercial developments shall collect, treat, or otherwise mitigate, runoff from paved parking and access lanes by using onsite retention storage, stormwater filtration, or by doing continuous good housekeeping, as described below.

11.3.1 Onsite Retention and Water Harvesting

As previously stated, the minimum required treatment volume is equal to the first ½-inch of runoff over the entire parking lot paved surface, and is calculated as follows:

$$V = ((0.5\text{ in}) / (12\text{ in/ft})) \times A \qquad \text{(Equation xx xx)}$$

Where: V = First Flush Volume, in Ac-Ft

A = Area of paved parking lot surface, including Parking Area Access Lanes (PAAL's), in Acres.

This volume of stormwater runoff may be sequestered in the bottom of a stormwater detention basin (i.e., the area below the lowest outlet) or in rainwater-harvesting basins receiving runoff from parking areas.

Disposal or runoff is by infiltration, the design of which is to be supported by the result from a percolation tests, with a 50% factor of safety added for silting over time. The maximum time allowable for disposal is 12 hours after cessation of rainfall. A satisfactory geotechnical report is required during plan review, and a favorable percolation test (ASTM D 3385, Double-Ring infiltrometer, or approved equal) is required prior to release of subdivision assurances. Typically, stormwater retention is not allowed except for minimal volumes used for water-quality mitigation, which are no deeper than one foot (1 ft.), and disposed of within 12 hours.

Water harvesting is encouraged if the maximum depths of ponding do not exceed 4 inches. The volume of runoff stored in rainwater-harvesting basins may be used to satisfy first-flush storage requirements.

11.3.2 First Flush Filtration and Hydromechanical Separation

The minimum required treatment rate is equal to the peak runoff from the paved parking lot and PAAL during a 2-year (or 50% AEP), 3-hour storm as calculated using the Category 1 (C1FHP) described in Chapter 3. Excess flow rates may bypass the filter or separator if it is done according to the manufacturer's design guidance.

There are several proprietary filtration devices and hydrodynamic separators that have proven successful in the Town. Examples, which are not intended as an endorsement or preference, include):

- REM Triton Filtration Devices -- Drop-Inlet and Catch-Basin Filters
- Oldcastle Filtration Devices -- Flogard Catch-Basin Inserts, Trench-Drain Filters,
- Oldcastle Hydrodynamic Separators -- Cyclonic, Dual-Vortex
- Contech Hydrodynamic Separators -- Vortech, Stormceptor, Cascade

11.3.3 Good Housekeeping and Pavement Sweeping

Good housekeeping and pavement sweeping are forms of "source control" intended to prevent pollutants from discharging from parking area and onto the Town's MS4. These Best Management Practices include pavement sweeping/cleaning, litter control, waste disposal, employee training, tenant guidance, and storm drain inlet cleaning. Each practice acts to reduce the accumulation of pollutants on impervious surfaces or within the storm drain system during dry weather, thereby reducing the supply of pollutants that can wash off when it rains. To be effective, these practices need to be done frequently, and with continued adjustments made to practice types and frequencies when indications arise suggesting that current level of effort is insufficient.

Implementation of this practice will require an Inspection and Maintenance Plan describing Best Management Practices to be used and their measurable performance standards; inspection and maintenance schedule; legally responsible parties; as-built plans of completed structures; and description of how and under what conditions the Maintenance Plan will be revised.

11.3.4 Design

First Flush treatment can be accomplished using a variety of differing technologies but must be able to remove 80% of the total suspended solids from the influent and ensuring the effluent will be free from oils and greases with no ongoing or recurring visible sheen. In addition, the first flush device will be capable of removing all water quality pollutants to the level prescribed in the ADEQ numeric water quality standards for the Aquatic and Wildlife (ephemeral), Partial Body Contact (PBC), and Agricultural Livestock Watering (AgL) designations where applicable. If determined to be not applicable, then a statement by the engineer of record is required identifying compliance with ADEQ requirements.

11.3.5 Access and Maintenance

As routine maintenance is required to ensure adequate performance for the life of proposed treatment systemsthe drainage report shall address the following:

1. Access to all First Flush treatment systems and appurtenant structures and facilities. Access is to be clearly identified on the grading and paving plans.
2. Manufacturer's maintenance recommendations.
3. Means of keeping inlet and outlet structures free of vegetation and debris.

11.3.6 Maintenance Records

The Drainage Report will describe the maintenance records to be kept, including:

a. Date of maintenance activities;
b. Description of maintenance activities;
c. Photographs of the treatment systems and appurtenances before and following maintenance activities;
d. Signature of responsible individual (i.e. HOA President) certifying that the noted maintenance was completed according to the approved schedule and requirements; and,

The report shall say maintenance records are to be retained by the responsible party for no less than three years following the most recent maintenance activity and shall be made available to the Town upon request.

12.1 Introduction

Post-developed stormwater peak discharges and total runoff volumes are generally larger than those found under pre-developed conditions, especially for high-density developments with paved roadways and constructed channels. Downstream impacts resulting from development can be mitigated by including stormwater detention as part of the site design. Stormwater detention is a flood control mitigation measure which delays the downstream progress of floodwaters in a controlled manner, generally through the combined use of a temporary storage area and a metered outlet device, which has the effect of reducing the volume by lengthening the duration of the flow and thereby reducing downstream flood peaks. Stormwater detention has a secondary water-quality benefit by reducing sediments and removing other pollutants.

12.2 Criteria

The following policies specify the type of development project requiring stormwater detention.

1. For the purpose of hydrologic analysis, all watersheds within the Town shall be designated as Balanced Drainage Basins. A Balanced Drainage Basin is one which has been identified as having the potential for severe increase in flood hazards because of urbanization within the basin, and therefore stormwater detention is required to the extent necessary to ensure that proposed 2-, 10-, and 100-year peak discharges do not exceed predevelopment values. Flood peaks are to be calculated for a 3-hour storm using the Category 1 Flood Hydrology Procedure (C1FHP) described in Chapter 3.
2. Some isolated areas of the Town already incur severe flooding problems, resulting in the Town Engineer potentially designating them as a Critical Drainage Basin based on recommendations in a Basin Management Study . A Critical Drainage Basin is one which has been identified as already having severe flooding problems due to existing watershed conditions. In these areas stormwater detention is required to the extent necessary to reduce existing 2-, 10, and 100-year peak discharges from the site. When deemed necessary, reductions are generally no less than 10%, with specific reduction requirements set by the Town.
3. Stormwater detention is to be incorporated into all residential developments larger than one acre in size and which have 30% or more impervious cover, equivalent to a development density of two or more unit per acre.
4. Stormwater detention is to be incorporated into all commercial developments larger than one-half acre, regardless of the amount of impervious cover.
5. In no case shall stormwater retention requirements be imposed upon an individual residential lot used for single-family residential applications, regardless of lot size.

12.3 Variance/Detention Waiver

The requirement for a waiver of stormwater detention may be satisfied if certain structural flood control measures are to be built or it can be demonstrated that detention at the site does not provide offsite flood relief due to parcel size, location within the drainage basin, or other factors. Requests for a variance are to be made separately from the drainage report submittal. In all situations, detention may be waived if it can be shown that such a waiver will not result in any adverse downstream effects, nor create any disturbance to the existing drainage patterns both within and adjacent to any such developments. At least one of the following shall be demonstrated when a detention waiver is requested, however doing so does not guarantee approval of a detention waiver request.

1. The rate of release from the project does not exceed the pre-developed flow rate. This can happen when walled-in yards in residential subdivisions reduce the effective contributing drainage areas.
2. The development is upstream of an existing or proposed flood-control channel or regional stormwater detention structure that have been designed to accept post-development flood peaks from the subject property.
3. Compensatory structural flood control measures are proposed instead of stormwater detention systems.
4. Affected downstream property owners accept, in writing to the Town Engineer, the potential increase in volume, flow, velocity, erosion/deposition of resultant flows. This acceptance shall be followed by recordation of a private drainage easement assigning responsibility for maintenance and liability, as well as for restricting future development.
5. The residential development, regardless of size, has a density that is less than 2 units per acre and preserves natural drainage patterns. The development shall not rely on constructed drainage facilities, such as constructed channels and storm drains to convey stormwater runoff.
6. The project is located adjacent to a major watercourse. A major watercourse is defined as one having a 100-year peak discharge of 10,000 cfs or greater. Approval to classify another watercourse as a major watercourse may be granted if engineering justification demonstrates adequate downstream capacity within the watercourse to convey the 100-year flood peak to a logical downstream terminii under conditions of ultimate watershed urbanization.
7. The project is located on a secondary tributary of a primary tributary, draining a watershed of no more than ten square miles, of a major watercourse and the relationship between the travel time of the discharge from the project and the rise times of the hydrographs of the project flows and the primary tributary satisfy Equation 9.1 of the 2015 Pima County Flood Control District “Design Standards for Stormwater Detention and Retention”; it can be demonstrated that the natural watercourses and drainage infrastructure within the secondary tributary watershed have adequate capacity to convey the future 100-year flood peak emanating from the watershed under conditions of ultimate watershed urbanization; and it can be demonstrated that the

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primary tributary peak discharge is not affected by the future 100-year flood peak from the secondary tributary or, if it is affected, the primary tributary and all drainage infrastructure downstream of the confluence of the secondary and primary tributaries have adequate capacity to convey the future 100-year flood peak under conditions of ultimate watershed urbanization.

8. The project site is eligible for a waiver due to other engineering justification acceptable to the Town Engineer.

12.4 Engineering Design

Hydraulic design is the responsibility of the design professional and must be demonstrated in the drainage report. The amount of required reduction of post-developed 2-, 10- and 100-year peak discharge rates depends on whether the project site is in a Balanced or Critical Drainage Basin. The following guidelines are provided to assist in meeting these requirements.

1. Peak flow rates at all points of concentration leaving the site cannot exceed pre-developed flow rates for Balanced Drainage Basins, and at least 10% smaller (or as required by the Town Engineer) for Critical Drainage Basins.
2. To demonstrate compliance, drainage reports shall include a summary table of 2-, 10, and 100-year flood peaks for existing and proposed conditions, and their numeric differences, for all drainage concentration points exiting the downstream perimeter of the project.
3. Basin outflow rates shall be computed using the reservoir routing procedure found in Section 3.3.2 of the 1987 Pima County DOT and City of Tucson Stormwater Detention/Retention Manual, with supplemental calculations for basin stage/storage/discharge values, including the hydraulic characteristics of the outflow structures.
4. Basin outflow rates can also be calculated using the most current version of the PC-ROUTE spreadsheet available from the Pima County Regional Flood Control District.
5. Stormwater discharge points for developed conditions shall be as similar as possible to those of pre-developed conditions.
6. Stormwater detention basins shall drain within 48 hours or less. Calculations and a summary table of the time to drain for each storm event are to be included in the drainage report. RILEY—DRAIN TIME WAS CHANGED FROM 12 HOURS TO 48 HOURS TO BE CONSISTENT WITH RFCD, IS THIS ADVISABLE IF PIGGY-BACK STORMS ARE TO BE ACCOUNTED FOR?
7. All basins shall have an emergency overflow structure (OR 100-YEAR WEIR STRUCTURE?) with a height that gives a minimum storage volume equivalent to the combined volumes of a 10-year and a 100-year runoff event ($\approx 1.5\ Vol_{100}$). Further, additional storage volume shall be included to account for incoming sediment if the basin is the on-line type. RILEY -- HOW ARE THESE ADDITIONAL STORAGE VOLUMES CALCULATED (E.G.

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EQUN. 3.8 IN OLD DET/RET MANUAL, AND THEN ACCOUNTED FOR IN THE RESERVOIR ROUTING CALCULATIONS?

8. Except for large-scale, regional stormwater detention basins, the Town will not accept small-scale, local detention basins for operation, maintenance, or liability. The Town may accept large-scale, regional detention basins on a case-by-case basis. If it is intended that a proposed basin be operated and maintained by the Town, the Town Engineer is to be consulted in advance.
9. Stormwater detention basin that cause floodwater to inundate parking lots are to be discouraged whenever a separate set-aside area can be provided within the development site.

12.5 Stormwater Retention Policy

A detention basin also can be designed to retain stormwater, and the volume of the retention within the basin below the outlet can be used to meet the first-flush retention requirements. On-line Stormwater Retention is allowed, with the following conditions, because it can significantly reduce the downstream impacts created -due to sediment deficit. Design policies are as follows.

1. Stormwater retention may be used for first flush, if the retention depth is no deeper than one foot, and it drains by percolation in less than 48 hours after a rainfall.
2. Disposal of the retention volume, if included as part of the site design, shall be by infiltration into the bottom of the basin, the design of which is to be supported by the result from a percolation tests, with a 50% factor of safety (RILEY PLEASE RECOMMEND A SAFETY FACTOR HERE) added to account for silting over time. A satisfactory geotechnical report is required during plan review, and a favorable percolation test (ASTM D 3385, Double-Ring infiltrometer, or approved equal) is required prior to release of subdivision assurances or permit closure.
3. If stormwater retention is planned and not used for first flush, it is recommended that the bottom of the basin be fitted with a low-flow drain pipe with an orifice plate, located only at the inlet end, to meter outflow rates.
4. If stormwater retention is planned and is used for first flush, it is recommended that the bottom of the basin still be fitted with a low-flow drain pipe with a semi-permanent steel plate affixed to the inlet end, which has the ability to be removed should the amount of time needed to drain the basin exceed 48 hours. If this occurs, the water quality maintenance plan would need to be updated to show what other equivalent filtration or good housekeeping measures will be put into place to prevent pollution discharges.
5. With prior approval by the Town Engineer, and with proper engineering design, stormwater retention systems may be allowed in lieu of stormwater detention systems for highly urbanized areas. This is primarily because retention offers multiple benefits that cannot be realized by detention systems alone—including: (1) elimination of flooding problems and channel instability issues through removal of urban-produced flow peaks

and volumes; (2) reducing channel instability issues that stormwater detention facilities typically create by exacerbating system-wide sediment imbalance; and (3) the total on-site control of urban stormwater runoff, which would provide better opportunities for use of stormwater runoff (e.g., stormwater harvesting) and aid in compliance with NPDES regulations. The design of on-line stormwater retention basins is beyond the scope of this Design Criteria Manual, and should the engineer wish to design such a structure, they are encouraged to meet with Town staff for guidance.

12.6 Design of On-Line Stormwater Detention Basins

An "on-line" stormwater detention basin is a storage facility located within the main sand-bed channel of a watercourse, thereby intercepting the entire water and sediment flow from the upstream watershed. In contrast, an "off-line" stormwater basin is a facility located within the development in question and away from the sand-bed channel. On-line stormwater detention basins almost always disrupt system-wide sediment balance of a watershed by "starving" sediment supply emanating from the contributing watershed areas; which will, in the long term, likely lead to destabilization of the overall fluvial system, primarily through the process of streambed degradation. It is for this reason on-line stormwater detention basins are not allowed unless the following conditions are met.

1. It can be favorably shown by a sediment-transport analysis of the downstream channel that the trapped sediment will not result in downcutting of the downstream channel over time.
2. On-line detention basins require greater storage volume to accommodate sediment accumulation, and the sizing of this additional storage is to use the procedure found in Section 3.4 (Sedimentation Impacts) of the 1987 Pima County DOT and City of Tucson "Stormwater Detention/Retention Manual."

12.7 Design Guidelines

Safe, appropriate design together with provisions for maintenance are the responsibility of the design professional and must be demonstrated in the drainage report. To assist with these measures the following guidelines are provided:

1. Basin aesthetics:
    a. The designer must follow the 1987 Pima County DOT and City of Tucson "Stormwater Detention/Retention Manual" for guidance on layout, safe side slopes, and aesthetic treatments.
    b. Landscape plans for basins are to be submitted for review as part of a development plan or site plan.
2. Detention Basin Depth Prohibition:

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   a. The maximum 100-year depth of water shall be less than three feet (3 ft.) in residential developments, school sites, parks, or other areas where children might congregate.
   b. The maximum 100-year depth of water in non-residential areas (excluding parks and schools) may exceed three feet (3 ft.) however the entire perimeter of the basin shall have security barriers to prohibit entry by people and pets.
   c. The maximum 100-year depth of water, regardless of development type, shall never exceed 6 feet (6 ft.), unless prior written approved is granted by the Town Engineer.

3. Detention Basin Security Barriers:
   a. Basins designed for 100-year water depths of greater than 2 feet (2 ft.) and with side slopes steeper than 3H:1V shall have a security barrier at all locations where side slopes are steeper than 3H:1V.
   b. A security barrier is required when a vertical drop is more than 30 inches, exceeds a down slope grade of 2:1, and is located less than 4 feet from the edge of the trail, pathway, walkway, or sidewalk, railing needs to be installed along the extent of the grade drop.
   c. Security barriers are required on any side of a basin where buildings or other restrictive structures are within five feet of the top of side slope and have no points of exit or entry into the basin area.
   d. Security barriers shall meet the following requirements:
      i. The security barrier shall consist of metal, masonry, or a combination of the two, installed in a manner consistent with the current building code.
      ii. Security barriers shall be a minimum,42 inch high railing-.
      iii. When constructed with steel pickets the barrier shall have vertical posts, bars, and top and bottom rails spaced so that a 4-inch sphere cannot be passed through the bars, or as otherwise required by Code.
      iv. Chain-link fencing is discouraged because due to appearance.
      v. The use of vegetation as a security barrier is prohibited.
      vi. Security barriers, if required, shall not restrict the hydraulic capacity of the structures.

4. Underground Detention Basin Restrictions:
   a. Underground stormwater detention basins are discouraged and will only be allowed if inflows are substantially free of sediment.
   b. Minimum underground storage chamber depth shall not exceed 5 feet unless manholes are provided for inspection and maintenance.
   c. Safe physical access for inspections, cleaning, and repair is required.
   d. The drainage report is to include a detailed maintenance plan consistent with the manufacturer’s recommendations and with OSHA rules for work in confined spaces.
   e. No buildings are placed over storage chambers.
   f. It is not located within the public right-of-way.
   g. The designer demonstrates that a field life of 50-years can be achieved.

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h. Underground storage systems shall provide 1.5 times the required 100-year detention volume. The additional 50% of the volume can be provided on the surface so that issues with improper drainage are observable. RILEY—DOES THIS ADDED VOLUME SATISFY THE PIGGY-BACK STORM REQUIRMENT?
i. Infiltration as a means of disposal is prohibited, without specific guidance from a Geotechnical Engineer, and with prior approval of the Town Engineer.
j. Providing first flush retention underground is prohibited.

5. Other Design Requirements:
   a. In no case shall detention basins discharge to streets.
   b. In no case shall roof-top detention basins be allowed.
   c. Provisions must be made to prevent velocity damage or clearwater scour damage at outflow structures.
   d. First Flush requirements may be incorporated into detention basin designs as described in DCM Section 11.7. However, the first flush retention volume shall not be drained by a low-flow outlet pipe, as this does not keep pollutants onsite.
   e. Low-flow outlet pipes, if used, shall be no smaller than 18 inches in diameter (preferably at least 24 inches) and can have orifice plates affixed to their inlet ends with an opening size sufficient to drain the basin in no more than 48 hours. The orifice plate can be solid, without an opening, so long as the plate is removable, if the basin is to be used for first flush, should percolation fail to adequately drain the basin.

12.8 Dry Well Standards

The following standards, from the 2015 Pima County Regional Flood Control District's Design Standards for Stormwater Detention and Retention, are to be followed in the design of dry wells.

1. When site constraints justify use of a single or multiple dry well to dispose of detention volume, approval to include them in a basin design shall be obtained from the Town Engineer prior to the first submittal of the Conceptual Site Plan or development plan. When requesting approval, the engineer must submit field investigation resultswith a preliminary site plan.
2. The field investigations shall include:
   a. Logs for soil borings to the anticipated depth of the dry well; and
   b. Determination of depth to groundwater in the proposed locations of drywells;
3. The preliminary site plan shall include at minimum:
   a. The location of the proposed dry well(s);
   b. The location of proposed structures with building footprints;
   c. Parking lot layout including pedestrian circulation; and
   d. The general drainage scheme.
4. Where dry wells are proposed as the sole method of outflow, the basin shall only receive sediment-free runoff, and be designed to retain the total of the 100-year runoff event.

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5. Where a dry well is proposed, failure of the system shall not pose a hazard to public safety or property.

6. Dry wells shall be registered with the Arizona Department of Environmental Quality (ADEQ) and designed, operated, and maintained in conformance with the most current ADEQ guidelines.

6. The engineering design of dry wells, including prohibitions, shall follow the guidelines in Section 4.19 in the 2015 Pima County Regional Flood Control District's "Design Standards for Stormwater Detention and Retention."
7. A separate covenant which specifies, or Conditions, Covenants and Restrictions (CCRs) which include, inspection and maintenance responsibilities shall be recorded when a dry well is used as a method of stormwater disposal.

12.9 Access and Maintenance

Routine maintenance is required to ensure adequate performance for the life of proposed treatment systems. The drainage report shall address the following:

1. Physical and legal access shall be provided to all stormwater detention basins and appurtenant structures. This shall include permanent ramps for vehicular access into all basins, the width of which is to be determined by the types of vehicles specified in the maintenance plan, otherwise they shall be at least14 feet wide. When needed, locking gates are to be provided on pedestrian barricades
2. Access is to be clearly identified on the grading and paving plans, final plats, and development plans.
3. A maintenance plan is required. Refer to DCM Chapter 15?.

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References:

1. Pima County Regional Flood Control District, 2015. Design Standards for Stormwater Detention and Retention.
2. Pima County Regional Flood Control District, 2020. User's Guide to PC-ROUTE Spreadsheet.
3. Pima County DOT and City of Tucson, 1987. Stormwater Detention/Retention Manual.
4. FHWA, 1980. Underground Disposal of Storm Water Runoff, Design Guidelines Manual. FHWA-TS-80-218
5. ASTM 2018. Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer, Standard D 3385-18.

13. STORMWATER HARVESTING

13.1 Introduction

Stormwater harvesting basins are depressed earthen areas located and designed to collect and retain rainfall or runoff from impervious or disturbed areas such as parking lots or rooftops for irrigation of vegetation. More specifically, Town Code Sec. 27.6.4 defines rainwater harvesting as "intercepting, catching, storing, diverting, or directing storm water runoff from roofs, parking areas, etc., during rain events and putting it to beneficial use." The terms "stormwater harvesting" and "rainwater harvesting" will be used interchangeably in the chapter.

13.2 Criteria

Town Code 27.6.C.1 (Applicability) says a landscaping plan is required, with some exceptions, for all residential and commercial projects requiring a development plan or plat, and

Specific sections of the Town Code pertaining to the water harvesting:

27.6.4.b.i.B.iv. Standing water for passive rainwater harvesting systems must infiltrate or dissipate within twelve (12) hours of rainfall cessation.

27.6.D.4.b.v. All water collected and utilized for rainwater harvesting from parking lots and streets must meet the same discharge quality as stipulated within the Town of Oro Valley Drainage Criteria Manual, Section 11.7, First Flush Requirements.

27.6.D.4.d.iii. Rainwater harvesting basins may be combined with site detention basins; provided, that the residual ponding will dissipate within twelve (12) hours. This shall be demonstrated by a combination of percolation, evapotranspiration and positive outflow device such as a metered pipe. At a minimum, a positive outflow pipe shall be installed no higher than four (4) inches above the basin invert.

27.6.D.4.e. Detention volume may be reduced at a one-to-one (1:1) volumetric ratio by the volume utilized for rainwater harvesting. This volumetric ratio must be confirmed for the two (2), ten (10), twenty-five (25) and one hundred (100) year storm events and approved by the Town Engineer.

27.6.E.4.c The maintenance schedule and requirements for rainwater harvesting basins shall be detailed in private covenants, conditions and restrictions (CC&Rs). The homeowner's association (HOA) or responsible party shall be clearly identified within the CC&Rs and shall be responsible for maintenance of basins and rainwater harvesting appurtenances and maintenance recordkeeping.

Town Code 15-18-10
Residential Rainwater Harvesting

Active and passive rainwater harvesting systems are allowed for use by residents of Oro Valley; provided, that active systems meet the requirements of applicable parts of Oro Valley Town Code Chapter 6 (Building) as to construction methods and that any system that ties into or uses potable water must have backflow protection installed and meet requirements of Article 15-23 of the Town of Oro Valley Water Code: Backflow Prevention and Cross-Connection Control Program. All collection vessels must be covered and mosquito proof. In passive systems, all standing water must infiltrate into the soil within twelve (12) hours. No ponding allowed.

Town Code 15-18-11
Rainwater Harvesting for Commercial and Master Subdivision Development Projects

Active and passive rainwater harvesting systems are allowed for use by these projects. All systems must first be approved under the Drainage Review Criteria Manual and the Town Engineer prior to installation and meet current Oro Valley Town Code Chapter 6 (Building) as to construction methods. Any system that ties into or uses potable water must have backflow protection installed and meet requirements of Article 15-23 of the Town of Oro Valley Water Code: Backflow Prevention and Cross-Connection Control Program. All collection vessels must be covered and mosquito proof. For passive systems, all standing water must infiltrate into the soil within twelve (12) hours. No ponding allowed. ((O)07-19, 2007.)

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Chapter 14. Maintenance of Drainage Infrastructure

14.1 Introduction and Purpose

Whether a facility is public or private, there is typically a stormwater control and/or conveyance component incorporated into its development. Stormwater components consist of catch basins, curb inlets, drop inlets, manholes, pipes, culverts, ditches, swales, channels, detention basins, levees, landscaping (both hardscape and softscape) and any other structure that collects, conveys and/or controls stormwater.

Maintenance of stormwater infrastructure is necessary to ensure as-designed function, stormwater quality requirements and maximization of service life. It is the responsibility of the Town of Oro Valley to ensure that all stormwater infrastructure components are properly maintained. The Town is directly responsible for those components located within the Town's right-of-way. The majority of stormwater infrastructure in the Town are privately owned and maintained by homeowner's associations (HOA's), property management companies, school districts and commercial/industrial site owners.

Subsection 2.6.11 of this Manual stipulates the preparation, during a development's design phase, of a maintenance and inspection plan as an element of the Drainage Report. The maintenance and inspection plan is to be prepared by a Civil Engineer, registered by the State of Arizona, to assure verification of proper operation and maintenance; evaluation of functional adequacy and structural stability; identification of aspects of drainage features to monitor over time; and providing a means to communicate the overall condition and aesthetics of the drainage infrastructure all while assuring the quality requirements of stormwater discharges from the site are met.

Considerations integral to a drainage infrastructure maintenance plan include growth of undesirable vegetation, debris accumulation, sediment deposition, erosion, scour, soil piping, soil settlement, structural damage and/or deterioration and adherence to Federal stormwater quality mandates.

Access required to provide maintenance of drainage infrastructure, as specified by the Drainage Report, and shown on the Improvement Plans, is considered an essential element requiring inspection and maintenance.

14.2 Criteria

14.2.1 Annual and Post-Storm Event Inspections

As specified in Subsection 2.6.11e, all drainage infrastructure and systems are to be routinely inspected, visually, at least once each year, by a Civil Engineer registered by the State of Arizona, to verify proper operation and maintenance. Inspections are also required after any storm event that could be anticipated to cause erosion, sediment deposition, deposition of trash/debris or damage to stormwater conveyance or control infrastructure.

Annual and post-storm inspections are to be documented in a brief report that will identify immediate maintenance and/or repair needs and be submitted to the owner. A copy of inspection report(s) will be available to the Town upon request.

14.2.2 Five-Year Inspections

Every 5 years a more rigorous inspection and assessment is to be conducted. The quinquennial inspection is to include a subjective evaluation by the Engineer of the criteria used at the time the drainage infrastructure was designed, inclusive of watershed and/or regulatory changes versus current design criteria in order to assess the ability of each drainage component and the overall system to function as intended as well as identify the potential need to upgrade the system to meet current design standards. Detailed design calculations are not required as part of the evaluation.

Depending on the significance of a deficiency noted during the quinquennial inspection, additional support from a Geotechnical Engineer, Structural Engineer and/or Landscape Architect registered by the State of Arizona, may, in the judgment of the Engineer, be deemed necessary. In the case where pumps may be a part of the system, a registered Electrical Engineer may be required.

The 5-year report will identify both immediate maintenance as well as anticipated maintenance needs over the ensuing 5-year period. This report will also include an opinion of the probable cost of maintenance and repair reasonably anticipated over the upcoming 5-year period to ensure adequate, available capital necessary to address such maintenance and/or repair. A copy of the 5-year inspection report is to be available to the Town upon request.

14.2.3 Report Content & Format

The inspection report is to generally address the following:

a. Project name.

b. Owner name and address.

c. Project location, including legal description and vicinity map.

d. Description of the site and all constructed stormwater control and conveyance components.

e. Purpose of the report (i.e., annual inspection, post-storm inspection or 5-year inspection)

f. History of system maintenance and deficiencies.

g. Deficiencies corrected since the last inspection and past deficiencies not yet corrected.

h. Evaluation of design criteria (quinquennial report only).

i. Inspection check list.

j. Photos depicting the current condition of drainage infrastructure.

k. Physical condition and any displacement of riprap or other forms of slope protection.

l. Verification that vegetation (trees, brush or undesirable weeds or grasses) is not creating an adverse effect on drainage infrastructure or conveyance.

m. Identification of immediate maintenance needs.

n. Identification of anticipated maintenance and/or repair needs over the upcoming 5-year period (quinquennial report only).

The report format generally will be as follows:

a. Text: Font use is limited to Helvetica, Arial, Calibri, Times and Times New Roman. Font size for general text is to be 11 or 12 point. All sections and paragraphs are to be numbered. Text is to be printed on 8-1/2 by 11-inch paper with a sufficient margin on the left for binding.

b. Content Headings:
   i. Table of Contents
   ii. Introduction
   iii. Project Information (i.e., location, vicinity map)
   iv. System Background Information (historic information including build date and past maintenance and/or repairs of significance)
   v. Inspection Findings and Evaluations
   vi. Conclusions and Recommendations
   vii. Appendices

c. Reproduction can be by any available process with printing done head-to-head, if possible.

d. Drawings or plates are to typically be 8-1/2 by 11-inch with a sufficient margin on the left for binding. Foldouts normally should not exceed 11 by 17 inches. Drawings and photos are to be included in the text or placed entirely in the appendices. Figures or drawings included in the text are to support the written material.

e. Photos taken during the inspection are to be interfaced with the appropriate inspection comments and include description, location and date taken.

f. Report covers are to be flexible paper or card stock with comb bindings or be bound in a loose-leaf binder. The name of the Civil Engineer conducting the inspection and the date of the inspection are to be noted on the cover together with the Engineer's seal.

g. The report shall also include the name of the Geotechnical Engineer, Structural Engineer, Landscape Architect and/or any other technical specialist should the inspection warrant such expertise.

h. Tree trimming, when required, is to be under the direction of a Certified Arborist who shall be identified in the report.

14.2.4 Recommendations and Follow-Up

Recommendations resulting in significant mitigation and/or repair work shall not be undertaken prior to their details, including a plan if required, being submitted to the Town of Oro Valley Stormwater Utility to assure appropriate review, acceptance and permit issuance and that follow-through by the owner occurs in a timely fashion.

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Significant mitigation and/or repair is defined as any work required to resolve a threat to future damage of stormwater conveyance or control infrastructure; cause deflection of stormwater flow; or result in overtopping of stormwater conveyance or control infrastructure.

Mitigation and/or repair recommendations will assure drainage will either (i) remain in its natural state and not be altered, disturbed or obstructed; (ii) be returned to the condition noted on the Improvement Plans accepted by the Town at the time of development; or (iii) modified in conformance with a plan, submitted by a Civil Engineer registered by the State of Arizona, and accepted by the Town.

Work of a significant mitigation or repair scope will require a follow-up inspection by the Engineer-of-Record within 10 business days of its completion. The Engineer will document the findings of this inspection and append it to the applicable inspection report that served as the basis of the work being undertaken.

Removal of sediment and debris from stormwater infrastructure components is considered solid waste requiring disposal in conformance with Federal, State, and local requirements.

Pesticide applications are to be administered only by a pesticide applicator licensed by the State of Arizona. All pesticide applications are to strictly follow the manufacturer's instructions and comply with applicable Federal, State and local regulations and ordinances.

Evidence of illicit discharges into components comprising stormwater infrastructure and conveyance systems include:

- Odor
- Color
- Clarity
- Floatables
- Deposits/stains
- Vegetation condition

Any such evidence of illicit discharge is to be noted during the inspection and reported to the Town of Oro Valley. The owner shall take take all appropriate steps to mitigate the source(s) of illicit discharge(s).

14.2.5 Authority

Subsection 15-24-14Q – Authority to Inspect of the Town of Oro Valley Code provides the Town Engineer/Public Works Director or their representative the right to periodically inspect drainage and detention facilities to verify that scheduled and unscheduled maintenance activity have been adequately performed. Should the Town find the owner or owners of the drainage and/or detention facilities deficient in their obligation to adequately operate and maintain their facilities, The Town, will place the owner or owners on notice of such deficiency by issuance of a *letter of opportunity to correct*, requesting a plan of action and time frame in which the deficiency will be corrected.

14.2.6 MAINTENANCE CONSIDERATIONS FOR DRAINAGE INFRASTRUCTURE

Inspection of drainage infrastructure components shall address the maintenance considerations which follow:

14.2.6.1 Catch Basins, Curb Inlets and Drop Inlets

Catch basins, curb inlets, and drop inlets are subsurface concrete structures that receive stormwater through a metal grate or slotted opening located at the surface. Structures are typically square or rectangular but can be round. The basin or inlet can also be designed with flow control and/or stormwater quality devices.

The primary function of the basin or inlet is to convey storm flow from the surface to a below grade conveyance system. Basins and inlets are typically designed with a sump to mitigate debris and sediment from being conveyed to and inhibiting flow in the piped conveyance system.

Catch basin, curb inlet, and drop inlet maintenance considerations include:

- Structural integrity of the grate and support frame. Ensure lid does not rock or move due to traffic.
- Pre-cast barrel sections in proper alignment, grade rings free from cracks, lifting or movement or damage.
- Free flowing inlets free of any debris or blockages.
- Evidence of infiltration into the structure at joints and/or grouting or discoloration above the sump indicating water intrusion.
- Cracks and/or deterioration of the structure or grouting including spalling concrete, exposure of reinforcing bars and/or discontinuous sections of grout or damage.
- Signs of abrasion, corrosion and/or deterioration of conveyance pipe at its connection to the basin or inlet.
- Remove sediment accumulations, debris and/or trash from inlet and/or basin. Excessive sediment accumulation is defined as that exceeding the depth of the sump or extending above the invert of the conveyance pipe.
- Mitigate sources of sediment, pollutant and/or debris impacting basins.
- First flush devices are maintained and replaced in conformance with the manufacturer’s recommendations.
- ???????

14.2.6.2 Manholes, Stormdrains and Culverts

Manholes provide surface access to underground pipe systems at deflections in horizontal alignment and at specified distances in the conveyance system.

Stormdrains convey surface stormwater, collected by stormdrains, or drop inlets, to receiving bodies of water.

Culverts convey streams or washes through embankments or under roads, railroads, or other infrastructure.

a. Manhole maintenance considerations include:
   - Structural integrity of the cover and support frame. If in traffic area, ensure lid does not rock or move.

- Manhole security and access features are in place and fully functional including locking lids (if applicable) and access ladder rungs.
- Barrel sections in proper alignment, grade rings free from cracks, lifting or movement.
- Evidence of infiltration into the structure at joints and/or grouting or discoloration above the sump indicating water intrusion.
- Cracks and/or deterioration of the structure or grouting including spalling concrete, exposure of reinforcing bars and/or discontinuous sections of grout.
- Signs of abrasion, corrosion and/or deterioration of conveyance pipe at its connection to the manhole.
- Excessive sediment accumulation in basin or inlet. Excessive sediment accumulation is defined as that exceeding the depth of the sump or extending above the invert of the conveyance pipe.

b. Stormdrain and culvert pipe maintenance considerations include:

- Inlet and/or outlet structures are free of. cracks, spalling, grout deterioration or discontinuous sections of grout, exposed reinforcing bars, settlement, lifting or rotational movement of pipe sections and damage.
- Evidence of infiltration at pipe joints.
- Cracks, abrasion, corrosion and/or deterioration along the inner surface of the pipe.
- Pipe segments are in alignment and show no evidence of shifting, shearing, cracking, lifting settlement, movement, or damage.
- Removal of sediment accumulations in excess of 20 percent of the diameter or height of the culvert.
- Signs of abrasion, corrosion and/or deterioration of conveyance pipe along its length.
- Remove debris and/or vegetation inhibiting conveyance from entering or exiting the stormdrain or culvert.
- Repair erosion at stormdrain or culvert inlet and/or outlet structures including spreader aprons.
- Repair riprap at culvert inlets or outlets where filter fabric has been exposed or if thickness of rock has been reduced.
- Ensure debris barriers and trash racks are free of debris, trash, and sediment accumulations. Replace or repair bars that are deteriorated, misaligned, bent, or damaged.
- The area within a 20-foot radius of stormdrain inlets or outlets is to be vegetation free.
- ??????

Note: Manholes, stormdrains and culverts are subject to Occupational Safety and Health Administration (OSHA) confined space regulations.

14.2.6.3 Detention Basins

a. Detention basin maintenance considerations include:

- Access gates, handrail, protective fencing, and security barriers are sound and in good condition and free from damage.
- Trees of shrubs hindering access to the basin are to be trimmed or removed if within the maintenance access road.
- Signage is in good condition and legible.
- Graduated basin depth marker(s) or story pole(s) are in place with all 0.1-foot marker increments legible.
- Vegetation control shall minimize and limit disturbance to areas requiring such control.
- Remove oil or other pollutants from the surface having a thickness greater than a surface sheen.
- Basin maintenance will ensure removal of excess sediment from sediment traps. Sediment accumulations greater the 12 inches above the lowest bottom floor elevation, as noted on the as-built plans for the basin, require removal. Sediment shall be disposed of, off-site, at a permissible location.
- Water harvesting basin design depth is to be restored when the design depth of the basin is reduced by more than 4 inches.
- Vehicular traffic, to the extent possible, is to be limited to access routes and sediment trap areas.
- Unless otherwise specified in the maintenance plan, vegetation is to be left as natural as possible. Pruning of vegetation is to be minimized unless otherwise specified in the plan.
- It is generally permissible to allow native trees, shrubs, grasses, and forbs to establish naturally from seed.
- Non-native trees and invasive grasses (i.e., Buffelgrass, Johnson Grass, Fountain Grass, etc.) are to be removed. A list of invasive, non-native plants can be found in Appendix E, List of Noxious & Invasive Plant Species & Best Management Practices of the Pima County Regional Flood Control District publication *Regulated Riparian Habitat Mitigation Standards and Implementation Guidelines* available on the Rules and Procedures Page under the Riparian Habitat tab of the District's web page. The link to this document is:

https://webcms.pima.gov/UserFiles/Servers/Server_6/File/Government/Flood%20Control/Rules%20and%20Procedures/Riparian%20Habitat%20Mitigation%20Plan%20Guidelines/onsite-guidelines.pdf

- Basin slope treatment does not show signs of settlement, vegetative growth, erosion, piping, slumping, sinkholes, seepage, animal burrows or other detrimental effects.
- Repair rill erosion of basin slopes, including mitigating runoff down slope areas.
- Basin inlet, outlet and overflow structures are free of sediment and debris.
- Ensure moveable components at outlet control structures are operable through their full range of motion and are free of damage.
- Repair erosion at basin inlet and/or outlet structures including spreader aprons and energy dissipators.
- Ensure inlet, outlet, overflow spillway and debris control structures are free of cracks, spalling, deterioration, exposure of reinforcing bars and/or discontinuous sections of grout, settlement, lifting, rotational movement or damage.
- Evidence of water ponding for more than 12 hours after a storm event for contributing watersheds up to 10 acres or 24 hours for contributing watersheds greater than 10 acres.
- Evidence of water ponding for longer than 24 hours for after a storm event in stormwater harvesting basins.
- Ensure aesthetic expectations.
- The area within a 20-foot radius of basin inlets, outlets and/or overflow structures are to be vegetation free.

b. Dry wells are perforated, open-bottom, circular structures used to infiltrate stormwater into subsurface, well-drained soils. Drywells are more likely to collect pollutants and oily sediments unless treatment elements are included.

- Remove sediment when its depth exceeds 10 percent of the height dry well.
- Structural integrity of the grate and the support frame. Ensure lid does not rock or move or is otherwise damaged.
- Replace drywell if it does not dissipate standing water after 24 hours of a storm event.
- Comply with all manufacturer's maintenance recommendations.

14.2.6.4 Channels

Channel maintenance considerations include:

a. General

- Access gates, handrail, protective fencing, and security barriers are sound and in good condition and free of damage.
- Trees of shrubs hindering access to the basin are to be trimmed or removed if within the maintenance access road.
- The area within a 20-foot radius of channel inlets or outlets is to be vegetation free.

b. Natural Channels

- Assess the extent of required vegetation control to ensure the design capacity of the channel has not been compromised.
- Sediment accumulations shall not exceed 20 percent of the channel depth or the design freeboard of the channel, whichever is less.
- Ensure the channel has the proper cross-section, flow line and is free of debris accumulations and obstructions.
- Maintenance equipment, to the extent possible, is to be limited to access routes and the channel bottom.
- Unless otherwise specified in the maintenance plan, vegetation on the banks is permissible and is to be left as natural as possible. Pruning of vegetation is to be limited to the conveyance area of the channel unless otherwise specified in the plan.
- Debris and/or vegetation inhibiting conveyance within the channel.
- Native trees, shrubs, grass, and forbs are generally not permitted to establish naturally from seed.
- Non-native trees and invasive grasses (i.e., Buffelgrass, Johnson Grass, Fountain Grass, etc.) are to be removed. (Refer to provisions addressing basin maintenance).
- Note areas of scour, particularly at stormdrain and culvert outlets.
- Repair rill erosion or slumping of side slopes.
- Repair rill erosion of channel slopes, including mitigating runoff down slope areas.
- Note if the channel is incurring downcutting due to excessive slope and/or clear runoff flows.
- Mitigate debris, pollutants, and sediment from being discharged into washes.

c. Constructed Channels

i. General

- Inlet and/or outlet structures are free of cracks, spalling, exposure of reinforcing bars and/or discontinuous sections of grout, settlement, lifting, rotational movement or damage.

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- Evidence of infiltration at joints.
- Cracks, abrasion, corrosion and/or visual deterioration along the surface of the channel.
- No evidence of shifting, shearing, cracking, lifting, settlement, or movement.
- Sediment accumulations shall not exceed 20 percent of the channel depth or the design freeboard of the channel, whichever is less.
- Remove debris and/or obstructions from the channel.
- Remove trees and bushes within 15 feet of the side of concrete lined channels.
- Remove woody vegetation growing through riprap.
- Repair riprap in areas where filter fabric has been exposed or if thickness of rock has been reduced.
- Repair erosion at channel inlet and/or outlet structures including spreader aprons.
- Repair rill erosion of channel slopes, including diversion of runoff from slope areas.
- Animal burrows along or in proximity to

ii. Partially Lined (Bank Protected)

- Scour undermining toe of bank protection.
- ___???????________

iii. Fully Lined (Sides and Bottom)

- ___???????________
- __???????________

14.2.6.5 Energy Dissipators and Outfalls

Energy dissipators are drainage elements installed to prevent erosion at storm drain outfalls, culvert outlets and where lined channels discharge into natural channels. A variety of energy dissipator designs exist including reinforced concrete structures, gabion baskets, riprap splash pads, and pools.

Outfalls are discharge points where stormwater enters a receiving wash at the end of a stormwater conveyance system.

Energy dissipator and outfall maintenance considerations include:

- Access gates, handrail, protective fencing, and security barriers are sound and in good condition.
- Trees of shrubs hindering access to the basin are to be trimmed or removed if within the maintenance access road.

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- Remove accumulated litter, debris, or sediment accumulations in excess of 20 percent of the conveyance area of the stormwater outlet/outfall.
- Repair riprap in areas where filter fabric has been exposed or if thickness of rock has been reduced.
- Remove vegetation and/or debris blocking the outlet of the dissipator or outfall.
- Concrete structures are free of cracks, spalling, deterioration of grout, exposure of reinforcing bars and/or discontinuous sections of grout, settlement, lifting, rotational movement or damage.
- The area within a 20-foot radius of energy dissipators or outfalls is to be vegetation free.
- ??????

14.2.6.6 Levees

Levees are embankments whose primary purpose is to furnish flood protection from seasonal high water and subject to water loading for periods of ranging from only a few hours to several days or weeks a year.

Levee maintenance considerations include:

- Indications of seepage through the levee.
- Settlement, slumping, sliding, cracking or erosion of levee embankment.
- Rutting.
- Rodent burrows.
- Trees and vegetation having a diameter greater than 1-inch are not permitted on or within 15 feet of the tow of the levee.
- Vegetation density or height which inhibits inspection of the levee surface.
- Access roads within the levee cross section are able to support the intended maintenance traffic and do not show signs of rutting or other deficiencies in their all-weather surface.
- Top of levee is graded so as to prohibit ponding.
- Erosion protection provided on the floodwater side slope is intact. Refer to the items contained in Subsection 14.2.6.5 for Partially Lined Channels.
- Evaluation of locations where pipelines or other utility lines cross the levee for settlement or signs of leakage.
- Closure devices on gravity pipe penetrations of the levee embankment are operational.
- Pressure pipe penetrating the levee embankments can be closed in order to isolate the portion of the pipe within the levee prism.

Appendix A – Drainage Report Checklist

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Report Checklist

The submittal of a drainage report, hydrology report, or drainage statement to the TOV requires the inclusion of a "checklist" which has been certified by an Arizona Registered Professional Civil Engineer. The submittal should contain all of the information necessary to be in conformance with the minimum submittal requirements of the TOV. The checklist should indicate that the following engineering information, at a minimum, has been included in the submitted report. An example checklist is provided here.

<u>Checklist Certification of Report Contents</u>

- ☐ Submittal Number (i.e., first submittal, second submittal, first addendum, etc.)
- ☐ Name and address of the parcel, project, or development for which the report is being submitted
- ☐ Proposed zoning of the development
- ☐ Planning Case Number
- ☐ Approximate location of the project site relative to Section, Township, and Range
- ☐ TOV Project Number
- ☐ Name, address, and telephone number of the client for whom the report was prepared
- ☐ Name, address and telephone number of the engineering firm responsible for the report
- ☐ Submittal date
- ☐ Seal and signature of the Arizona Registered Professional Civil Engineer responsible for preparation of the report
- ☐ Table of Contents
- ☐ All report pages numbered sequentially, including pages in appendices
- ☐ Site Location and Project Description
  - ☐ Brief description in report introduction of general location of parcel relative to nearby streets, drainageways, and washes
  - ☐ Site-location map, at a minimum scale of three inches equal to one mile, which shows the geographical relationship of the project to nearby properties, streets, and watercourses.
  - ☐ Legal description of the specific parcel or parcels in question.
  - ☐ Brief description of the type and approximate size of the project to be constructed
  - ☐ Clear indication to the reviewer, for detention considerations, whether the parcel being developed meets stormwater detention size requirements.
  - ☐ Identification of drainageways and roadways for which improvement plans will be prepared so the reviewer understands additional information will be forthcoming
- ☐ Purpose for submitting report (e.g., Tentative Plat/Development Plan approval, Building Permit Application, Floodplain Use Permit Application, condition of rezoning, etc.)
- ☐ Enumeration of report objectives

☐ Listing of applicable drainage and land-use policies (e.g., Basin-Management Plans, Neighborhood Plans, Specific Plans, etc.) that apply to the project site or its immediate vicinity; with a description of how these policies have been satisfactorily addressed during the design of the development

☐ Listing, as appropriate, of any rezoning requirements that relate to drainage and grading, with a description of how these specific requirements have been satisfied

☐ Identification of all known drainage studies for the subject parcel and for adjacent parcels which share drainageways and/or storm runoff.

☐ Listing of previous submittals of the subject report, if any, and reference to earlier staff correspondence, as appropriate

☐ Listing of the name(s), address(es), and telephone number(s) of the person(s), firm(s), agency(s) that will be responsible for the ownership, operation, scheduled and unscheduled maintenance, and liability of drainage improvements (i.e., roads, parking areas, washes, drainageways, detention/retention basins, common areas, etc.) described in the drainage report. List other documents where these responsibilities are documented (i.e., CC&Rs, Final Plats, Development Plans, etc.)

☐ Listing of permits which either have been, or will shortly be, obtained from those governmental agencies whereby approval is required by federal or state Law (e.g., a 404 Permit, a NPDES Permit, etc.)

☐ Indication that the owner/developer has, or will, obtain all necessary permits from other agencies prior to granting approval of the project

☐ Listing of the name and hydrologic information for any minor wash, major wash, or regional watercourse into which the project site drains

☐ Description of the size, location, and hydrologic characteristics of upstream and adjoining watersheds which may potentially affect the site

☐ Inclusion of a topographic map or (preferably) a photo-topo, at the appropriate scale, which shows:

- ☐ Parcel boundaries, major streets, drainageways, and nearby storm-drain systems (if applicable)
- ☐ Boundaries of the offsite watersheds affecting the site
- ☐ Principal points of drainage concentration
- ☐ Flowlines and grade breaks used to compute basin lengths and average watercourse slopes.

☐ Identification and description of both the existing natural and/or man-made impacts and the proposed major developments to be located within the contributing watershed which may impact the subject development, relative to flooding and erosion or sedimentation

☐ Identification and description, as appropriate, of the effects that nearby impending drainageway and/or roadway-improvement projects may have upon site drainage or site design. Also, specification of the time frame within which these improvements are planned.

☐ Inclusion of hydrologic data sheets for each principal point of drainage concentration for both pre-development and post-development conditions.

- ☐ Summary table of the watershed characteristics and flood peaks within the text of the report, listing whether the flood-peak estimates are for existing or future watershed conditions, or both
- ☐ Description of the size, location, and hydrologic characteristics of the onsite watersheds
- ☐ Inclusion of onsite drainage conditions on topographic maps scaled to an acceptable degree of resolution (e.g., one inch equal to 40 feet, with one-foot contour intervals)
- ☐ A Grading Plan, Tentative Plat, Development Plan, or Site Plan, modified to show onsite drainage conditions including:
    - ☐ Watershed boundaries
    - ☐ All points of drainage concentration
    - ☐ Flowlines and grade breaks used to compute basin lengths and average watercourse slopes.
- ☐ Hydrologic Data Sheets for each significant point of drainage concentration for both pre-development and post-development conditions, if applicable
- ☐ Summary table of the watershed characteristics and flood within the text of the report, listing whether the flood-peak estimates are for existing or future watershed conditions, or both.

Appendix B – Chapter X of the City of Tucson Standards Manual for Drainage Design and Floodplain Management

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CHAPTER X: STORM DRAINS

10.1 Purpose

The purpose of this chapter is to provide guidelines for the hydrologic and hydraulic analysis and design of storm drains. Hydrologic analysis consists of establishing the design discharge at specific points along the storm-drain system. Hydraulic design consists of inlet and conduit sizing, as well as quantifying various other storm-drain components, such as outlet works, conduit grade, junctions, transitions, etc.

10.2 Introduction

The primary purpose for the construction of storm drains is to remove stormwater runoff from streets and parking lots in as efficient a manner as possible. Water on street pavement can be a severe traffic hazard due to hydroplaning and loss of visibility from splash and spray. Ponded water in sumps can be especially hazardous due to the loss of vehicular control which results from one wheel catching the ponded water and the other remaining on drained pavement. Water on streets also causes traffic delays which result in loss of time and money. Storm drains are designed to reduce these risks to acceptable levels.

There are many reference sources available for storm-drain design. Some of these are listed in the "References and Selected Bibliographies" section of this Manual. Portions of this chapter were taken from these sources--primarily from the Federal Highway Administration (1984) and the Arizona Department of Transportation (1975). It is the purpose of this chapter to provide sufficient information for basic storm-drain design which will be applicable to most situations that will be found in the City of Tucson. More-detailed information can be found in the listed references, if a design situation should arise that is not covered in this manual. Where information in other references conflicts with this manual, the guidelines presented herein must be used, unless prior approval to the contrary is obtained, in writing, from the City of Tucson Engineering Division.

A storm-drain system designed for street drainage consists of a series of inlets designed to intercept street flow and convey it in an underground conduit to some logical outlet, such as a natural watercourse. Curbs, gutters, and transverse street slopes all function together to collect the water along either one or both sides of a street, where it can drain into the inlets. In order to understand how the complete storm-drain system operates, it is first necessary to understand how the individual components function. For this reason, design discharge and hydrology, street and gutter flow, and pavement inlet capacity and pipe flow are discussed in detail prior to presenting the overall design procedure.

10.3 Policies

Although there are many *technical* guidelines listed in this chapter regarding storm-drain design, there are also several other guidelines that could be considered as the *policies* which form the basis for storm-drain design in the City of Tucson. These policies are as follows:

1. The City of Tucson Hydrology method shall be used for storm-drain design, unless another method is approved in advance, and in writing, by the City of Tucson Engineering Division.

2. All offsite runoff, from whatever source, must be taken into account in storm-drain design, if the runoff could affect the street which the storm drain is designed to service.

3. Storm drains on arterial streets must keep at least one lane of traffic free from runoff during a 10-year flow. On lesser streets, storm drains must keep at least the 10-year flow between the curbs along the street.

4. The minimum pipe size allowable for public storm-drain systems is eighteen inches, unless otherwise approved in advance, and in writing, by the City of Tucson Engineering Division. In general, main-line sewers should be at least twenty-four inches in diameter.

5. Public storm drains may be designed for either open-channel or pressure flow, unless debris or the depth of cover is expected to be a problem. However, prior approval is required, in writing, from the City of Tucson Engineering Division if the storm drain is to be designed for open-channel flow conditions.

6. The self-cleaning flow velocity in storm drains shall be a minimum of three feet per second at a flow depth equal to 0.15 the pipe diameter.

7. The minimum allowable storm-drain slope for concrete pipe or smooth metal pipe shall be 0.1 percent. However, it is desirable that a slope of not less than 0.3 percent be maintained for all storm-drain pipe, whenever possible.

8. The minimum right-of-way width for storm drains should be the pipe diameter plus ten feet on each side of the pipe, unless a different right-of-way width is approved in advance, and in writing, by the City Engineer.

9. Public storm sewers shall be either RCP or an approved equal, except where CMP is required for the installation of vein drains or similar structures.

10.4 Design Discharge and Hydrology

Because storm drains are relatively expensive in comparison to surface-drainage systems, it is not often economically justifiable to design them for infrequent flow events, such as a 100-year flood. Where pavement drainage is concerned, the design is usually based upon keeping a specified width of pavement free from flow during a defined return-period runoff event. City of Tucson Development Standards (3.01.3.7A & B) require that, on arterial streets, storm drains be adequately designed to keep at least one lane of traffic, in each direction, free from accumulated runoff during a 10-year runoff event. On lesser streets, storm drains are required only if the 10-year runoff event cannot be contained between the curbs of the street.

There are other purposes for storm drains, such as drainage of detention basins or parking lots. The specified design discharge for these purposes may vary. In such cases, the design discharge would depend upon the particular situation, as well as the needs of the developer and/or regulatory agency.

Discharges and times of concentration for storm-drain inlets shall be computed according to the guidelines found in Chapter IV of this Manual, "City of Tucson Method for Estimating Flood Peaks and Flood Hydrographs". Where discharges less than the 100-year flood are required, as will be the case most of the time for storm drains, the appropriate ratios for peak flow rates, and corresponding formulas for their times of concentration, should be used at inlets.

10.5 Street and Gutter Flow

Flow in a street and gutter is normally confined by a curb, usually six to eight inches in height. The street-and-gutter cross section is generally either triangular or compound. A triangular section has a continuous grade, normally two percent, from the base of the curb to the crown of the roadway. The roadway surface can be one material, such as asphalt, from the base of the curb to the crown. However, it is not uncommon to install a concrete gutter adjacent to the curb. A typical compound section has a 1.75-foot-wide concrete gutter at a 4.8% grade adjacent to the curb, then a flatter (usually 2%) pavement grade to the roadway crown.

The capacity of a gutter depends upon its cross-sectional geometry, grade, and roughness. Grade and roughness are normally fixed by the topography and other design considerations. Consequently, cross-sectional geometry is usually the most flexible variable when increased capacity is desired.

The Manning equation should not be applied to flow in shallow, triangular gutter sections without modification, since the term for the hydraulic radius does not adequately describe the gutter cross section. Instead, the following equation should be used to compute gutter flow, rather than the conventional Manning equation:

$$Q = 0.56 \left(\frac{Z S_o^{1/2} Y^{8/3}}{n} \right) \tag{10.1}$$

or

$$Q = 0.56 \left(\frac{S_x^{5/3} T^{8/3} S_o^{1/2}}{n} \right) \tag{10.2}$$

Where:

Q = Discharge, in cubic feet per second;
n = Manning's roughness coefficient (see Table 10.1);
S_o = Longitudinal slope, in feet per foot;
S_x = Cross-slope of pavement, in feet per foot;
Y = Flow depth at curb, in feet;
T = Top width of water surface, in feet (commonly referred to as "spread"); and,
Z = Invert of pavement cross-slope, in feet per foot = $1/S_x$.

TABLE 10.1: MANNING'S ROUGHNESS COEFFICIENT ("n") FOR STREETS AND GUTTERS

Description	n-value
A. Concrete Gutter (Troweled Finish)	0.012
B. Asphalt Pavement	
(1) Smooth Texture	0.013
(2) Rough Texture	0.016
C. Concrete Gutter With Asphalt Pavement	
(1) Smooth	0.013
(2) Rough	0.015
D. Concrete Pavement	
(1) Float Finish	0.014
(2) Broom Finish	0.016
E. Brick	0.016
For gutters with small slopes (i.e., ≤ 0.3%), where sediment may accumulate, increase all values of "n" listed above by 0.002.	

Equations 10.1 and 10.2 apply only to streets with a uniform cross-slope. The solution to the equations can be obtained arithmetically, or by using the nomograph in Figure 10.1. Flow in composite gutters can be computed by breaking the cross-sectional area of flow into a triangular section and a trapezoidal section, and using the following formulas:

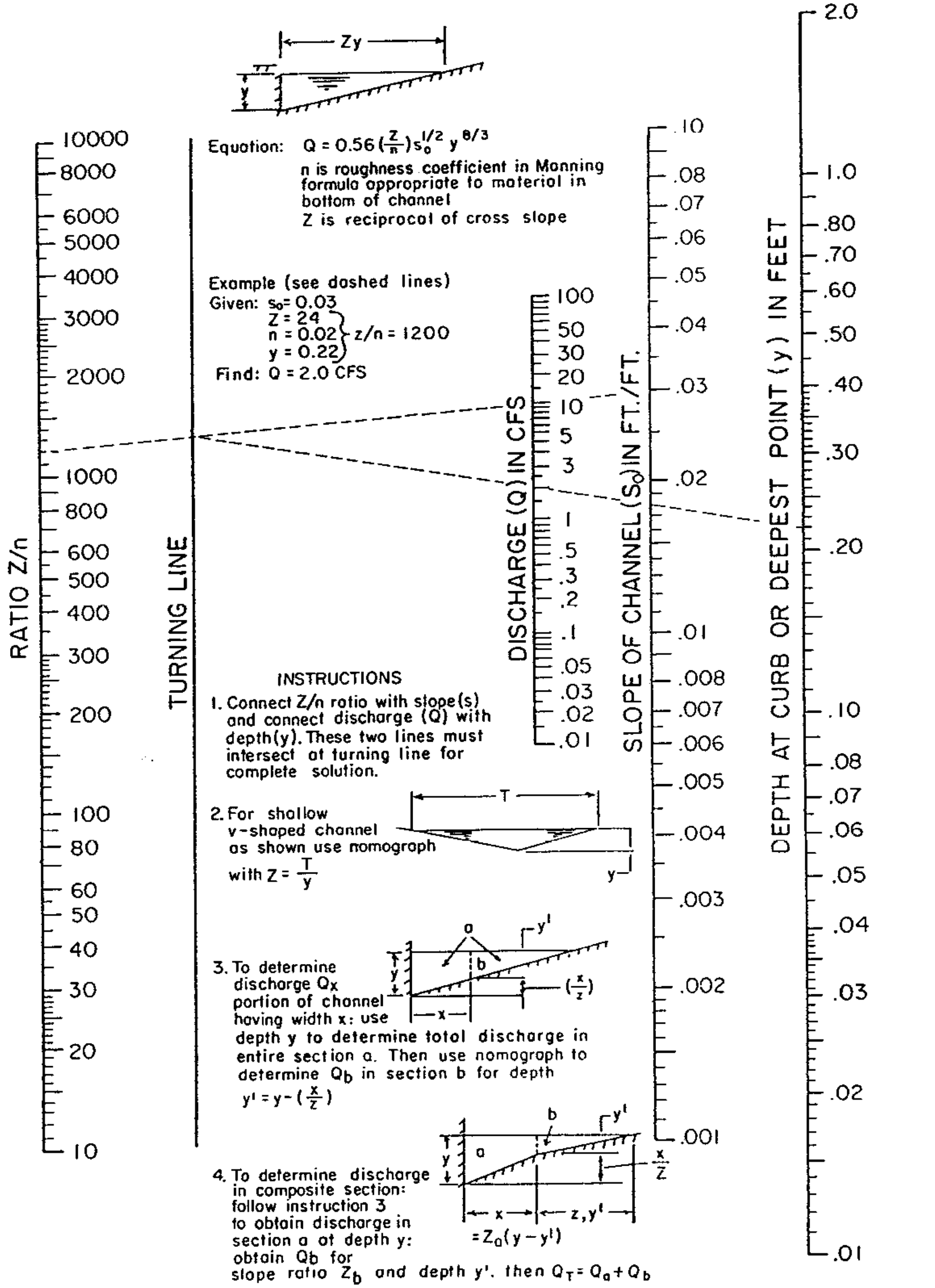


FIGURE 10.1

NOMOGRAPH FOR FLOW IN A WIDE, SHALLOW TRIANGULAR CHANNEL

TRIANGULAR SECTION:

$$Q = 0.56\left(\frac{Y^{5/3}\, S_o^{1/2}\, T}{n}\right) \tag{10.3}$$

TRAPEZOIDAL SECTION:

$$Q = 0.56\left(\frac{S_o^{1/2}\, T}{n}\right)\left(\frac{Y_{cf}^{\ 8/3} - Y_{gb}^{\ 8/3}}{Y_{cf} - Y_{gb}}\right) \tag{10.4}$$

The variables Y, Y_{cf}, Y_{gb}, and T are defined as shown in Figure 10.2. All remaining variables are as defined for Equations 10.1 and 10.2, and as shown on Figure 10.2.

The nomograph in Figure 10.1 can also be used to compute flow in composite gutter sections.

When gutters are on a continuous grade, the depth of flow at the curb affects the capacity of curb inlets. Correspondingly, the discharge across the width of a grate inlet determines grate capacity. Thus, the ideal gutter section for hydraulic efficiency will carry the design discharge concentrated near the curb, and at the greatest practical depth. This is more effectively accomplished with a composite section, rather than a triangular section. The recommended composite section has a 1.75-foot-wide concrete gutter, with a one-inch drop from the edge of the gutter to the base of the curb. Gutters with composite cross sections have the added advantage of carrying more water than triangular gutters, and without increasing the spread of water on the street.

Pavement cross sections in older streets are often more closely parabolic than triangular. Where the parabolic cross section rises upward from the base of a curb to a crown, it can be described by the following equation:

$$Z_p = 2\left(\frac{x\, H_c}{B_p}\right) - \left(\frac{x^2\, H_c}{B_p^{\,2}}\right) \tag{10.5}$$

Where:

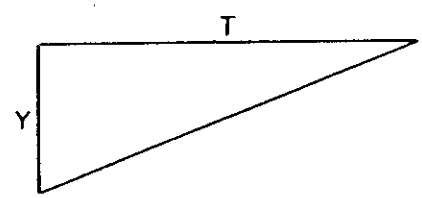
Z_p = Vertical rise of the pavement elevation along distance x of a parabolic curve, in feet.

x = Horizontal distance from the base of curb ($x \le B_p$), in feet; and,

H_c = Crown height of pavement cross section, in feet;

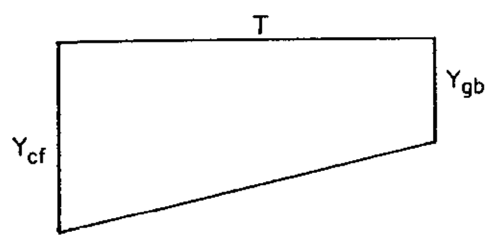
B_p = Horizontal distance from the base of curb to the crown, in feet;

TRIANGULAR SECTION



$$Q = \frac{0.56}{n} Y^{5/3} S_o^{1/2} T$$

TRAPEZOIDAL SECTION



$$Q = \frac{0.56}{n} S_o^{1/2} T \left[\frac{Y_{cf}^{8/3} - Y_{gb}^{8/3}}{Y_{cf} - Y_{gb}} \right]$$

FIGURE 10.2
COMPOSITE GUTTER SECTIONS

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:

$$Q_i = 3.0\ P_g Y^{3/2} \qquad \textbf{(10.10)}$$

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:

$$Q_i = 5.35\ AY^{1/2} \qquad \textbf{(10.11)}$$

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:

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

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:

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

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:

$$Q_i = 2.3\ L Y_i^{3/2} \qquad \textbf{(10.14)}$$

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:

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

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:

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

or

$$Q_i = 5.35\ AY_o^{1/2} \tag{10.16b}$$

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:

$$Q_i = 6.42\ LWY^{1/2} \tag{10.17}$$

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

L = Length of Opening

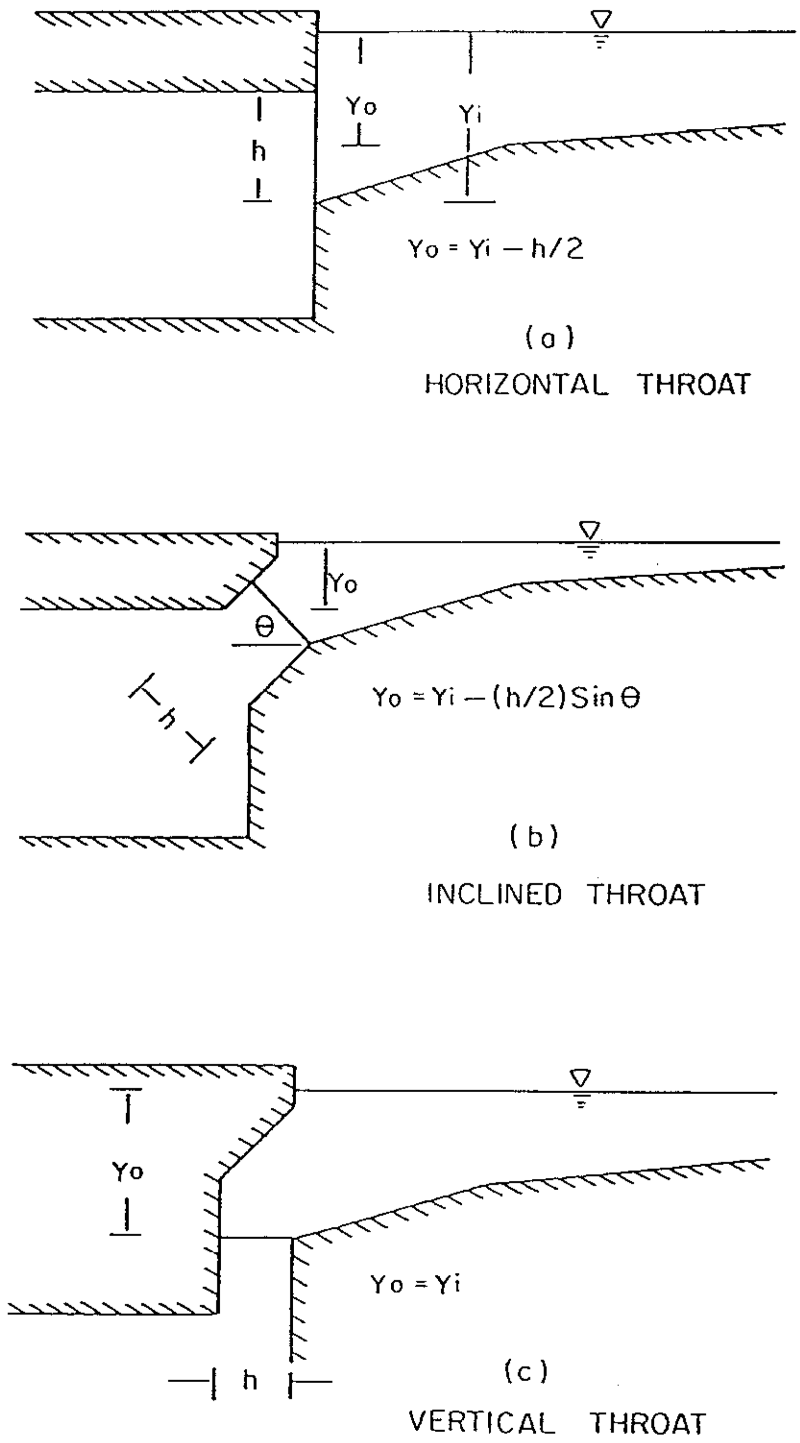


FIGURE 10.3
CURB-OPENING INLETS

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.

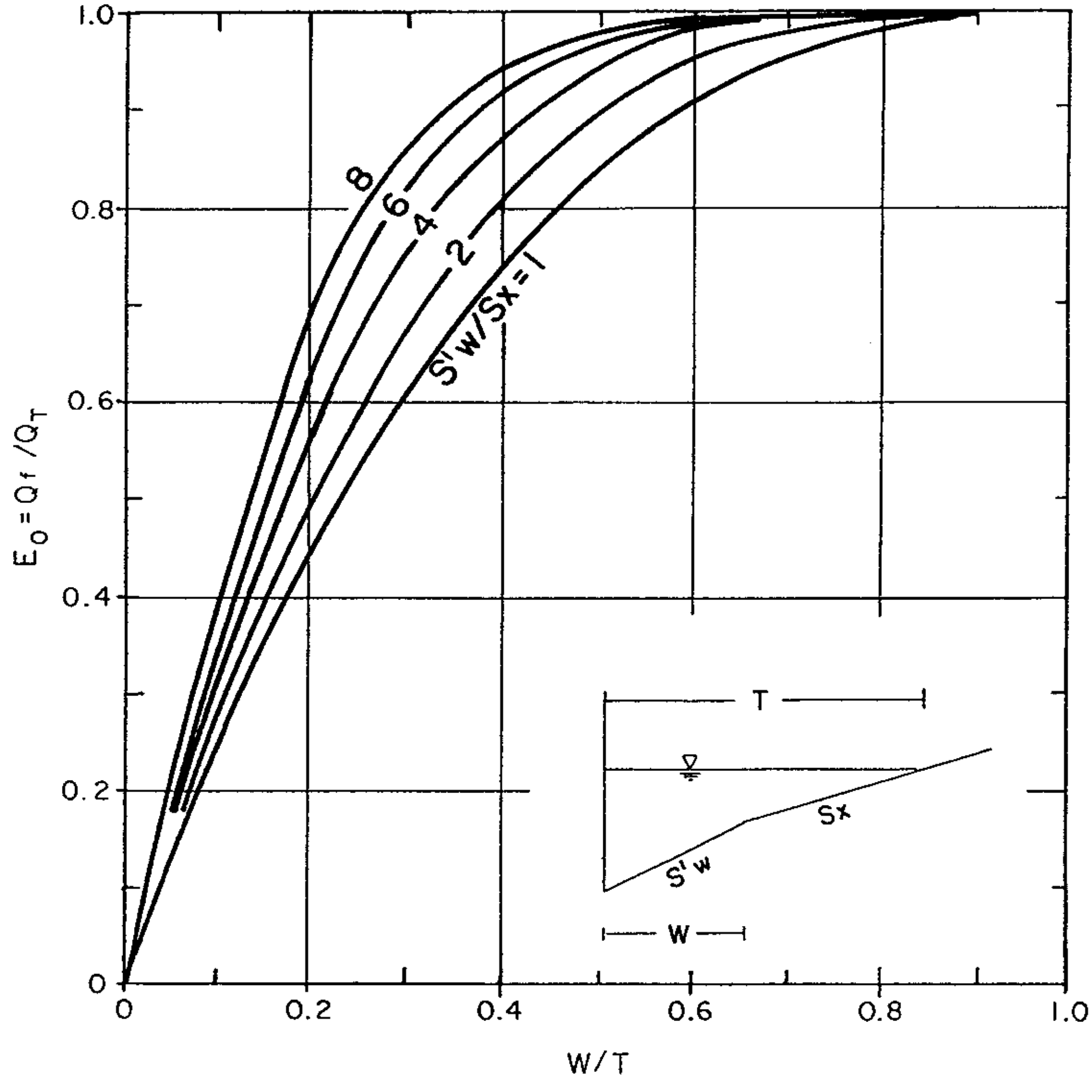
Figure 10.4 provides a graphical solution of the frontal-flow equation.

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

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

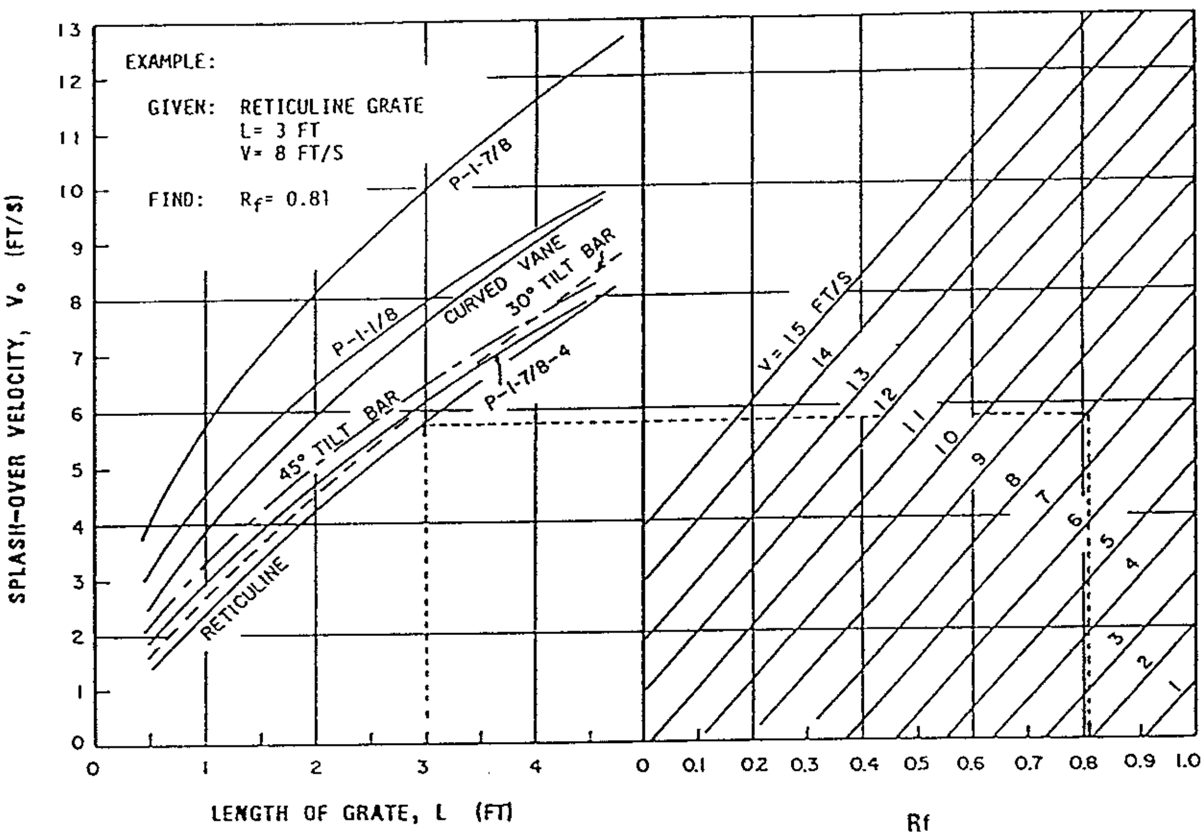
Where:
- V = Velocity of flow in the gutter, in feet per second; and,
- V_o = Gutter velocity at which splash-over first occurs, in feet per second.

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.



$$\boxed{Q_f = E_o Q_T}$$

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:

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

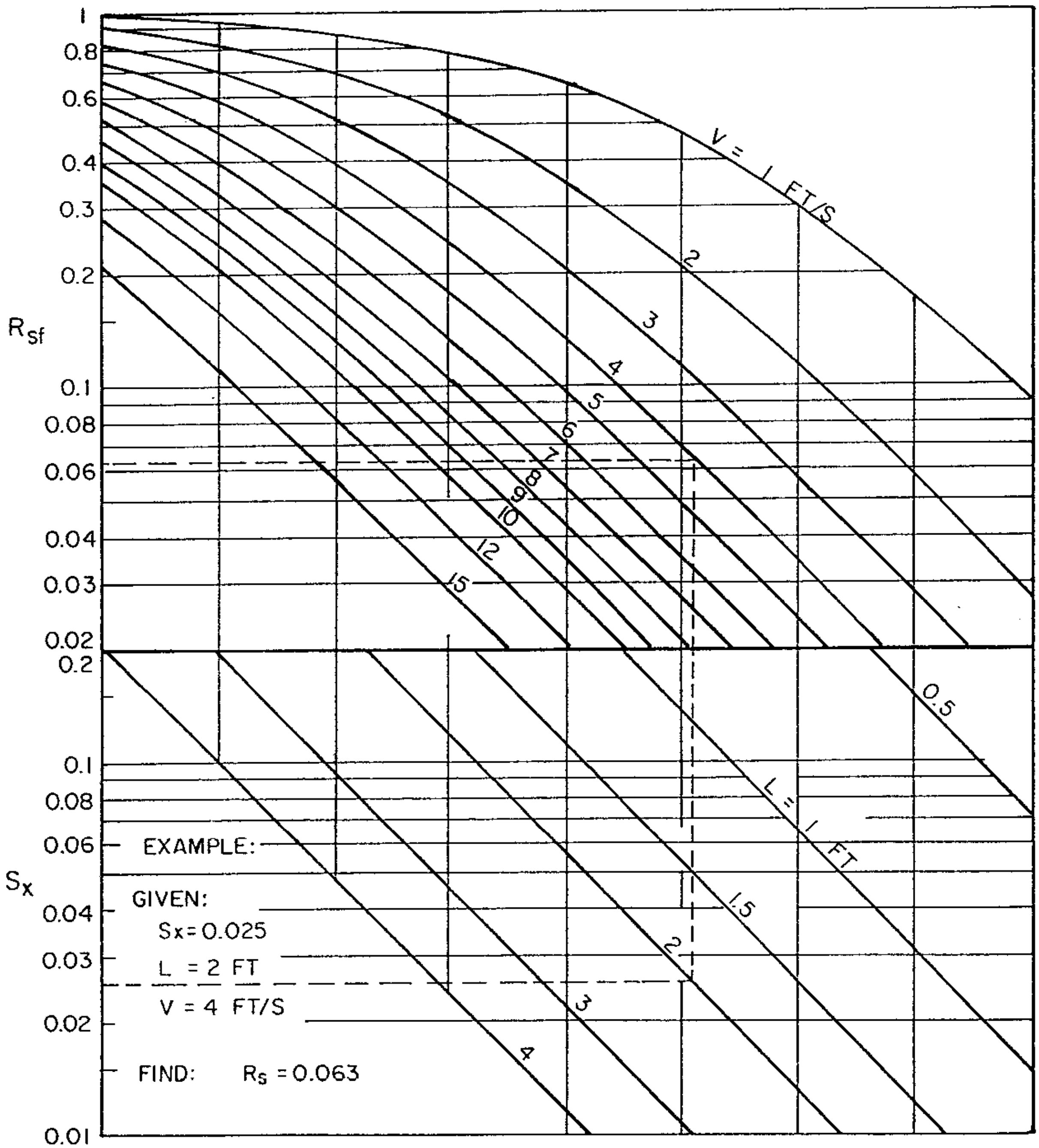
10.6.6 Capacity of a Curb Inlet on a Continuous Grade

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:

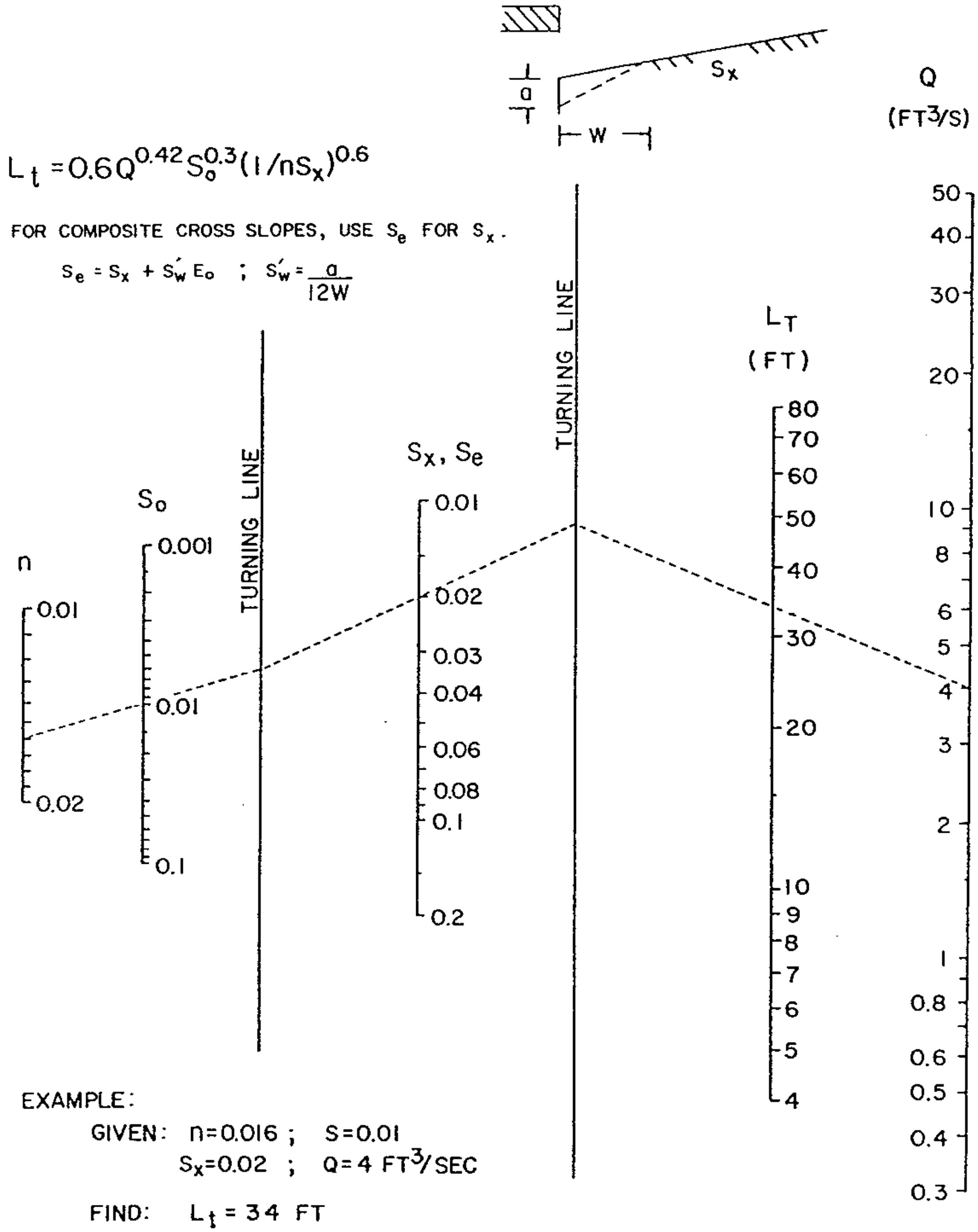


FIGURE 10.7
INLET LENGTH FOR TOTAL INTERCEPTION BY CURB OPENINGS AND SLOTTED DRAINS

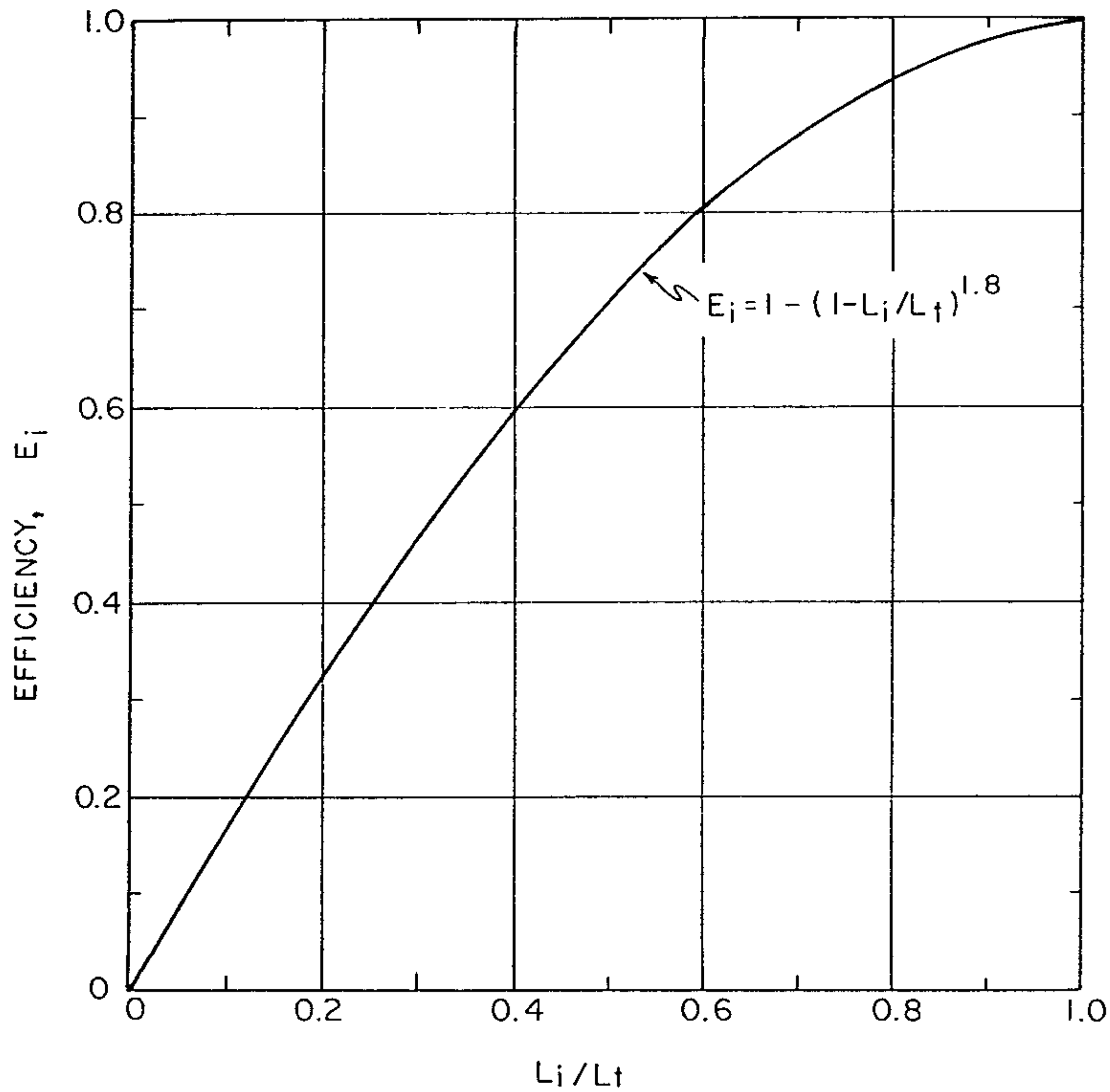


FIGURE 10.8

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.

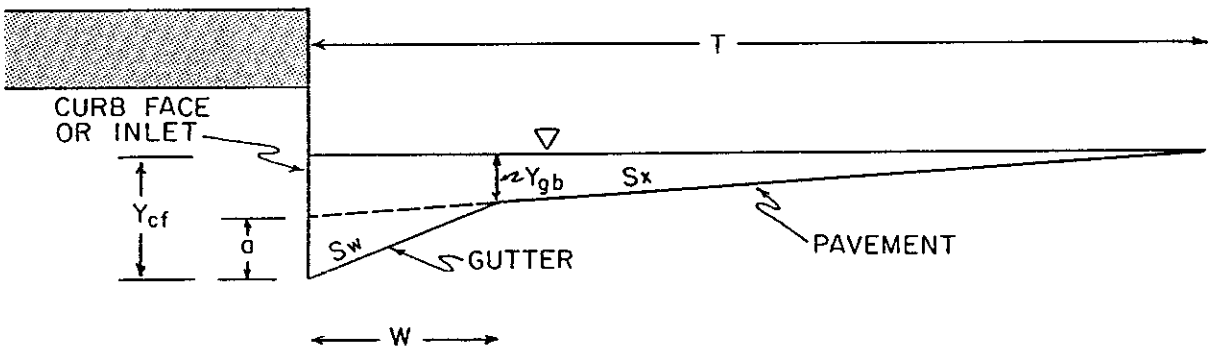


FIGURE 10.9
COMPOUND GUTTER SECTION

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.

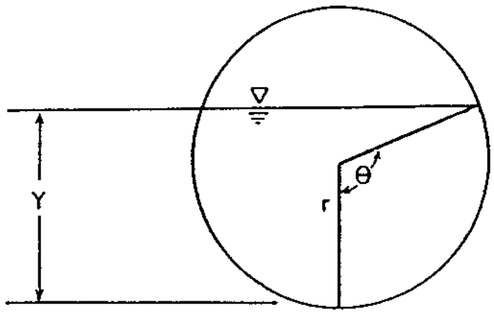
10.8.2 Pressure-Flow Calculations: Computation of Hydraulic Grade Line

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.



$\theta = \cos^{-1}\left(\frac{r-Y}{r}\right)$ (in radians)

$A = r^2(\theta - \sin\theta \cos\theta)$

$P = 2r\theta$

D = DIAMETER PIPE = 2r

MAX Q AT $\frac{Y}{D} = 0.938$

MAX R AT $\frac{Y}{D} = 0.813$

R = HYDRAULIC RADIUS = $\frac{A}{P}$

FIGURE 10.11
HYDRAULIC PARAMETERS OF A CIRCULAR CROSS SECTION

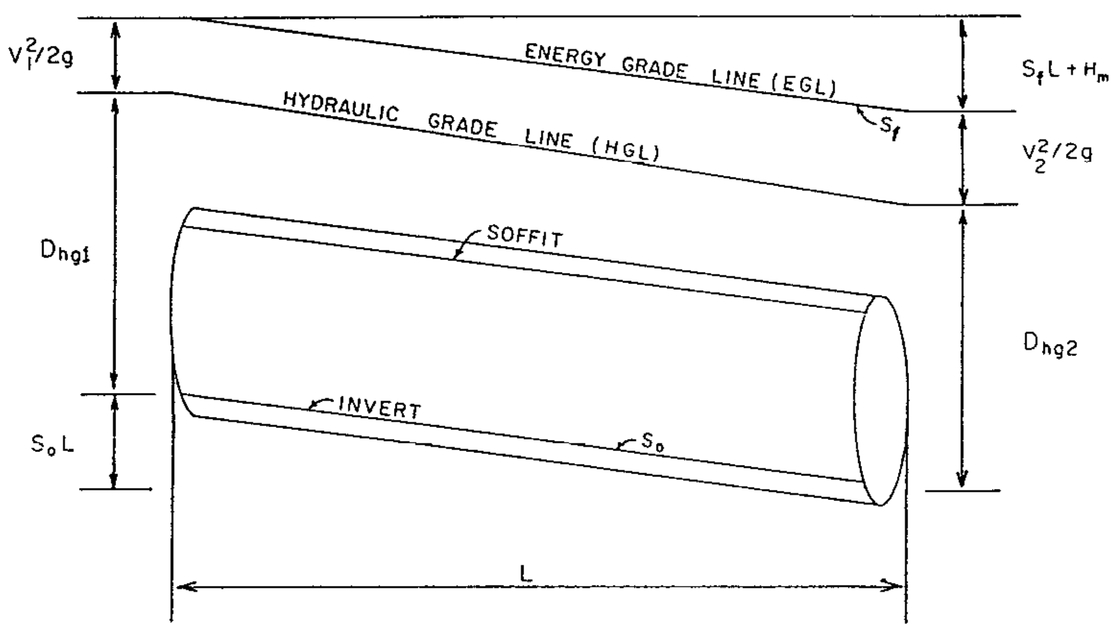


FIGURE 10.12
HEAD-LOSS DIAGRAM FOR PIPES

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:

$$S_f = \frac{29.2n^2}{R^{1.33}} \left(\frac{V^2}{2g} \right) \qquad (10.29)$$

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:

$$h_f = S_f\, L = \text{Friction loss} \qquad (10.30)$$

10.8.4 Minor Losses

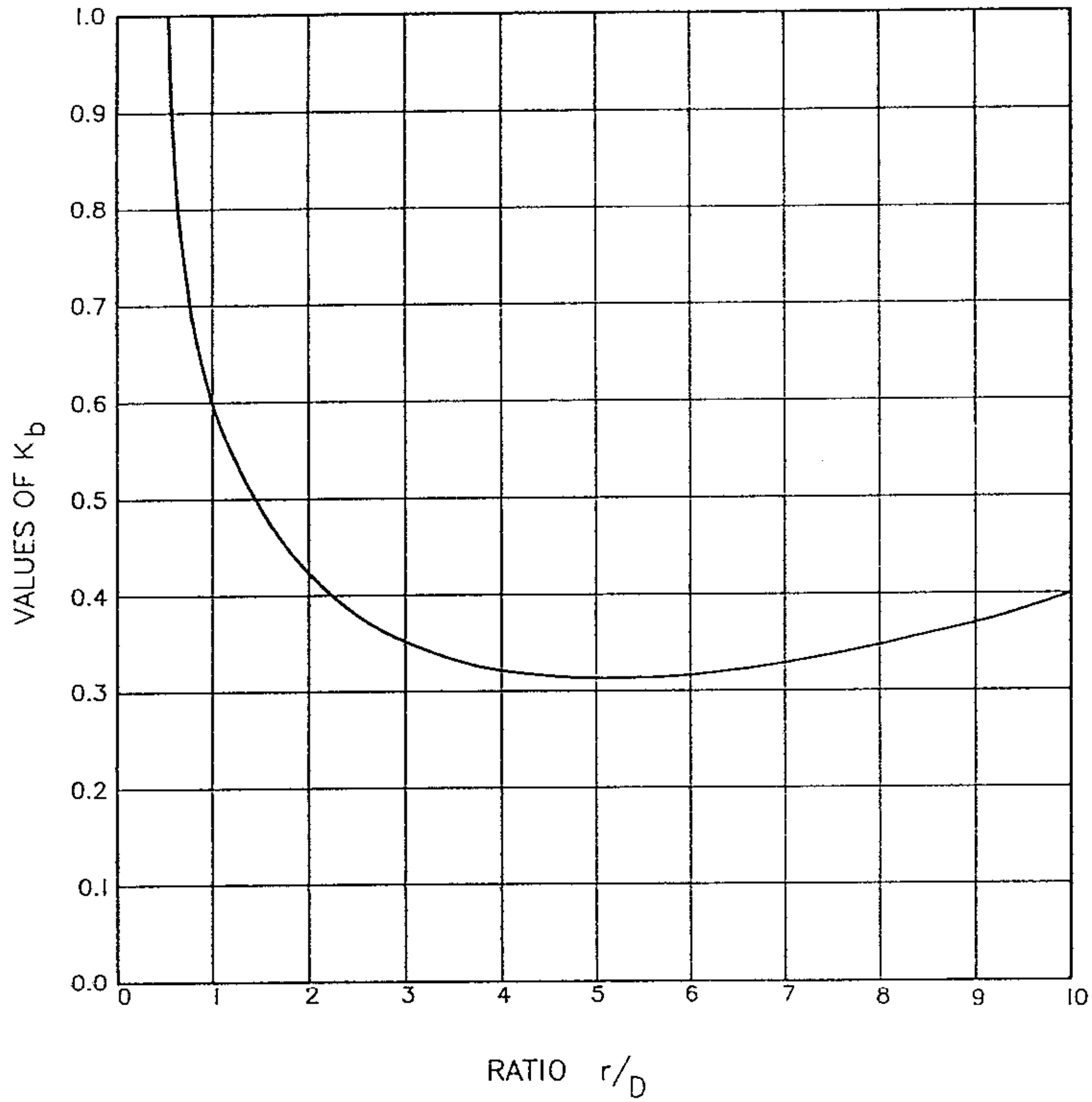
"Minor" losses in a storm drain are those that are associated with the energy necessary for the passage of water through areas such as junctions, manholes, and transitions. The total head loss is the sum of friction losses and minor losses. Minor losses, H_m, are normally represented as a factor K of velocity head:

$$H_m = K \left(\frac{V^2}{2g} \right) \qquad (10.31)$$

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:

$$h_j = \frac{V_2^2}{2g} - \frac{V_1^2}{2g} - \left(\frac{A_3V_3^2}{A_2g}\right) Cos\theta \tag{10.34}$$

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:

$$\Delta HG = \left(\frac{2}{A_1+A_2}\right)\left(\frac{Q_2V_2 - Q_1V_1 - Q_3V_3Cos\theta}{g}\right) \tag{10.35}$$

Figure 10.14B illustrates this type of junction.

Where:

ΔHG = Difference in hydraulic gradient for the two ends of the junction, in feet;

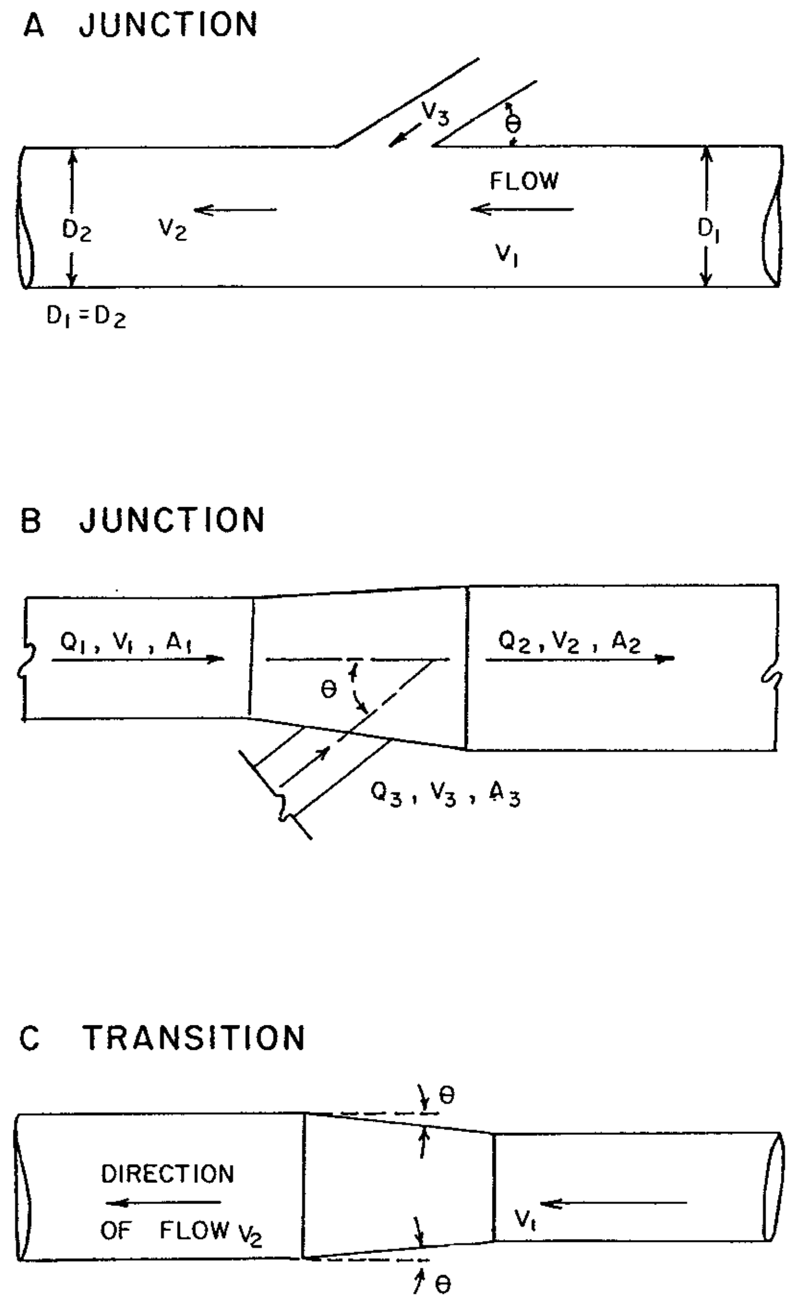


FIGURE 10.14
JUNCTION AND TRANSITION CONFIGURATIONS

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 $Q_n V_n \mathrm{Cos}\theta$ 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:

$$h_{te} = 0.2\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \quad (10.38)$$

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:

$$h_{mh} = K_{mh}\left(\frac{V^2}{2g}\right) \quad (10.39)$$

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:

$$h_e = K_e\left(\frac{V^2}{2g}\right) \quad (10.40)$$

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

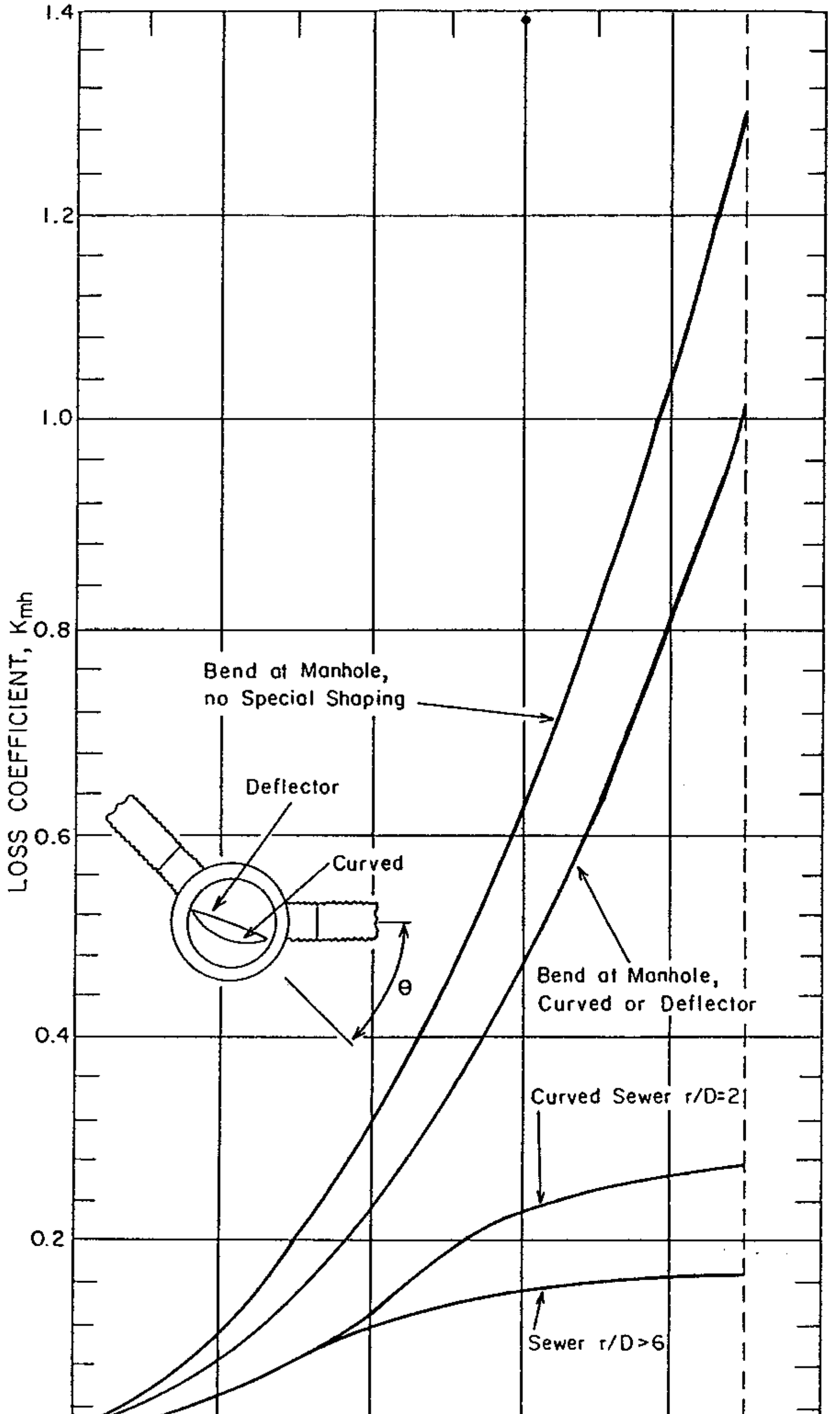


FIGURE 10.15
MANHOLE HEAD LOSS

Outlet losses, h_o, are only considered if the outlet is fully submerged, and are calculated by the following equation:

$$h_o = \frac{V^2}{2g} \tag{10.41}$$

10.9 Storm-Drain Design Procedure

10.9.1 Preliminary Design

After the storm-drain inlet locations have been established, preliminary pipe sizes should be chosen. Design discharges for the storm drain are computed using the City of Tucson hydrology method, according to the guidelines presented within Chapter IV of this Manual. *Basin factors should be computed according to Equation 4.2, using appropriate Manning's "n" values for the pipe sections.* A separate hydrologic data sheet is required at each inlet. The watershed area at each inlet is equal to the entire watershed draining to that point, including offsite watersheds that are to be connected into the storm-drain system. Consequently, a five-minute time of concentration *cannot* be assumed in every case.

Preliminary pipe sizes are established by using Manning's equation, as well as assuming "full-flow" conditions and a roughness coefficient 25-percent higher than what would be contemplated for use in final design (see Guideline No. 15, to follow) in order to tentatively account for the so-called "minor" losses in the system. Pipes should be sized for total discharge, minus allowable street flow. At first, the pipe slope can be assumed to be equal to either the surface grade or street grade, whichever is appropriate.

Junction and bend locations will already be known from the overall layout and inlet locations. In addition, transitions and manholes can be located and designed at this time, on a preliminary basis. In setting up the preliminary design, the guidelines listed below shall be followed:

1. Velocities shall always equal or exceed three feet per second at a flow depth equal to 0.15 the pipe diameter, and shall increase in the downstream direction in order to prevent sediment buildup.

2. The minimum pipe diameter for public storm-drain systems shall be eighteen inches, unless unusual design situations are encountered and written approval to use a smaller pipe size is provided, in advance, by the City of Tucson Engineering Division.

3. If possible, gutter flow should not cross intersections during the design runoff event.

4. The minimum allowable storm-drain slope for concrete pipe or smooth metal pipe shall be 0.1 percent. However, it is desirable that a slope of not less than 0.3 percent be maintained for all storm-drain pipe, whenever possible.

5. Channel bank protection shall be required at points where storm drains discharge into earthen channels.

6. Unless calculations show that head loss will not be excessive, the angle of confluence between a mainline and a lateral shall not exceed forty-five degrees; and, as an additional requirement, shall not exceed thirty degrees under any of the following conditions:

   a) Where the flow in the lateral exceeds ten percent of the mainline flow;

   b) Where the velocity of flow in the lateral is twenty feet per second, or greater;

   c) Where the diameter or equivalent size of the lateral is sixty inches, or greater; and,

   d) Where hydraulic conditions indicate that excessive head losses may occur in the mainline due to a confluence.

7. The soffits of adjoining pipes in a transition or junction shall be set at the same elevation.

8. Manholes shall be located at such places as junctions; changes in pipe size; sharp curves and angle points in excess of ten degrees; and points where an abrupt change in grade occurs. In addition, it is suggested that manholes be located at regular intervals along the line. It is recommended that the minimum spacing interval for manholes along conduit less than or equal to thirty inches in diameter be 300 feet; and if the conduit is greater than thirty inches in diameter, but less than or equal to forty-five inches in diameter, the minimum spacing be 400 feet. Also, if the conduit is greater than forty-five inches in diameter, the recommended minimum spacing is 500 feet; and for conduit less than or equal to thirty inches in diameter, the recommended minimum spacing is 200 feet, if there are bends and angles within the system.

9. Manholes shall be located in the center of the street travel lane, where possible; and not in the wheel path or within street intersections.

10. Storm-drain conduits shall always be designed to flow full and under pressure, unless debris is expected to be a problem; or unless prior approval is obtained, in writing, from the City of Tucson Engineering Division which permits designing for open-channel flow conditions.

11. Pipe sizes shall generally increase in the downstream direction, unless smaller pipe would operate just as effectively and thereby allow for a savings in the overall cost of the system. For such cases, the minimum diameter to which a conduit can be decreased shall be thirty inches. Six inches shall be the maximum allowable decrement in the conduit diameter.

12. Where storm drains discharge into an open channel, the water-surface elevation within the channel which has the equivalent return-period that is used for the storm-drain design discharge shall be the controlling water-surface elevation for hydraulic grade-line calculations, unless approval to the contrary is obtained, in writing, from the City of Tucson Engineering Division. When discharge is into another conduit, the design hydraulic grade line must become the controlling water-surface elevation.

13. In most cases, the hydraulic grade line of conduits flowing under pressure shall be at or below the level of the ground. At inlets, the hydraulic grade line shall be at least six inches below the ground surface.

14. Storm drains draining long roadways need not always run the entire length of the roadway, even though the roadway may be on a continuous grade. Where feasible, discharging into convenient outlets along the way can reduce costs by minimizing the required pipe size.

15. It is recommended that roughness coefficients used for final design be for "aged" conditions, and be approximately fifteen percent greater than those coefficients ordinarily used for new conduits.

16. A drop of 0.1 foot shall be provided at a through manhole, and a drop of 0.3 feet at a manhole intersection with two laterals. If a conduit changes direction in a manhole without changing size, a drop of 0.4 feet shall be provided.

17. When two laterals intersect a manhole, the laterals shall not be aligned opposite one another. The centerlines of the two laterals shall be separated laterally by at least the sum of their pipe diameters. A deflection shall be used, if necessary, to achieve this layout.

18. Storm drains shall be straight, with uniform slopes between manholes, if possible. The minimum radius of curvature for bends shall be 100 feet. Shorter radius curves, but not less than the minimum values given in the following table, are acceptable only by obtaining written permission from the City of Tucson Engineering Division.

MINIMUM ALLOWABLE RADIUS OF CURVATURE

PIPE DIAMETER	MINIMUM RADIUS OF CURVATURE
24" - 54"	28.5 Feet
57" - 72"	32.0 Feet
78" - 108"	38.0 Feet

Short-radius bends, such as the above, shall only be used where the conduit is to make a direction change just upstream of junctions or manholes, in order to meet the previous design criterion (i.e., item 17 above).

19. Crossings with other underground utilities shall be avoided, if at all possible. Crossings, if absolutely necessary, shall be at angles greater than forty-five degrees.

20. The minimum allowable clearance between sanitary sewers and storm drains shall be twenty-four inches, unless the upper pipe is founded (such as with piles) in the vicinity of the crossing.

21. There shall be a minimum of three feet of cover over the crown of the conduit, wherever possible.

22. The minimum right-of-way width for the installation of a storm drain shall be the pipe diameter, plus ten feet on each side of the pipe.

10.9.2 Final Pipe Sizing: Hydraulic Grade-Line Calculations

The final design of a storm-drain system is done either through backwater calculations (if the flow has a free surface) or through hydraulic grade-line computations (if the pipe flows full and under pressure). The hydraulic grade line is the level to which water would rise were it not constrained by the physical boundaries of the pipe. Where there are manholes and inlets with clear passageways to the surface, water will rise to the level of the hydraulic grade line. For this reason, it is very important that the hydraulic grade line be kept below the ground surface. Otherwise "blow-outs" of manhole covers could occur, and water could flow out of the storm-drain system into the streets. In order to provide for a safety factor, the hydraulic grade line should be kept a minimum of six inches below the level of surface inlets such as grates, curb inlets, and manholes.

If the hydraulic grade line extends above the ground surface, or drops below the soffit of the pipe, revisions to the design will be needed. These revisions could include changing the pipe size, slope, depth, or roughness; or making transitions, bends, manholes, and junctions more efficient in order to reduce head loss.

Hydraulic grade-line calculations normally proceed in an upstream direction, according to the following procedure (An example of this procedure is provided at the end of this chapter):

1. Establish a control water-surface elevation and total energy level at the storm-drain outlet. Generally, it shall be assumed that the tailwater elevation at the storm-drain outlet is equivalent to the water-surface elevation within the receiving channel which has the same return period as the storm-drain design discharge, unless approval to the contrary is obtained, in writing, from the City of Tucson Engineering Division. If the tailwater is above the soffit of the pipe, the control water-surface elevation and total energy level shall both be assumed to be equal to the elevation of the tailwater pool. If the drain has a free outfall, the elevation of the soffit of the pipe at the outfall shall be assumed to be the control elevation. Add velocity head to this elevation to obtain total head.

2. Using the known discharge, preliminary pipe size, and layout computed previously, compute friction slope, S_f, for the flow at the outlet, using Equation 10.29.

3. Choose a straight length of pipe, L, upstream of the outlet for which the pipe discharge, slope, diameter, and roughness do not change; and compute the friction loss, h_f, for that reach by Equation 10.30. Add the friction loss, h_f, to the total energy elevation at the outlet. This is the elevation of the energy grade line, *EGL*, at distance L. Subtract velocity head from this elevation in order to obtain the elevation of the hydraulic grade line, *HGL*.

4. Step 3 is repeated in logical steps, in an upstream direction, to the end of the pipe. "Minor" losses at bends, junctions, and transitions are taken into account, using the formulas given in Section 10.9. Hydraulic grade-line calculations are to be performed at every point where a "minor" head loss exists; where there is a change in a pipe slope or diameter; and at any other point where hydraulic characteristics change. Hydraulic grade-line calculations should be based on *total* head, instead of hydraulic head, in order to avoid confusion and error. The sheet in Figure 10.16 is provided for use in organizing and documenting information as the computations proceed upstream.

5. Plot the hydraulic grade line and total energy grade line in relation to the storm drain and ground surface. *Care should be taken to ensure that the pressure within a pipe does not exceed the manufacturer's maximum safe limits for joints and seals.*

6. Make design revisions, as necessary, and repeat the procedure until the desired optimization of the storm-drain system is achieved.

10.10 Suggested Design Practices

In addition to the information presented in the earlier portions of this chapter, there is a certain amount of minimum information which must be available to the storm-drain designer in order to produce a complete and accurate design. Some of this information, which is likely to apply to most storm-drain designs, is listed below. Because every design case is different, this list should not be considered as complete. Additional information, as needed, should be obtained by the designer as the situation arises.

1. Accurate, recent topography, at a scale no smaller than 1" = 200', preferably with at least two-foot contour intervals superimposed on aerial topography, should be obtained for delineation of all offsite watersheds and for determining watershed parameters. Zoning maps, area plans, and projected zoning predictions should be used for estimating future watershed development.

HYDRAULIC GRADE–LINE CALCULATION SHEET

Sheet ______ of ______

PROJECT: ______________ CALCULATED BY: ______

LINE: ______________ DATE: ______

STA.	ELEV. INV.	D	ELEV. H.G.L.	SEC–TION	D_{hg}*	A	Q	V	$\frac{V^2}{2g}$	S_f	AVG S_f	L	h_f	h_b	h_j	h_t	h_{mh}	ELEV. E.G.L.

*Conduit unseals when D_{hg} is less than D.

FIGURE 10.16
HYDRAULIC GRADE–LINE CALCULATION SHEET
FOR PRESSURE FLOW

2. Detailed site information for all points of offsite drainage concentration should be obtained. This should include topography, land ownership, construction plans, and all other information necessary to determine flow hydraulics and to design pavement inlets.

3. The storm-drain designer must have detailed information, or control of detailed information, on final grades, street geometries, and all other street details affecting the design, construction, or operation of the storm drain.

4. Records must be obtained ("as-builts," when available) showing the size, construction details, and location of all existing utilities, pipelines, and structures above and below ground. This would include, but not be limited to, water lines, sanitary sewers, storm drains, gas lines, electric lines, telephone lines, and traffic-signal lines. Current plans for future installations should be checked to prevent conflicts. Information that is incomplete should be checked by a field survey.

5. Traffic, pedestrian, and general public-safety considerations should be listed in order to produce a compatible design. For instance, these considerations could have an impact upon the type of inlets proposed at certain locations.

6. Right-of-way constraints should be accurately mapped. In most cases, the storm drain will be entirely within a street right-of-way; but there can be cases in which additional right-of-way will be needed, such as at offsite inlets or outlets.

7. All possible storm-drain outlet locations should be identified and investigated. Outlet locations along the length of the storm drain could be used to reduce the cost of the storm drain. All locations where outlets are contemplated should be studied to determine governing tailwater elevations, and possible erosion and right-of-way concerns.

8. As-built drawings of all streets and highways affecting the project should be obtained and thoroughly reviewed.

10.11 Check List for Design Submittals

The following check list is provided as an aid to the in-house, quality-control reviewer of storm-drain design reports and plans. This check list should be used as a guide to the minimum items necessary for storm-drain design. It is the duty of the in-house, quality-control reviewer to ensure that any additional information that is necessary for the specific storm-drain design be identified and provided, and that all such information be technically accurate and correct. Included within, or on, any reports/plans should be:

1. An overall map of the project site showing the storm drain, streets, drainageways, and land use.

2. Offsite watershed map, at a scale no smaller than 1″ = 200′, with at least two-foot contour intervals superimposed on aerial topography. All concentration points and watershed definitions shall be clearly shown on these maps.

3. All offsite hydrologic data sheets and supporting information for offsite drainage calculations.

4. Site information for offsite drainage concentration points. This should include hydraulic information and calculations at the points where offsite drainage is to be intercepted. Two main points to consider are:

   a) Is the proposed inlet really adequate to intercept all of the offsite flow it is designed to intercept; and,

   b) Is there enough information provided to determine what impact flow into this inlet will have upon the downstream storm-drain system?

5. Proposed street cross sections, grades, and compositions. These should be in the form of plan-and-profile sheets, with street cross sections shown.

6. Calculations showing street capacities at the allowable design limit of flow spread along the entire length of the storm drain. Points where drainage must be entirely (i.e., 100 percent) removed from the street should also be located.

7. Inlet calculations, according to the procedure outlined in Section 10.8 (see Example 10.2) or by use of a similar method. All hydrologic data sheets used in this step should be included, as should assumptions, supporting calculations, and proposed inlet types and sizes. Also included should be detailed watershed maps showing drainage areas, land use, and topography. These maps should be of a more detailed scale than 1″ = 200′. A scale no smaller than 1″ = 100′ is mandated. A scale of 1″ = 40′ is preferable, where appropriate.

8. All hydrologic data sheets and supporting calculations for discharges used in conduit sizing. Additional watershed maps should be provided, if necessary, in order to show larger watershed delineations.

9. When a storm drain is designed for pressure flow, the intermediate calculations for pipe size need not be included. However, final pressure-flow calculations must be included. These shall include hydraulic grade-line calculations, as described in Sections 10.9 and 10.10 (see Example 10.3), or calculations which employ a similar procedure. There should be a storm-drain profile, drawn to scale; and showing, at a minimum, the storm-drain soffit and invert, ground surface, hydraulic grade line, energy grade line, outlet control elevation, manholes, junctions, transitions, bends, and inlets. A plan-view map should show the storm drain, adjacent streets, and proposed inlets and manholes. Calculations of minor losses should be provided, and should include a clearly labeled diagram of the structure, or structures,

involved. Calculations and supporting ground information showing how the controlling water-surface elevation was determined are required.

10. There should be a clear, concise, text description of the design process, and the assumptions upon which the analysis was based. The drainage report should follow the City of Tucson guidelines for drainage-report submittals.

EXAMPLE 10.1: STREET FLOW IN A PARABOLIC SECTION

Compute the discharge for a parabolic, crowned, street section with a longitudinal slope of one percent, in which the depth of flow at the curb is 0.48 feet.

The parabolic section has the properties shown in Figure 10.17.

Using Equation 10.5, and an incremental segment width of two feet, the table entitled "Computations for Example 10.1," found on page 10.47 of this Manual, is produced.

EXAMPLE 10.2: STORM-DRAIN INLET SPACING AND SIZING

In this example, it is assumed that a storm drain will be built along 22nd Street, between Beverly Avenue and Mountain View Avenue. The design criterion is to maintain two of the three lanes of traffic open to flow during the 10-year runoff event. For simplicity, only drainage south of the median island will be taken into account. The storm drain will discharge into the Naylor Wash at Mountain View Avenue.

This example is chosen to illustrate the design procedure for storm-drain inlets, as well as to demonstrate both the hydraulic and the economic impact of using a depressed gutter. For these reasons, no gutter depression is assumed for this example. However, at the end of this example, a comparison will be made of the assumed design versus a design with a depressed gutter.

PROCEDURE

1. The offsite watershed draining onto this street is generally rectangular in shape, with Naylor Wash as the south boundary (See Figure 10.18). Although Naylor Wash overflows into 22nd Street during a 100-year flood, all rainfall falling north of the wash in this area flows into 22nd street. Development along the watershed is heavy commercial, with an anticipated impervious cover of 90 percent in the future. There is assumed to be no inflow from upstream of Rosemont Blvd. The watershed slope, which is the same as the street slope, is one percent. The runoff coefficient is 0.95, and the rainfall intensity for a time of concentration, Tc, equal to five minutes is 9.6 inches/hour.

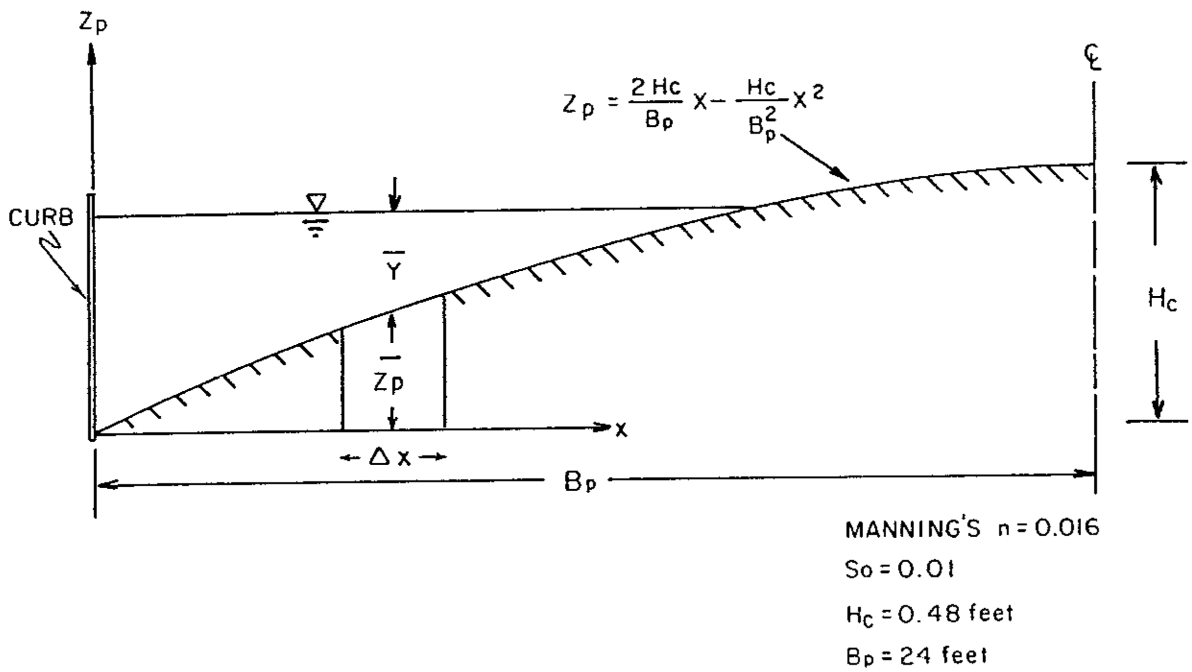


FIGURE 10.17
PARABOLIC STREET SECTION FOR EXAMPLE 10.1

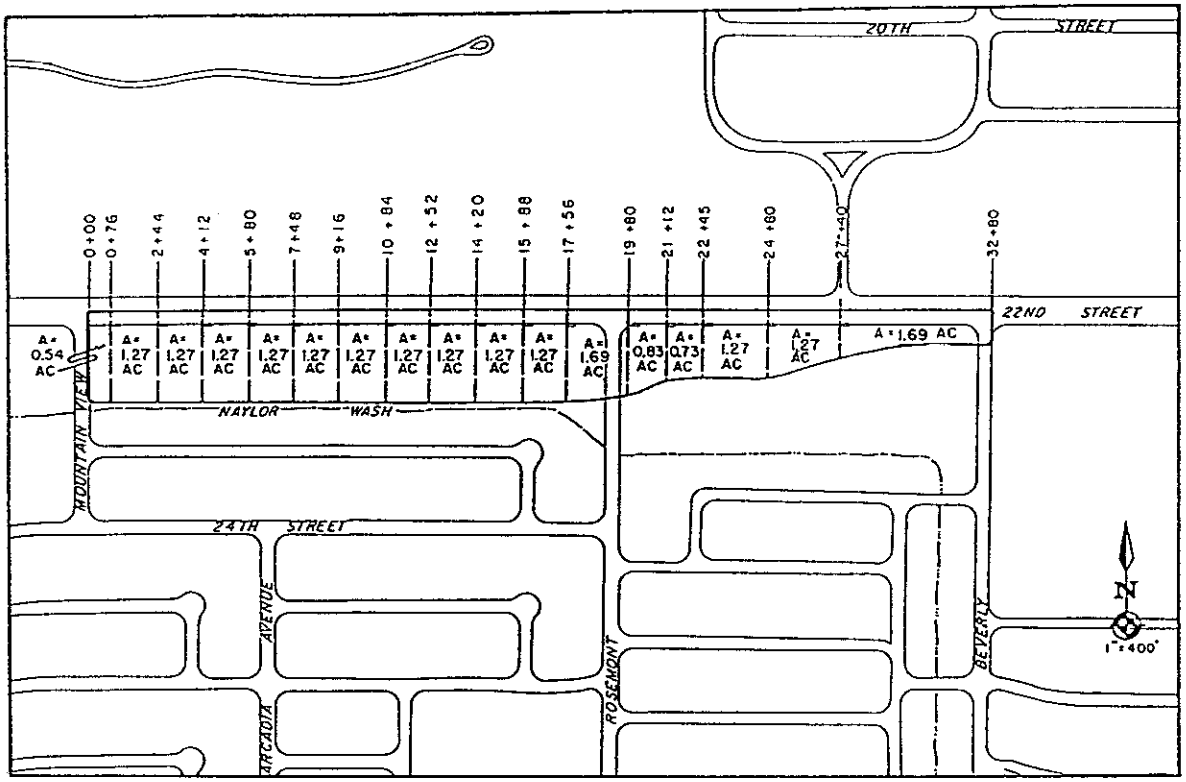
Computations for Example 10.1

$\underline{X}$ DISTANCE FROM CURB (ft)	$\underline{Z_p}$ VERTICAL RISE (ft)	$\underline{Y}$ DEPTH OF FLOW (ft)	$\underline{\Delta x}$ WIDTH OF SECTION (ft)	$\underline{\bar{Y}}$ MEAN DEPTH (ft)	$\underline{A}$ AREA OF SECTION (ft^2)	$\underline{K^*}$ CONVEYANCE FACTOR
0	0	0.480				
			2	0.442	0.884	47.64
2	0.077	0.403				
			2	0.368	0.736	35.10
4	0.147	0.333				
			2	0.302	0.604	25.25
6	0.210	0.270				
			2	0.242	0.484	17.46
8	0.267	0.213				
			2	0.188	0.376	11.46
10	0.317	0.163				
			2	0.142	0.284	7.18
12	0.360	0.120				
			2	0.102	0.204	4.14
14	0.397	0.083				
			2	0.068	0.136	2.10
16	0.427	0.053				
			2	0.042	0.084	0.94
18	0.450	0.030				
			2	0.022	0.044	0.32
20	0.467	0.013				
			2	0.008	0.016	0.06
22	0.477	0.003				
			2	0.002	0.004	0.01
24	0.480	0	$\Sigma K = 151.66$			

$*K = (1.486/n)(A/P)^{2/3}A$

From which is obtained:

$$Q = \Sigma K S^{1/2} = (151.66)(0.01)^{1/2} = 15.17 \text{ cfs}$$



EXPLANATION

- WATERSHED BOUNDARY
- SUBWATERSHED BOUNDARY
- CURB INLET

FIGURE 10.18
WATERSHED BOUNDARIES AND INLET SPACING FOR EXAMPLE 10.2

2. The assumed street cross section (not the real one) is shown in Figure 10.19. It consists of three lanes, with no gutter present. The maximum allowable spread of water is twelve feet, at a depth of 0.36 feet at the curb. (i.e., 4+ inches).

   Using Manning's "*n*" = 0.016, the maximum allowable discharge, Q_d, is computed using Equation 10.2:

$$Q_d = 0.56\left(\frac{(0.03)^{5/3}(12.0)^{8/3}(0.010)^{1/2}}{0.016}\right)$$

   Hence,

$$Q_d = \underline{\underline{7.7\ cfs}}.$$

3. For impervious cover = 90 percent,

$$Q_{100}/Q_{10} = 2.0$$

   Therefore,

$$Q_{100} = 2.0(7.7\ cfs) = \underline{\underline{15.4\ cfs.}}$$

   Using the Rational Formula, $Q = CiA$,

$$A = \frac{15.4}{(0.95)(9.6)}; \quad A = \underline{\underline{1.69\ acres.}}$$

   Place the first catch basin at a point where the watershed area = 1.69 acres (See Figure 10.18). The watershed area = 1.69 acres at STA 27 + 40.

4. Use a curb inlet.

   Equations 10.22 and 10.23 give the required curb-inlet length in order to intercept 75 percent of the flow.

$$L_i/L_t = 1-(1-0.75)^{0.556} = \underline{\underline{0.54}}.$$

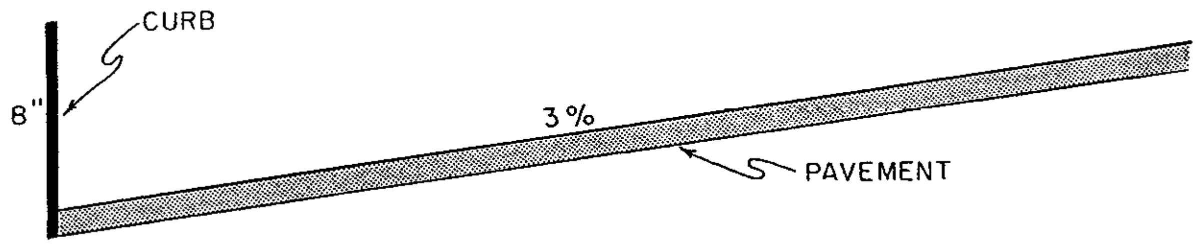


FIGURE 10.19
ASSUMED STREET CROSS SECTION FOR EXAMPLE 10.2

$$L_t = 0.6(7.7)^{0.42}(0.01)^{0.3}\left(\frac{1}{.016(.03)}\right)^{0.6} = \underline{\underline{34.8\ feet.}}$$

$$L_i = 0.54(34.8) = \underline{\underline{19\ feet.}}$$

Use a 20-foot-long standard ADOT catch basin, Type 3.

Carryover flow is: 7.7 cfs - 0.75(7.7 cfs) = $\underline{\underline{1.9\ cfs.}}$

5. The next downstream watershed, considering carryover, must generate:

$$0.75(7.7 \text{ cfs}) = 5.8 \text{ cfs}$$

$$Q_{100} = 2.0(5.8 \text{ cfs}) = \underline{\underline{11.6\ cfs.}}$$

Therefore,

$$A = \frac{11.6}{(0.95)(9.6)} = \underline{\underline{1.27\ acres.}}$$

A watershed area equal to 1.25 acres is reached at Station 24+80. Use another catch basin of the same size at this location.

6. Following the same procedure, another catch basin is needed at Station 22+45. However, looking ahead, a catch basin with 100-percent efficiency will be needed at Station 19+80, because of the intersection. Unfortunately, the intervening watershed is large enough to produce a flow spread beyond the allowable, unless proportionately more flow is collected. Therefore, install another standard catch basin at Station 21+12.

The watershed area at Station 21+12 is 0.73 acres.

$$Q_{100} = 0.73(0.95)(9.6) = \underline{\underline{6.7\ cfs.}}$$

$$Q_{10} = 0.5(6.7) = \underline{\underline{3.4 \text{ cfs.}}}$$

Adding carryover: $Q = 3.4+1.9 = \underline{\underline{5.3}}$ cfs.

The length of curb inlet required to intercept 5.3 cfs is:

$$L_t = 0.6\,(5.3)^{0.42}\,(0.01)^{0.3}\left[\frac{1}{0.016(0.03)}\right]^{0.6} = \underline{\underline{30\ feet.}}$$

The efficiency of a 20-foot-long curb inlet is:

$$E_i = 1 - \left[1 - \frac{20}{30}\right]^{1.8} = \underline{\underline{0.86.}}$$

Curb-inlet capacity is:

$0.86(5.3$ cfs$) = \underline{\underline{4.6\ cfs.}}$

Carryover flow is:

5.3 cfs $-$ 4.6 cfs $= \underline{\underline{0.7\ cfs.}}$

7. The watershed area at Station 19+80 is 0.83 acres.

$Q_{100} = 0.83(0.95)(9.6) = \underline{\underline{7.6\ cfs.}}$

$Q_{10} = \underline{\underline{3.8\ cfs.}}$

Adding carryover, $Q = 3.8 + 0.7 = \underline{\underline{4.5\ cfs.}}$

The curb-inlet length required to collect 100 percent of this flow is:

$$L_t = \frac{0.6(4.5)^{0.42}(0.01)^{0.3}}{[(0.016(0.03)]^{0.6}} = \underline{\underline{28\ feet.}}$$

Use a 20-foot-long standard ADOT catch basin, Type 3, with a 10-foot-long extension.

8. The watershed downstream of Station 19+80 is more or less uniform in width (329 ft). As before, the area required to generate the maximum allowable discharge of 7.7 cfs is 1.69 acres.

   Therefore, the distance to the next catch basin is:

$$\frac{1.69\ ac\ (43560\ ft^2/ac)}{329\ \text{ft}} = \underline{\underline{224\ feet.}}$$

   Accordingly a standard ADOT catch basin, Type 3, is placed at Station 19+80, minus 224 ft., which is Station 17+56. Catch basins are then placed at intervals of 168 feet, in order to collect intervening watersheds of 1.27 acres in size.

9. At Mountain View Ave., STA 0+00, 100 percent of the flow must be collected. The watershed area downstream of STA 0+76 is 0.57 acres.

   The 100-year peak discharge at this location is:

$$Q_{100} = 0.57(0.95)(9.6) = \underline{\underline{5.2\ cfs.}}$$

   Therefore, the 10-year peak discharge is:

$$Q_{10} = 0.5(5.2) = \underline{\underline{2.6\ cfs.}}$$

   The design discharge at Station 0+00 is Q_{10}, plus the carryover discharge:

$$2.6\ \text{cfs} + 1.9\ \text{cfs} = \underline{\underline{4.5\ cfs,}}\ \text{at station 0+00.}$$

   The length of curb inlet required is:

$$L_t = \frac{0.6(4.5)^{0.42}(0.01)^{0.3}}{[0.016(0.03)]^{0.6}} = \underline{\underline{28\ feet.}}$$

Use a 20-foot-long standard ADOT catch basin, Type 3, with a 10-foot-long extension.

The pavement-drainage worksheets for Example 10.2 can be found on the following three pages of this Manual.

- -

NOTE: In practice, it is probably cheaper to run the flow in the storm drain at Station 19+80 down Rosemont; then directly into the Naylor Wash, instead of continuing down 22nd St. A second storm drain would then begin at station 17+52.

The design criterion of one-lane maximum spread of water was chosen for illustrative purposes only. The normal requirement of leaving one lane open would allow the spread of water to extend across two lanes.

Normally, a clogging factor would be used to account for debris. Therefore, *the size of the curb inlets computed for this example should be increased by a factor of 1.25, for design purposes, in order to account for the possibility of clogging.*

Now, if a standard 1.75-foot-wide, one-inch-deep concrete gutter were added to the edge of the roadway, the street capacity would increase to 11.1 cfs. Therefore, the curb inlets could be spaced farther apart, with a total reduction in their number of about one-third. The number of curb inlets could be reduced even further by using a two-percent pavement cross-slope, and allowing water to spread across two of the three traffic lanes. This would reduce the number of required curb inlets to only about one-fifth of those shown in this example. However, the length of each curb inlet would then have to be doubled.

The purpose of presenting the preceding two scenarios is merely to demonstrate to the designer that one should always investigate several alternatives in order to develop the most hydraulically efficient, economical inlet design practicable.

Sheet 1 Of 3

PAVEMENT-DRAINAGE WORKSHEET FOR EXAMPLE 10.2

LOCATION DATA: 22nd St. STORM DRAIN

DESIGN DATA:

Frequency 10 YEARS

Location: Beverly to Mountain View

Project No: Example 10.2

T_{all} = 12 feet n = 0.016

Curb Height = 0.67 feet

RUNOFF CALCULATIONS:

From Station	To Station	L ft.	W ft.	D.A. acre,	C	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}
32 + 80	27 + 40	540	136	1.69	0.95	5	9.6	15.4	7.7	0	7.7	0.01	0.03	0.36	12
27 + 40	24 + 80	260	213	1.27	0.95	5	9.6	11.6	5.8	1.9	7.7	0.01	0.03	0.36	12
24 + 80	22 + 45	235	235	1.27	0.95	5	9.6	11.6	5.8	1.9	7.7	0.01	0.03	0.36	12
22 + 45	21 + 12	133	239	0.73	0.95	5	9.6	6.7	3.4	1.9	5.3	0.01	0.03	0.31	10.3
21 + 12	19 + 80	132	274	0.83	0.95	5	9.6	7.6	3.8	0.7	4.5	0.01	0.03	0.30	10.0

INLET CALCULATIONS

Station	Inlet Type	Y ft.	Q_i cfs	$Q_{co} = Q_T - Q_i$
27 + 40	ADOT TYPE-3 CATCH BASIN	0.36	5.8	1.9
27 + 80	ADOT TYPE-3 CATCH BASIN	0.36	5.8	1.9
22 + 45	ADOT TYPE-3 CATCH BASIN	0.36	5.8	1.9
21 + 12	ADOT TYPE-3 CATCH BASIN	0.31	4.6	0.7
19 + 80	ADOT TYPE 3 WITH 10' EXT.	0.30	4.5	0

Computed by:__________ Checked by:__________

Sheet 2 Of 3

PAVEMENT-DRAINAGE WORKSHEET FOR EXAMPLE 10.2 (continued)

LOCATION DATA: 22nd St. STORM DRAIN

Location: Beverly to Mountain View

Project No: Example 10.2

DESIGN DATA:

Frequency 10 YEARS

T_{all} = 12 feet n = 0.016

Curb Height = 0.67 feet

RUNOFF CALCULATIONS:

From Station	To Station	L ft.	W ft.	D.A. acre,	C	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}
19 + 80	17 + 56	224	329	1.69	0.95	5	9.6	15.4	7.7	0	7.7	0.01	0.03	0.36	12
17 + 56	15 + 88	168	329	1.27	0.95	5	9.6	11.6	5.8	1.9	7.7	0.01	0.03	0.36	12
15 + 88	14 + 20	168	329	1.27	0.95	5	9.6	11.6	5.8	1.9	7.7	0.01	0.03	0.36	12
14 + 20	12 + 52	168	329	1.27	0.95	5	9.6	11.6	5.8	1.9	7.7	0.01	0.03	0.36	12
12 + 52	10 + 84	168	329	1.27	0.95	5	9.6	11.6	5.8	1.9	7.7	0.01	0.03	0.36	12
10 + 84	9 + 16	168	329	1.27	0.95	5	9.6	11.6	5.8	1.9	7.7	0.01	0.03	0.36	12
9 + 16	7 + 48	168	329	1.27	0.95	5	9.6	11.6	5.8	1.9	7.7	0.01	0.03	0.36	12

INLET CALCULATIONS

Station	Inlet Type	Y ft.	Q_i cfs	$Q_{co} = Q_T - Q_i$
17 + 56	SINGLE ADOT TYPE 3	0.36	5.8	1.9
15 + 88	SINGLE ADOT TYPE 3	0.36	5.8	1.9
14 + 20	SINGLE ADOT TYPE 3	0.36	5.8	1.9
12 + 52	SINGLE ADOT TYPE 3	0.36	5.8	1.9
10 + 84	SINGLE ADOT TYPE 3	0.36	5.8	1.9
9 + 16	SINGLE ADOT TYPE 3	0.36	5.8	1.9
7 + 48	SINGLE ADOT TYPE 3	0.36	5.8	1.9

Computed by:____________ Checked by:____________

Sheet 3 Of 3

PAVEMENT-DRAINAGE WORKSHEET FOR EXAMPLE 10.2 (continued)

LOCATION DATA: 22nd St. STORM DRAIN

Location: Beverly to Mountain View

Project No: Example 10.2

DESIGN DATA:

Frequency 10 YEARS

T_{all} = 12 feet n = 0.016

Curb Height = 0.67 feet

RUNOFF CALCULATIONS:

From Station	To Station	L ft.	W ft.	D.A. acre,	C	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}
7 + 48	5 + 80	168	329	1.27	0.95	5	9.6	11.6	5.8	1.9	7.7	0.01	0.03	0.36	12
5 + 80	4 + 12	168	329	1.27	0.95	5	9.6	11.6	5.8	1.9	7.7	0.01	0.03	0.36	12
4 + 12	2 + 44	168	329	1.27	0.95	5	9.6	11.6	5.8	1.9	7.7	0.01	0.03	0.36	12
2 + 44	0 + 76	168	329	1.27	0.95	5	9.6	11.6	5.8	1.9	7.7	0.01	0.03	0.36	12
0 + 76	0 + 00	76	329	0.57	0.95	5	9.6	5.2	2.6	1.9	4.5	0.01	0.03	0.29	9.7

INLET CALCULATIONS

Station	Inlet Type	Y ft.	Q_i cfs	$Q_{co} = Q_T - Q_i$
5 + 80	SINGLE ADOT TYPE 3	0.36	5.8	1.9
4 + 12	SINGLE ADOT TYPE 3	0.36	5.8	1.9
2 + 44	SINGLE ADOT TYPE 3	0.36	5.8	1.9
0 + 76	SINGLE ADOT TYPE 3	0.36	5.8	1.9
0 + 00	ADOT TYPE 3 WITH 10' EXT.	0.29	4.5	0

Computed by:____________ Checked by:____________

EXAMPLE 10.3: HYDRAULIC GRADE-LINE COMPUTATION

Figures 10.20 and 10.21 show the plan-and-profile information for the storm drain to be analyzed. This information, along with preliminary information about inlet locations and sizes, manhole locations and sizes, transition and junction locations and sizes, pipe lengths and sizes, and design discharges, must be available before the hydraulic grade-line computations can proceed.

Corrugated metal pipe will be used, so Manning's "n" is assumed equal to 0.024. The following conditions apply at the outlet:

Design Discharge = 145 cubic feet per second;

Invert of pipe elevation = 95.0 feet;

Diameter of pipe = 5.5 feet;

Hydraulic grade elevation = 100.50 feet;

Area of pipe = 23.76 square feet;

Velocity of flow = 6.1 feet per second; and,

Hydraulic radius = 1.38 feet.

Using Equation 10.29, Compute $S_f = 0.0064$ feet/foot.

The elevation of the energy grade line at the outlet is $HG+V^2/2g = 100.50 + 0.58$ = 101.08 feet. At station 1+00, friction loss, S_f, = $100(0.0064)$ = 0.64 feet. The energy grade-line elevation at station 1+00 is $101.08 + 0.64$ = 101.72 feet. Computations proceed upstream in this manner. "Minor" losses are added in, as encountered, as illustrated on the hydraulic grade-line calculation sheet, and as follows:

1. Station 1+50: *bend loss*
   Radius of curvature, r, = 54 feet,
   Pipe diameter, D, = 5.5 feet,
   r/D = 9.8.

   From Figure 10.13, K_b for 90° bend = $\underline{\underline{0.39.}}$ Angle of deflection, θ, = *30°*.

   From Equation 10.32:

$$K_b = 0.39 \left\{ 1 - \left[\frac{90-30}{90} \right]^2 \right\} = \underline{\underline{0.22.}}$$

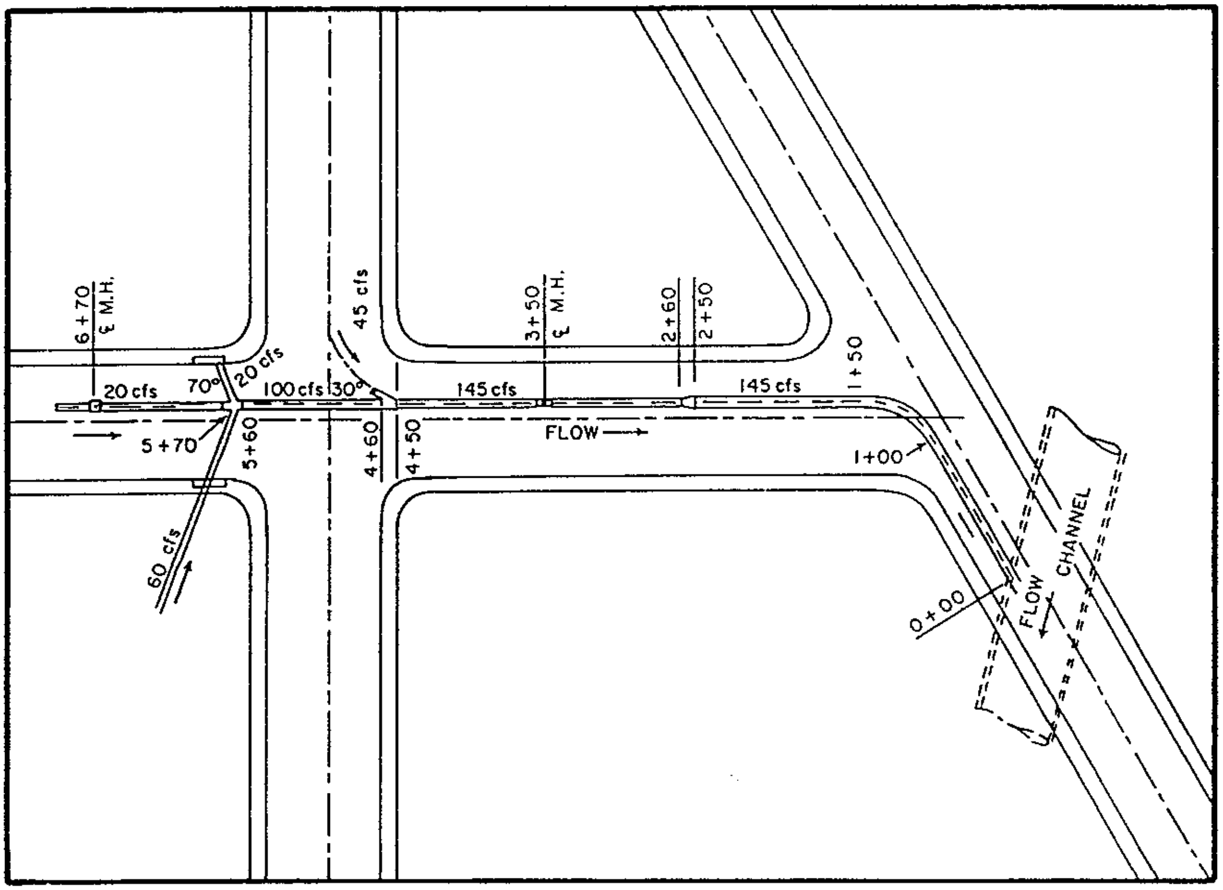


FIGURE 10.20
PLAN VIEW OF STORM DRAIN FOR EXAMPLE 10.3

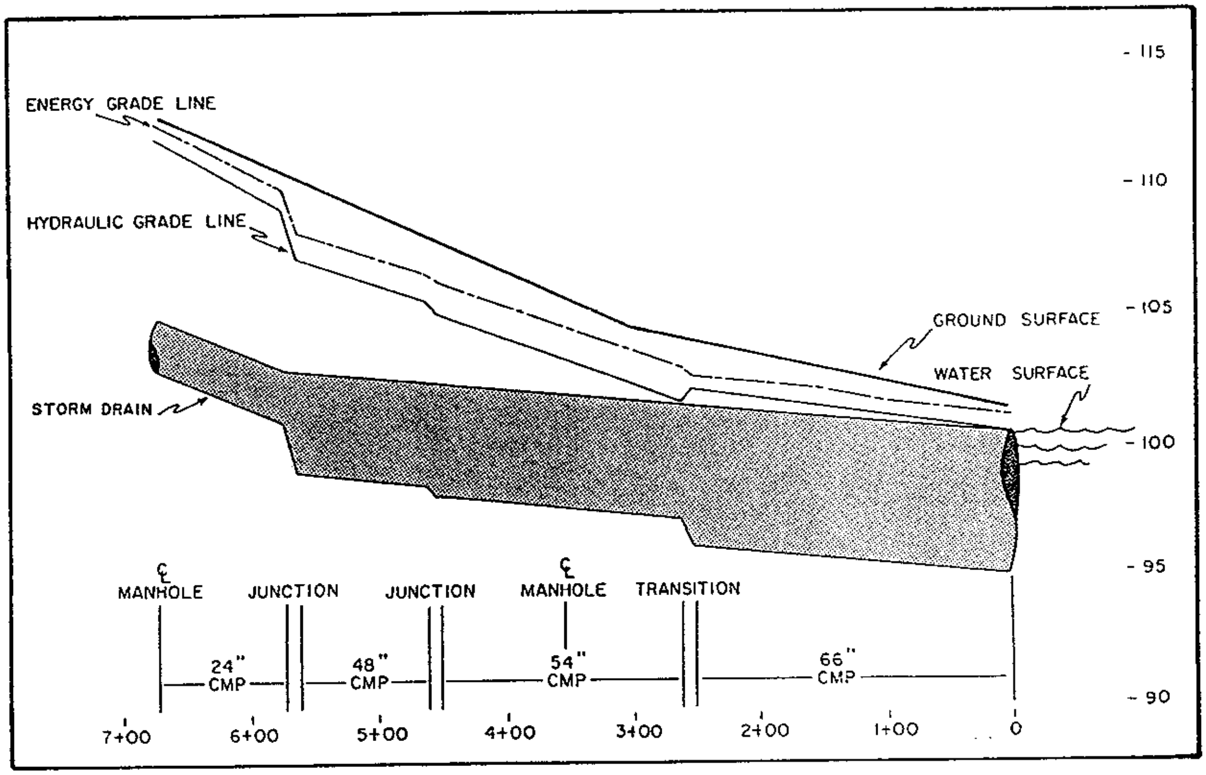


FIGURE 10.21
PROFILE OF STORM DRAIN FOR EXAMPLE 10.3

$$h_b = 0.22 \frac{V^2}{2g} = 0.22\ (0.58) = \underline{\underline{0.13\ feet.}}$$

2. Station 2+60: *transition loss*

   From Equation 10.38:

$$h_t = 0.2(1.29 - 0.58) = \underline{\underline{0.14\ feet.}}$$

3. Station 3+50: *manhole loss*

   From Equation 10.39:

$$h_{mh} = 0.05\left(\frac{V^2}{2g}\right) = 0.05\ (1.3) = \underline{\underline{0.06\ feet.}}$$

4. Station 4+60: *junction loss* (See Figure 10.22)

   Use Equations 10.35 and 10.36. Then:

Q_1 = 100 cfs	Q_2 = 145 cfs	Q_3 = 45 cfs
A_1 = 12.57 ft^2	A_2 = 15.90 ft^2	A_3 = 5.00 ft^2
V_1 = 8.0 cfs	V_2 = 9.1 cfs	V_3 = 9.0 cfs
θ = 30°		

   From which:

$$h_j = \underline{\underline{0.08\ feet.}}$$

5. Station 5+70: *junction loss* (See Figure 10.23)

   As with Step 4, above:

Q_1 = 20 cfs	Q_2 = 100 cfs
A_1 = 3.14 ft^2	A_2 = 12.57 ft^2

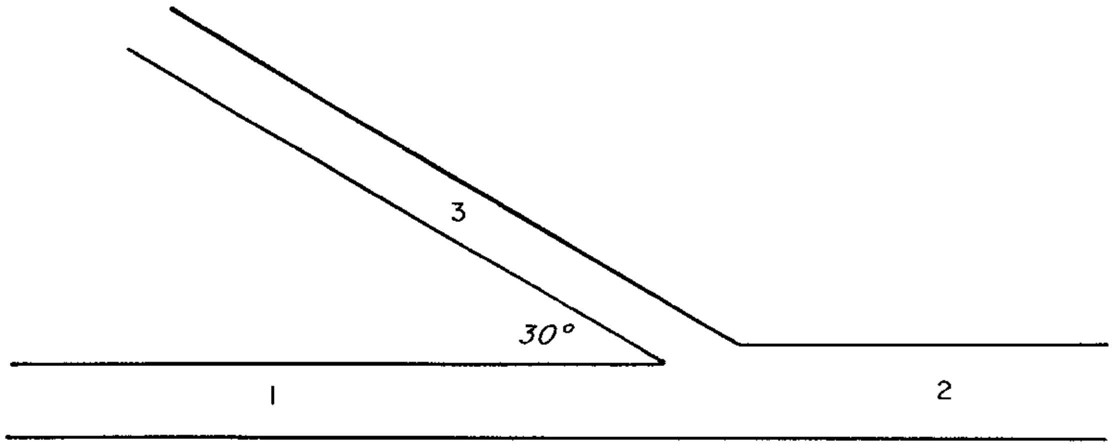


FIGURE 10.22
STATION 4+60: JUNCTION FOR EXAMPLE 10.3

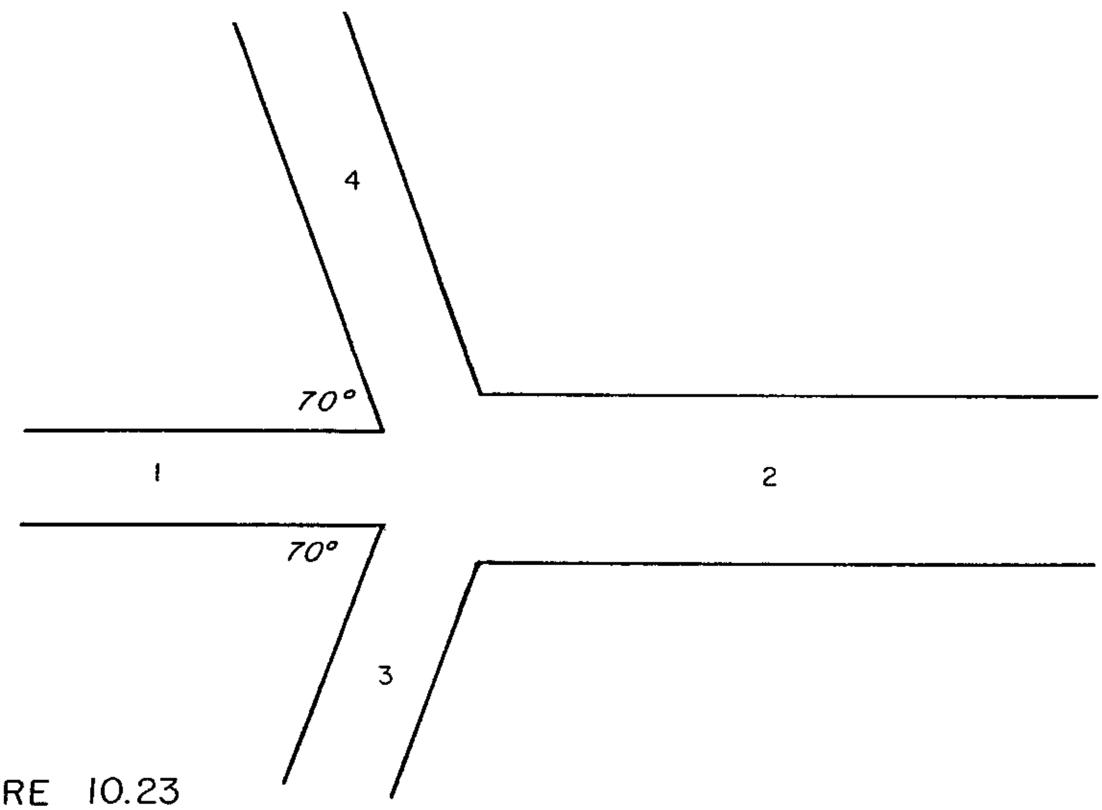


FIGURE 10.23
STATION 5+70: JUNCTION FOR EXAMPLE 10.3

$V_1 = 6.4$ fps		V_2	$= 8.0$ fps
$Q_3 = 60$ cfs		Q_4	$= 20$ cfs
$A_3 = 7.07\ ft^2$		A_4	$= 3.14\ ft^2$
$V_3 = 8.5$ fps		V_4	$= 6.4$ fps
$\theta_3 = 70°$		θ_4	$= 70°$

Now, use $\Delta HG = \left(\frac{2}{A_1+A_2}\right)\frac{Q_2V_2-Q_1V_1-Q_3V_3Cos\theta_3-Q_4V_4Cos\theta_4}{g}$

Then:

$$h_j = \Delta HG + \frac{V_1^2}{2g} - \frac{V_2^2}{2g}$$

From which:

$\Delta HG = \underline{\underline{1.79\ feet}}$ and $h_j = \underline{\underline{1.43\ feet.}}$

6. Station 6+70: *manhole loss*

Again, from Equation 10.39:

$$h_{mh} = 0.05\left(\frac{V^2}{2g}\right) = 0.05\ (0.64) = \underline{\underline{0.03\ feet.}}$$

NOTE: Using normal design procedures, the roughness coefficient would be increased by fifteen percent in order to account for aging effects upon the pipe. The roughness value used would then be 0.028, instead of 0.024.

Figure 10.24 on the following page is the hydraulic grade-line calculation sheet for this example.

HYDRAULIC GRADE–LINE CALCULATION SHEET

Sheet ______ of ______

PROJECT: ______________________ CALCULATED BY: ______________

LINE: ______________________ DATE: ______________

STA.	ELEV. INV.	D	ELEV. H.G.L.	SEC-TION	D_{hg}*	A	Q	V	$\frac{V^2}{2g}$	S_f	AVG S_f	L	h_f	h_b	h_j	h_t	h_{mh}	ELEV. E.G.L.
0+00	95.00	5.5	100.50	CIR	5.50	23.76	145	6.1	0.58	.0064								101.08
1+00	95.50	5.5	101.14	CIR	5.64	23.76	145	6.1	0.58	.0064	.0064	100	0.64					101.72
1+50	95.75	5.5	101.59	CIR	5.84	23.76	145	6.1	0.58	.0064	.0064	50	0.32	0.13				102.17
2+50	96.25	5.5	102.23	CIR	5.98	23.76	145	6.1	0.58	.0064	.0064	100	0.64					102.81
2+60	97.25	4.5	101.78	CIR	4.53	15.90	145	9.1	1.29	.0185	.0125	10	0.12			0.14		103.07
3+50	97.70	4.5	103.51	CIR	5.81	15.90	145	9.1	1.29	.0185	.0185	90	1.67				0.06	104.80
4+50	98.20	4.5	105.36	CIR	7.16	15.90	145	9.1	1.29	.0185	.0185	100	1.85					106.65
4+60	98.70	4.0	105.92	CIR	7.22	12.57	100	8.0	0.99	.0167	.0176	10	0.18		0.08			106.91
5+60	99.20	4.0	107.59	CIR	8.39	12.57	100	8.0	0.99	.0167	.0167	100	1.67					108.58
5+70	101.20	2.0	109.59	CIR	8.39	3.14	20	6.4	0.64	.0269	.0218	10	0.22		1.43			110.23
6+70	103.20	2.0	112.31	CIR	9.11	3.14	20	6.4	0.64	.0269	.0269	100	2.69				0.03	112.95

*Conduit unseals when D_{hg} is less than D.

FIGURE 10.24
HYDRAULIC GRADE–LINE CALCULATIONS FOR EXAMPLE 10.3