BAS

Basin Analysis Software

USER'S MANUAL

Nadeem H. Majaj, P.E., Developer/Editor

March 2022

TABLE OF CONTENTS

NOTICE

The End User License Agreement, which is included with this software, specifies the permitted and prohibited uses of this software. Any unauthorized duplication or use of Basin Analysis Software (BAS), in whole or in part, in print or in any storage and retrieval system, is forbidden.

DISCLAIMER

The BAS environment and its documentation has been released by Nadeem Majaj as a proprietary model and, as such is not available to unauthorized users. Nadeem Majaj took every care to provide error free code; however, due to the complexity and nature of this type of software, the author/Developer cannot make explicit warranties as to the documentation, function or performance of the software. Should any errors be found during program operation, the user should direct the problem to the Developer where every effort will be expended to quickly resolve the problem. Although the data checking facilities of the model are extensive, incorrect results may be produced if poor or inappropriate data is entered.

Users are expected to make final evaluation as to the usefulness of this model for their purposes. He/she must use his/her own engineering knowledge, judgment and experience as to the applicability of the model to the job at hand and perform reasonable and independent engineering cross-checks on the data and results. The Developer cannot assume responsibility for software operation, results or interpretation of results.

FORWARD

This software was developed by Nadeem H. Majaj, P.E. for Hydraulic Solutions Incorporated in order to reduce the complexity of detailed design and analyses of side-weirs and offline retarding basins. The software was developed within the Microsoft Excel and Microsoft Visual Basic for Applications (VBA) environments. Nadeem H. Majaj also authored this user's manual.

A word of caution is warranted regarding the use of any engineering software. Internal to all engineering software, the Developer, at times, must resort to approximations that overcome certain mathematical (and hydraulic) instabilities. The development of BAS includes such approximations in certain situations. One such example is when the water surface in a basin rises and approximately equals that of the channel. A small amount of weir flow may result in a higher water surface in the basin and the need to spill back into the channel. These conflicting conditions result in instabilities. BAS includes minor programming routines that overcome instabilities by forcing the calculations either into or out of the basin for a very small time increment. This is one example and others also exist. The overall effect on the resulting volumes and downstream channel flows is insignificant.

In addition, the user should be aware that although BAS includes several methods for the calculation of a side-weir's coefficient, he/she should perform his/her own research for applicability to the design or analysis of their specific system.

HOW TO USE THIS MANUAL

This manual was intended to provide the user with all the pertinent information for the successful operation of BAS. The theory behind the various analyses within BAS is too voluminous to be included herein and is referred to in the reference section at the end of the manual.

In order to simplify the BAS documentation, references to the menu commands are **bolded** while references to BAS buttons are boxed.

Chapter 1 – INTRODUCTION

This chapter presents the General Philosophy of BAS, End User Agreement, System Requirements and Installation Procedures.

GENERAL PHILOSOPHY OF BAS

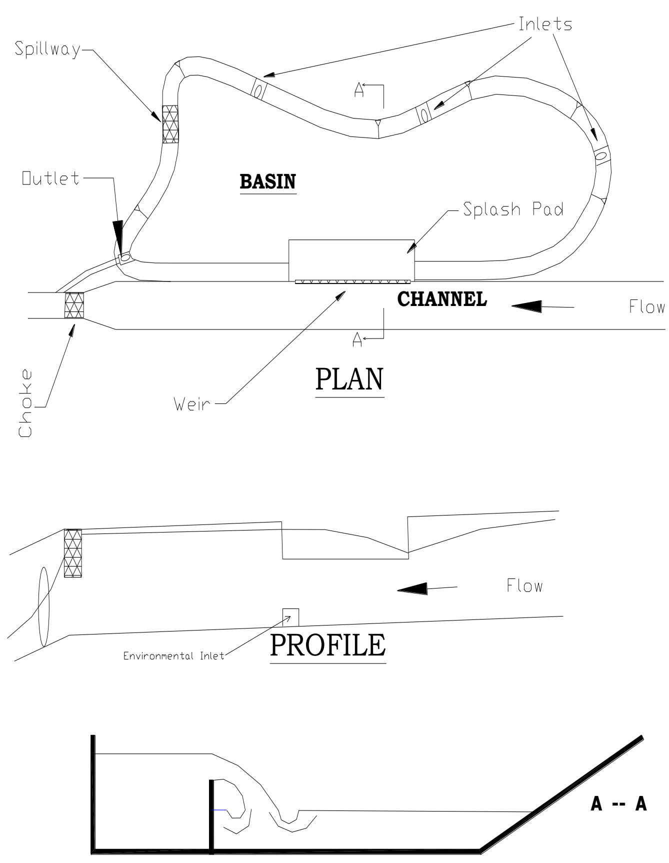
Limited funds, limited right-of-way and deficient flood control channels have encouraged public agencies to consider offline retarding basins as alternatives to expensive all-channel systems or online basin systems. Retarding is accomplished by diversion of the higher stages of a channel's hydrograph flows to a basin by means of a side-weir. Presently, analyses of side-weirs as diversion structures range from utilizing the basic weir equation to a spatially-varied-flow function with a varying coefficient of discharge. Computer models developed in the mid 1970's to analyze such problems greatly improved the design of side-weirs but omitted important hydraulic interactions between the basin, channel, storm drain inlets, emergency spillway and basin outlet that may significantly affect the basin volume required. In order to account for such interactions, hydraulic practitioners included many simplifications and assumptions that enabled them to complete the analyses. Despite these simplifications, the analyses absorbed weeks and often months for each design scenario. In most cases, the systems were thought to have been designed "conservatively" by over-sizing the basin.

The Basin Analysis Software (BAS) is a new hydraulic software tool developed for a complete design, analysis and optimization of **sharp crested side-weirs** and offline retarding basins. Due to the unavailability of reliable data on supercritical side-weirs, BAS was developed to solve **only cases of subcritical prismatic** channels along side-weirs.

BAS, which includes **Weir Design**, **Weir Analysis** and **Basin Analysis** program components, was developed to greatly reduce simplifications, which typically result in large sized structures but not necessarily greater flood protection. The essential roles of the basin's weir and outlet(s) in the dynamic interactions between the channel and the basin are included in BAS as major components of the analyses. The outlet analyses consider pressure and open channel flows either into or out of the basin. The figure below represents a typical offline system with its main hydraulic appurtenances.

While the main purpose behind the development of BAS is to simulate the real-life behavior of an offline system, ease of use including an abundance of visual forms and input media, was the second goal set during its development. Microsoft Excel spreadsheets are utilized for the storage and presentation of the resulting data. In addition, portions of the calculations are performed by the use of Microsoft's Visual Basic for Applications (VBA) programming language.

The state-of-the-art features of Excel provide programming power and user-friendliness unsurpassed in today's hydrology/hydraulic software. In essence, the Developer endeavored to reduce a very complex spatially-varied-flow system into a user-friendly set of forms and tables that enable a hydraulic engineer to easily operate this software. To facilitate the user's understanding of the modeling of an offline retarding system, the Developer created two main examples. One of the two examples is an actual basin that was constructed in 1999 in Orange County, California. The other is a basin that was created to compare an online with an offline system. Discussion of this comparison will be presented in future technical papers prepared by the author.



END USER AGREEMENT

LICENSE GRANT

Software means **Basin Analysis Software**, the installation program, example files and all other files provided with this product. **You** means the person, company or public agency who is being licensed to use the Software or Documentation. **We, us and our** means Nadeem Majaj.

We hereby grant you a nonexclusive license to use one copy of the Software on any single computer, provided the Software is in use on only one computer at any time. The Software is in use on a computer when it is loaded into temporary memory (RAM) or installed into the permanent memory of a computer--for example, a hard disk, CD-ROM or other storage device.

The Software is protected by United States copyright laws and international treaty provisions and is governed by the State of California. Therefore, you must treat the Software just like any other copyrighted material (e.g. a book or musical recording).

TITLE

We remain the owner of all right, title and interest in the Software and related explanatory written materials (Documentation) which includes, in part, the Users Manual as well as any other training documentation.

ARCHIVAL OR BACKUP COPIES

You may copy the Software for back-up and archival purposes, provided that the original and each copy is kept in your possession and that your installation and use of the Software does not exceed that allowed in the License Grant section above.

The Users Manual may not be copied, photocopied, reproduced or translated without our express written consent.

APPLICABILITY OF SOFTWARE

The applicability of this software is directly dependent upon your hydraulic systems similarity to the studies, reports, relationships and equations that this software was based upon. As in any complex hydraulic system, <u>we caution that a physical model should be utilized for final design of a side-weir. You agree to review all the pertinent documentation as you are fully responsible for the applicability of the software to your particular case</u>. If you should need additional copies of those studies, you may contact us and we will send you the requested reports. The cost of reproducing such reports will be borne by you.

THINGS YOU MAY NOT DO

United States copyright laws and international treaties protect the Software and Documentation. You must treat the Software and Documentation like any other copyrighted material--for example a book. You may not:

Copy the Software except to make archival or backup copies as provided above,

Modify or adapt the Software or merge it into another program,

Reverse engineer, disassemble, decompile or make any attempt to discover the source code of the Software,

Place the Software onto a server so that it is accessible via a public network such as the Internet, or within your entity's local network, or

Sell, sublicense, rent, lease or lend any portion of the Software or Documentation.

TRANSFERS

This software is for use only by a person who has successfully completed the training course. You may not transfer the use of the Software and Documentation to another person or legal entity. In the event the person trained in the software's use is no longer employed by the entity who financed the training, neither the person nor the entity may use the software.

NO WARRANTY PROVIDED WITH SOFTWARE

IN NO EVENT WILL WE BE LIABLE TO YOU FOR ANY DAMAGES, WHATSOEVER (INCLUDING ANY LOST PROFITS, LOST BUSINESS, LOST SAVINGS, OR OTHER INCIDENTAL OR CONSEQUENTIAL DAMAGES) ARISING FROM THE USE OR THE INABILITY TO USE THE SOFTWARE (EVEN IF WE OR AN AUTHORIZED DEALER OR DISTRIBUTOR HAVE BEEN ADVISED OF THE POSSIBILITY OF THESE DAMAGES), OR FOR ANY CLAIM BY ANY OTHER PARTY.

To the extent permitted by applicable law, THE FOREGOING LIMITED WARRANTY IS IN LIEU OF ALL OTHER WARRANTIES OR CONDITIONS, EXPRESS OR IMPLIED, AND WE DISCLAIM ANY AND ALL IMPLIED WARRANTIES OR CONDITIONS, INCLUDING ANY IMPLIED WARRANTY OF TITLE, NONINFRINGEMENT, MERCHANTABILITY OR FITNESS FOR A PARTICULAR PURPOSE, regardless of whether we know or had reason to know of your particular needs. No employee, agent, dealer or distributor of ours is authorized to modify this limited warranty, or to make any additional warranties.

SUPPORT

Support or training for this software may be purchased by contacting us.

TERM AND TERMINATION

This license agreement takes effect upon your first installation of the software and remains effective until terminated. You may terminate it at any time by destroying all copies of the Software and Documentation in your possession. It will also automatically terminate if you fail to comply with any term or condition of this license agreement. You agree on termination of this license to destroy all copies of the Software and Documentation in your possession.

GENERAL PROVISIONS

This written license agreement is the exclusive agreement between you and us concerning the Software and Documentation and supersedes any prior purchase order, communication, advertising or representation concerning the Software.

Only a writing signed by you and us may modify this license agreement.

In the event of litigation between you and us concerning the Software or Documentation, we will be entitled to recover attorney fees and expenses from you. The laws of the state of California govern this license agreement.

You agree that the Software will not be shipped, transferred or exported into any country or used in any manner prohibited by the United States Export Administration Act or any other export laws, restrictions or regulations.

SYSTEM REQUIREMENT

HARDWARE REQUIREMENT

BAS was configured to take full advantage of the powerful features of the latest hardware available.

SOFTWARE REQUIREMENT

The following is the minimum software requirements for BAS:

Windows Operating System, and Microsoft Excel

Note: BAS will not run on Apple systems.

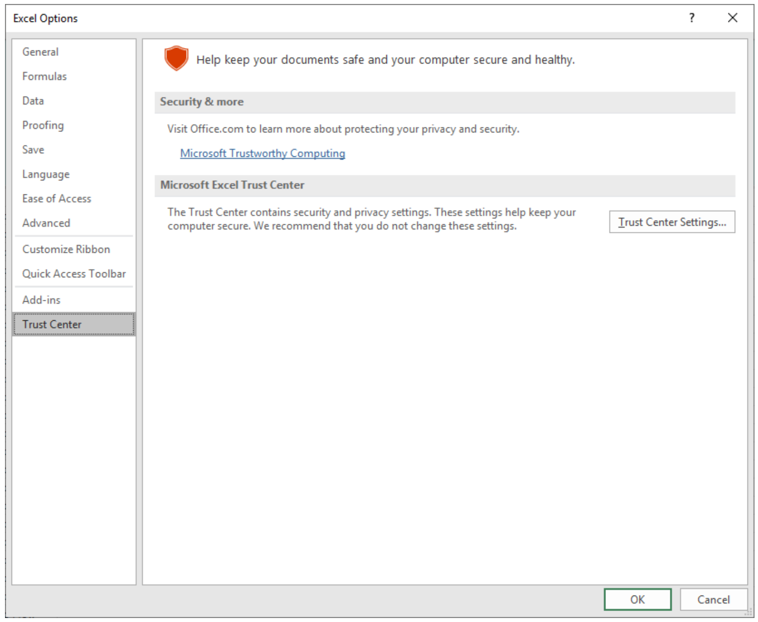
INSTALLATION PROCEDURE

Copy the file director provided on the flash drive anywhere on the computer system.

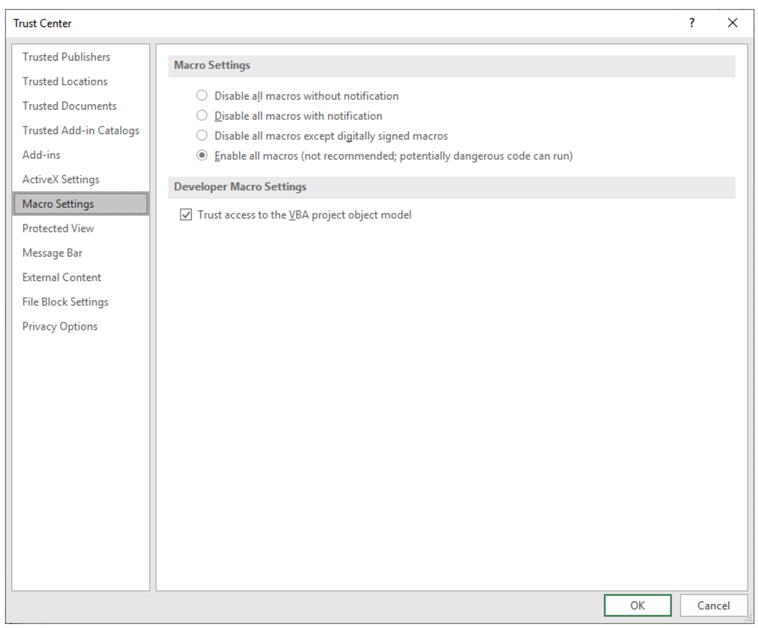
RUNNING B.A.S. FOR THE FIRST TIME (using Excel)

Prior to running BAS, you may need to set the Excel Macro security to Low.

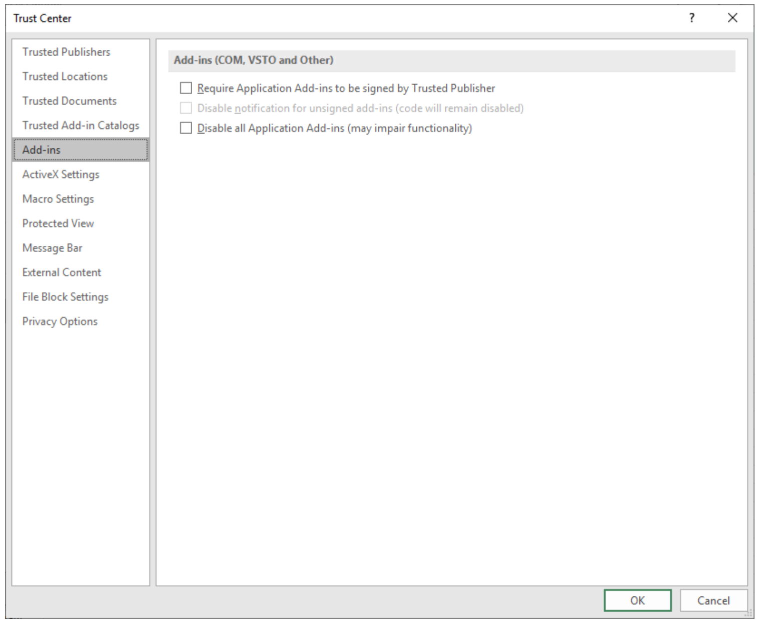
From the **File** menu, press **Excel Options** then **Trust Center**. On the right of the screen press **Trust Center Settings**.



In the **Trust Center Settings**, press **Macro Settings** and **Enable all macros ….**

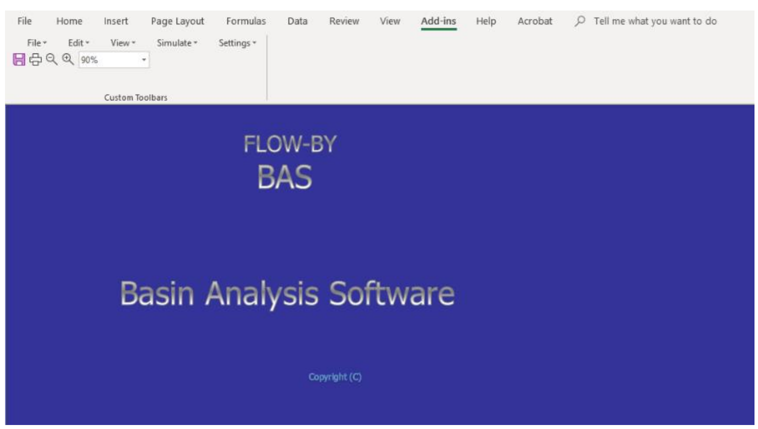


Ensure that the **Add-ins** boxes are not checked



Upon starting B.A.S. for the first time, the user is prompted to accept or reject the terms of the **End User Agreement**. Rejection of the agreement disables the software, while acceptance advances the user to the next step.

On the screen shown below, press the **Add-Ins** tab. The BAS Menu will appear under the **Add-ins** tab, located just below the standard Excel menu. You are now ready to run BAS. Close Excel then run BAS for the first time.

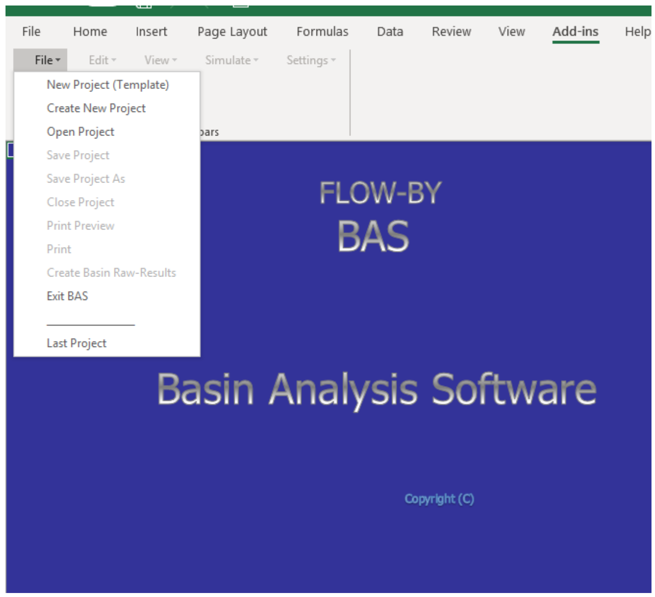


Chapter 2 – THE BAS MENU & FORMS

This chapter presents a detailed explanation of all the BAS menu and sub-menu commands.

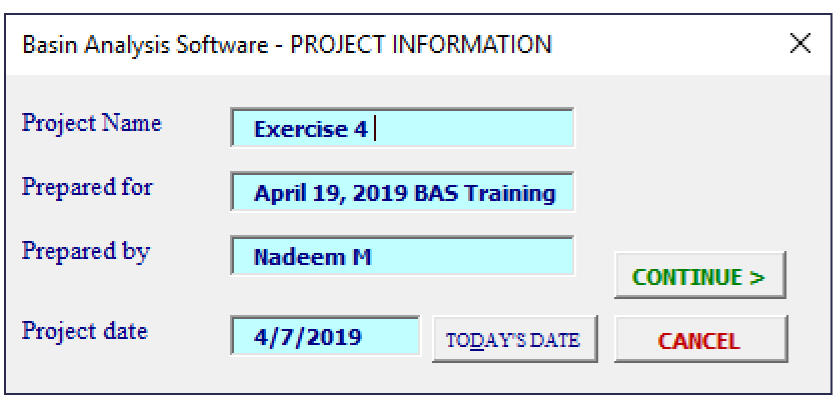
File

The File menu command includes several file management and printing commands.



New Project (Template)

This **New Project (Template)** opens the BAS file template and prompts the user for a new file name. For expediency, it is recommended to use this method for creating a new data file rather than the **Create New Project** method. BAS then displays the Project Information form.

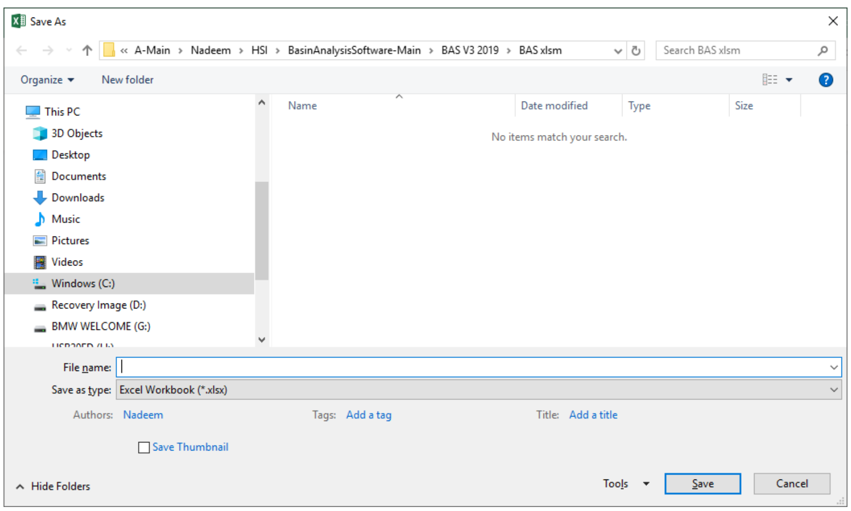


Information provided on this form is later displayed on various BAS report sheets including the report cover. Modifying this information at a later time may be accomplished by the selecting the **Settings/Project Information** sub-menu.

Create New Project

The **Create New Project** command is used in the event that the BAS data-file template is unavailable or has been corrupted. This command opens a new Excel workbook, inserts the required worksheets and formats the entire workbook as a BAS file. Depending upon the speed of the computer's processor and the amount of allocated memory, this process may take several minutes. Prior to creating the new project, BAS displays the Project Information form. Information provided on this form is later displayed on various BAS report sheets. Modifying this information later may be accomplished by the selecting the **Settings/Project Information** sub-menu.

Once the new project has been created, BAS displays Excel's **Save As** form where the project gets saved at a specific location and under a user-specified file name.



Open Project

The **Open Project** sub-menu displays the Windows Open form. The user then selects an existing BAS project file. If a file other than a BAS file is opened, a message will be displayed and the user will be prompted again to open a BAS file.

Save Project

This **Save Project** sub-menu utilizes the Windows **Save As** form to save an opened BAS file.

Save Project As

This **Save Project As** sub-menu displays the Windows **Save As** form which allows the user to save an open BAS project file under a different (or the same) file name. The originally opened file will not include any modifications that may have occurred since the file was opened.

Close Project

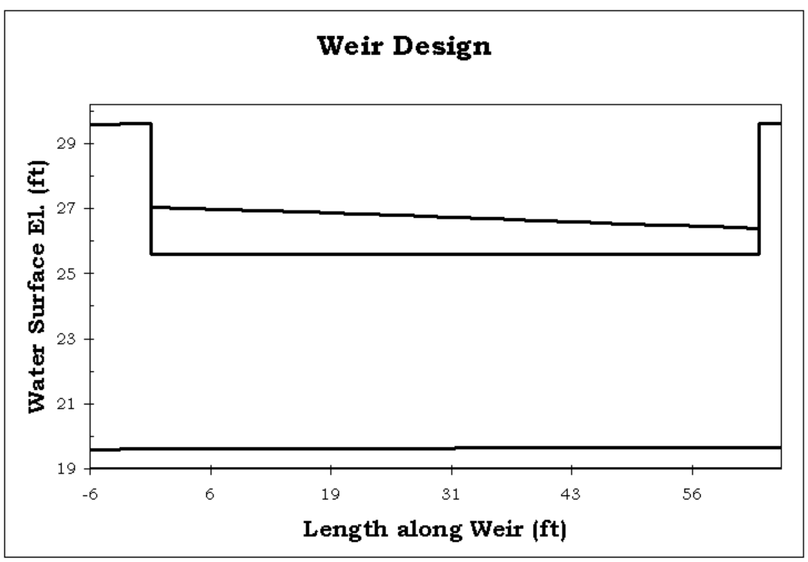
This **Close Project** sub-menu closes an open BAS file and prompts the user to save before closing the file.

Print Preview

The **Print Preview** sub-menu, depending upon the BAS worksheet that is active, previews specific information prior to printing.

With the exception of the **Main Page** (main BAS worksheet) the user is able to preview, prior to printing, any screen-displayed table or graph. For example, if the Weir Analysis graph is displayed, a preview of the Weir Analysis input and output data along with the resulting graph of the water surface along the weir is displayed. The user may choose to print or merely close the preview window.

To **Print Preview** a Basin Analysis Report, the user needs to return to the Main BAS worksheet. The user may then preview all the Basin Analysis report components that have been specified by the user. To set the Basin Analysis report components, please see the **Settings/Report Components** sub-menu. An example **Print Preview** of a Weir Analysis graph is presented below.

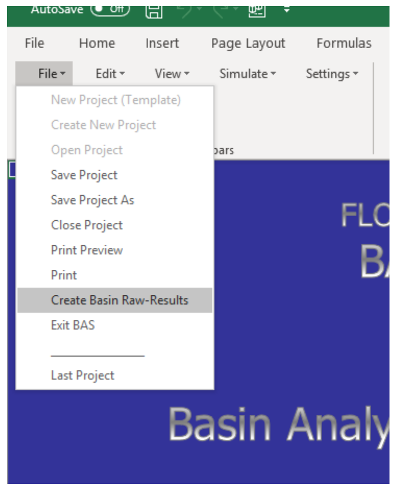


Print

The **Print** sub-menu follows the same logic as that of the Print Preview except that the print command is activated without displaying a preview screen.

Create Basin Raw Results

The **Create Basin Raw Results** sub-menu creates a basic (unformatted) Excel workbook that contains the most recent Basin Analysis table generated by BAS. The user may choose to use certain output, such as the resulting downstream hydrograph as input for other hydrological or hydraulic models.



Exit BAS

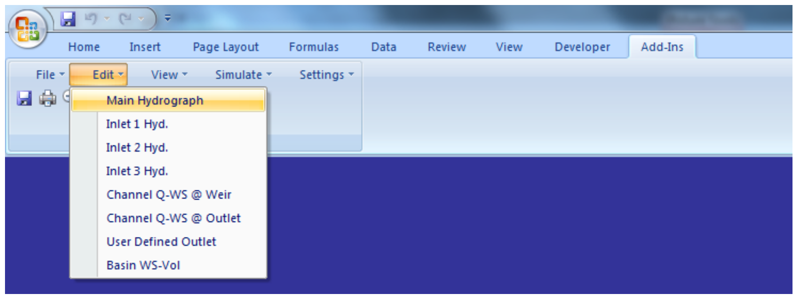
The **Exit BAS** sub-menu quits the Excel application. If a BAS data file is open, it prompts the user to save the opened data file.

Last File

The **Last File** sub-menu displays the name and path to the most recently opened BAS file. The user may choose to open this file or may reject it.

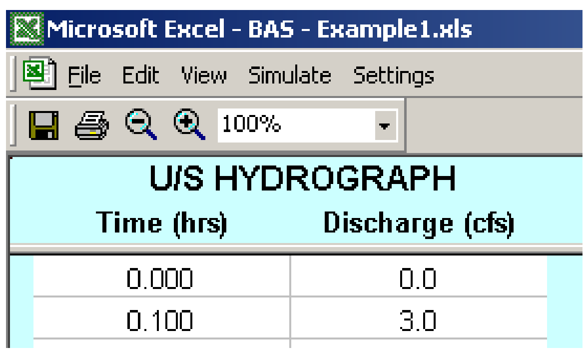
Edit

The **Edit** menu-command includes several submenu commands for editing Basin Analysis related data. Below is a description of each submenu command.



Main Hydrograph

The **Main Hydrograph** sub-menu displays the main channel's hydrograph at the upstream end of the weir (inflow hydrograph). A table is displayed which



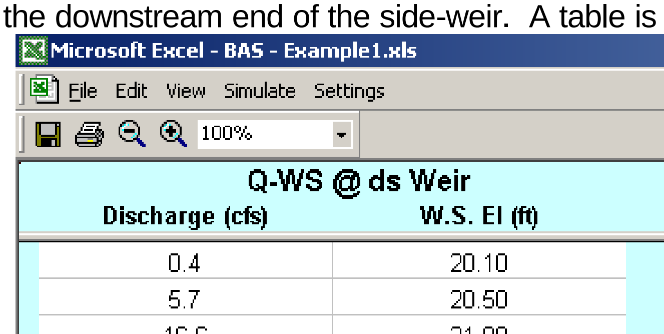
may be utilized to enter new values or edit existing ones.

Inlet 1, 2 and 3 Hyd.

The **Inlet 1, 2 and 3 Hyd.** sub-menus open their hydrograph tables for editing. A table similar to Main Hydrograph is displayed which may be utilized to enter new values or edit existing ones.

Q-WS @ Weir

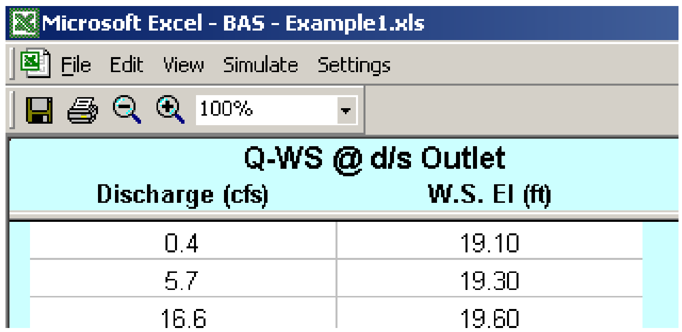
The **Q-WS @ Weir** sub-menu displays the channel's stage-discharge relationship at the downstream end of the side-weir. A table is displayed which



may be utilized to enter new values or edit existing ones.

Q-WS @ Outlet

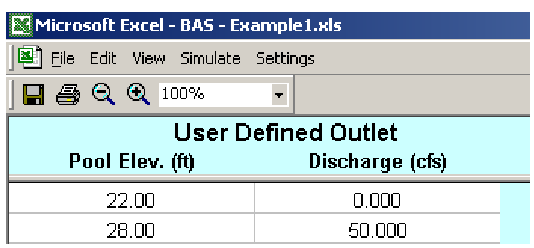
The **Q-WS @ Outlet** sub-menu displays the channel's stage-discharge relationship at the channel's confluence with the basin's outlet. A table is displayed which may be utilized to enter new values or edit existing ones. In the event that the basin’s outlet is located at the upstream of the channel’s choke, this table need not be utilized. BAS then automatically utilizes the values within the Q-WS @ Weir table.



Q-WS @ d/s Outlet	
Discharge (cfs)	W.S. El (ft)
0.4	19.10
5.7	19.30
16.6	19.60

UD Outlet

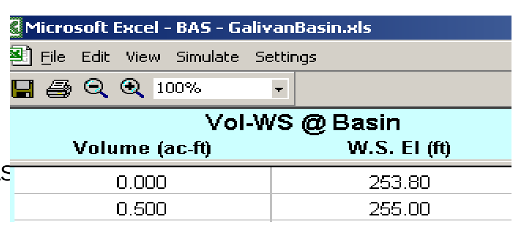
The **UD Outlet** (user-defined values) sub-menu displays the basin’s stage-discharge relationship for the basin's outlet. The table may be utilized to enter new values or edit existing ones. This option is useful when the user chooses to utilize a specific outlet stage-discharge relationship. Further, this option allows for inclusion of a separate outflow device that may be set at a different elevation than the standard pipe outlet.



User Defined Outlet	
Pool Elev. (ft)	Discharge (cfs)
22.00	0.000
28.00	50.000

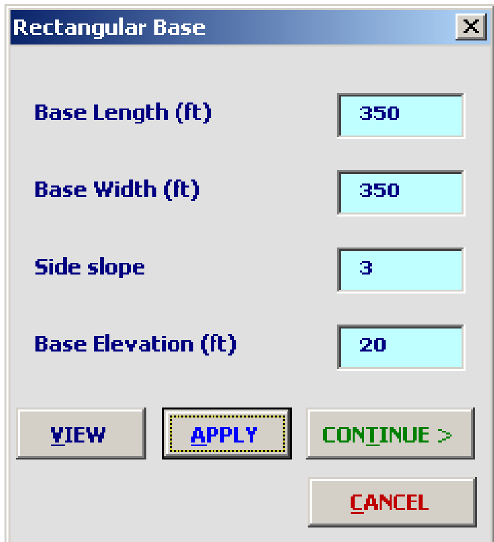
WS-Vol

The **WS-Vol** sub-menu displays the basin's stage-volume relationship. A table is displayed which may be utilized to enter new values or edit existing ones.



Vol-WS @ Basin	
Volume (ac-ft)	W.S. El (ft)
0.000	253.80
0.500	255.00

If the system includes a rectangular basin the above table will not be displayed and the BAS input form shown below will be displayed to access the basin's data.

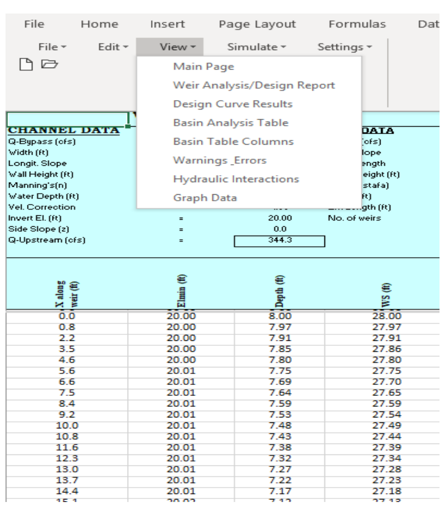


To change from a **User Defined** values to a rectangular shaped basin, review the procedure under the **Simulate** menu for **Basin Analysis**.

View

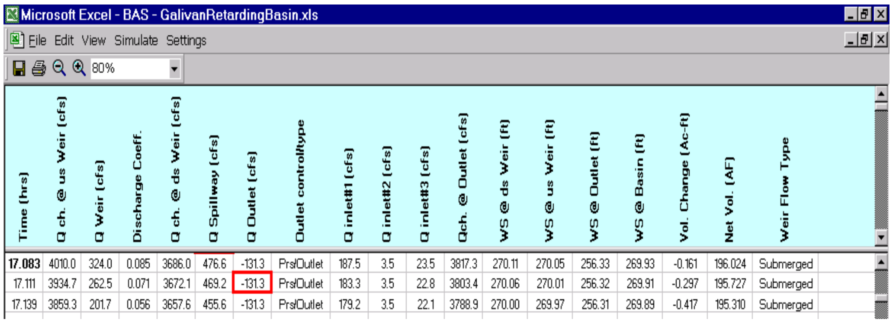
Main Page

The **Main Page** sub-menu displays the main BAS worksheet. This worksheet is a central location to access all the components of BAS. In addition, this worksheet is the location where the various components of a Basin Analysis Report may be previewed or printed.



Basin Analysis Table

The **Basin Analysis Table** sub-menu displays the resulting table after a basin analysis computation has been completed. Below is a sample Basin Analysis table. For a detailed discussion of the Basin Analysis table, see the

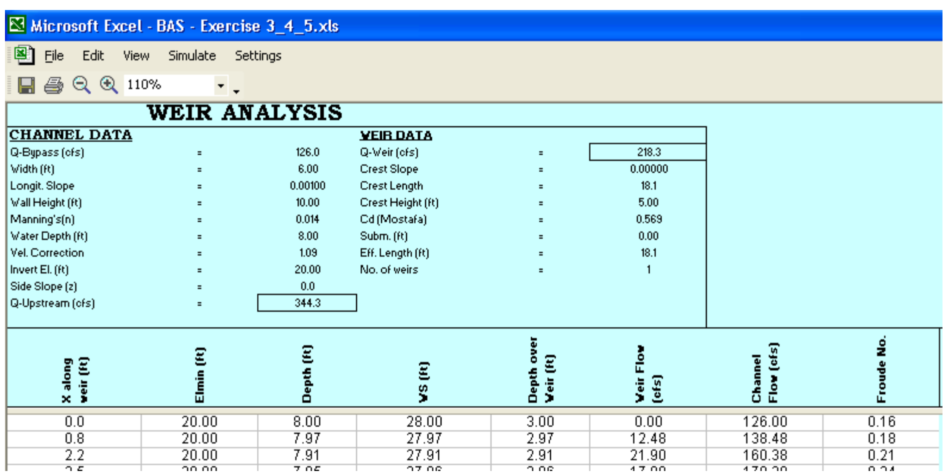


Time (hrs)	Q ch. @ us Weir (cfs)	Q Weir (cfs)	Discharge Coeff.	Q ch. @ ds Weir (cfs)	Q Spillway (cfs)	Q Outlet (cfs)	Outlet controltype	Q inlet#1 (cfs)	Q inlet#2 (cfs)	Q inlet#3 (cfs)	Qch. @ Outlet (cfs)	WS @ ds Weir (ft)	WS @ us Weir (ft)	WS @ Outlet (ft)	WS @ Basin (ft)	Vol. Change (Ac-ft)	Net Vol. (AF)	Weir Flow Type
17.083	4010.0	324.0	0.085	3686.0	476.6	-131.3	PrsOutlet	187.5	3.5	23.5	3817.3	270.11	270.05	256.33	269.93	-0.161	196.024	Submerged
17.111	3934.7	262.5	0.071	3672.1	469.2	-131.3	PrsOutlet	183.3	3.5	22.8	3803.4	270.06	270.01	256.32	269.91	-0.297	195.727	Submerged
17.139	3859.3	201.7	0.056	3657.6	455.6	-131.3	PrsOutlet	179.2	3.5	22.1	3788.9	270.00	269.97	256.31	269.89	-0.417	195.310	Submerged

"Understanding the Basin Analysis Report" section in Chapter 6.

Weir Analysis Report

The **Weir Analysis Report** sub-menu displays the resulting table after a **Weir Design** or **Analysis** computation has been completed. Below is a sample Weir Analysis Report.



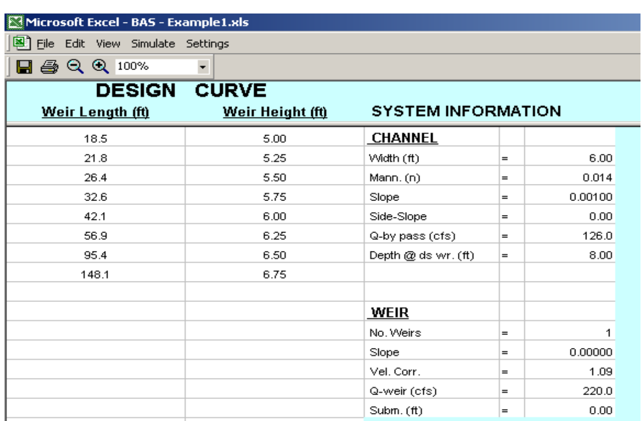
WEIR ANALYSIS

CHANNEL DATA			WEIR DATA		
Q-Bypass (cfs)	=	126.0	Q-Weir (cfs)	=	218.3
Width (ft)	=	6.00	Crest Slope	=	0.00000
Longit. Slope	=	0.00100	Crest Length	=	18.1
Wall Height (ft)	=	10.00	Crest Height (ft)	=	5.00
Manning's(n)	=	0.014	Cd (Mostafa)	=	0.569
Water Depth (ft)	=	8.00	Subm. (ft)	=	0.00
Vel. Correction	=	1.09	Eff. Length (ft)	=	18.1
Invert El. (ft)	=	20.00	No. of weirs	=	1
Side Slope (z)	=	0.0			
Q-Upstream (cfs)	=	344.3			

X along weir (ft)	Elmin (ft)	Depth (ft)	WS (ft)	Depth over Weir (ft)	Weir Flow (cfs)	Channel Flow (cfs)	Froude No.
0.0	20.00	8.00	28.00	3.00	0.00	126.00	0.16
0.8	20.00	7.97	27.97	2.97	12.48	138.48	0.18
2.2	20.00	7.91	27.91	2.91	21.90	160.38	0.21

Design Curve Results

The **Design Curve Results** sub-menu displays a table that includes the weir



DESIGN CURVE

Weir Length (ft)	Weir Height (ft)	SYSTEM INFORMATION		
18.5	5.00	CHANNEL		
21.8	5.25	Width (ft)	=	6.00
26.4	5.50	Mann. (n)	=	0.014
32.6	5.75	Slope	=	0.00100
42.1	6.00	Side-Slope	=	0.00
56.9	6.25	Q-by pass (cfs)	=	126.0
95.4	6.50	Depth @ ds wr. (ft)	=	8.00
148.1	6.75			
		WEIR		
		No. Weirs	=	1
		Slope	=	0.00000
		Vel. Corr.	=	1.09
		Q-weir (cfs)	=	220.0
		Subm. (ft)	=	0.00

lengths versus heights for a specified channel geometry and flow conditions. To the right of the table is the System Information (input data).

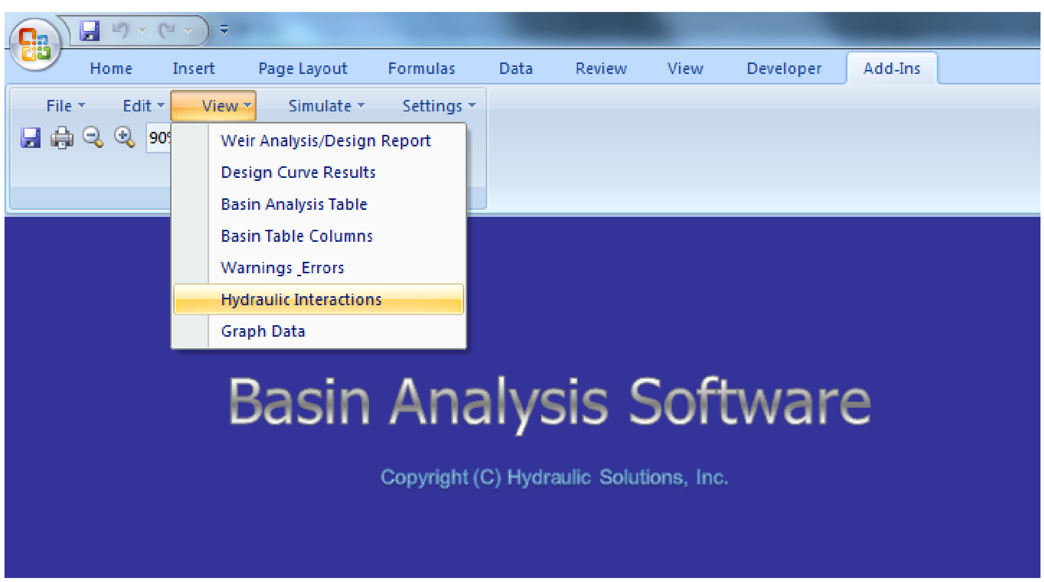
Warnings & Errors

The **Warnings & Errors** sub-menu displays any error and/or warnings that may have been generated while BAS attempted to perform a Basin Analysis computation. If no warnings or errors were generated, BAS will not display the table. An example warnings and errors table is presented below.

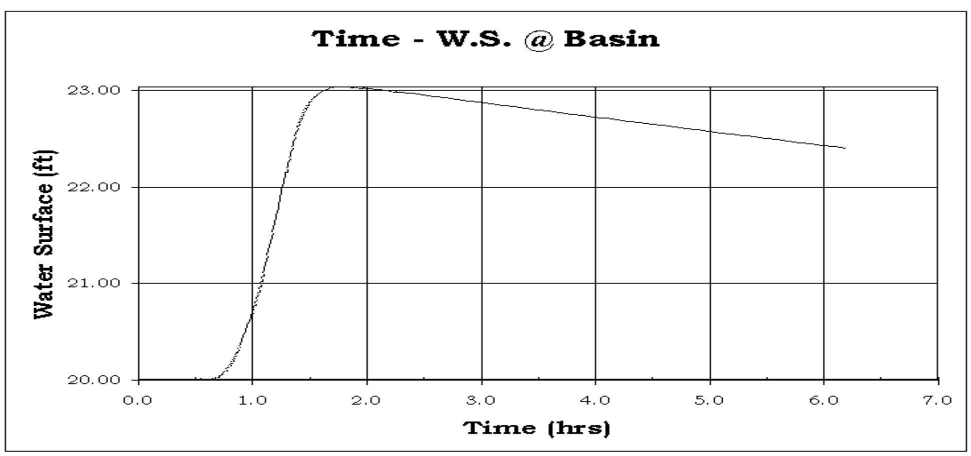
	WARNINGS & ERRORS
1	An error was entered in the # 22 row of the Stage-Discharge at d/s Weir Table (Q Column)
2	A 0 or negative number was entered in the channel Wall Height field
3	A 0 or negative value was entered in the Weir Length field
4	No Outlet Pipe Diameter was entered
5	The weir crest height exceeds channel wall
6	Outlet pipe diameter exceeds channel wall height
7	Channel invert intersects weir crest. Reduce channel slope

Hydraulic Interactions

The **Relationships** sub-menu provides the user an opportunity to view various timing related relationships of a **Basin Analysis** calculation.

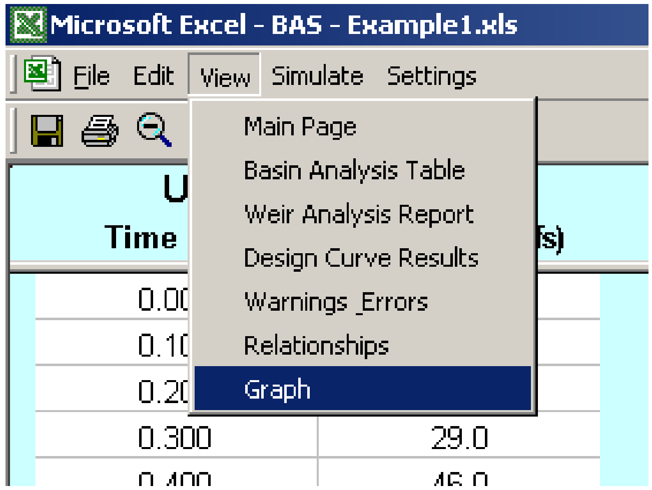


Below is an example of a **Hydraulic Interactions** graph. Other relationships generated by BAS are discussed in the “Understanding the Basin Analysis Report” in Chapter 6.



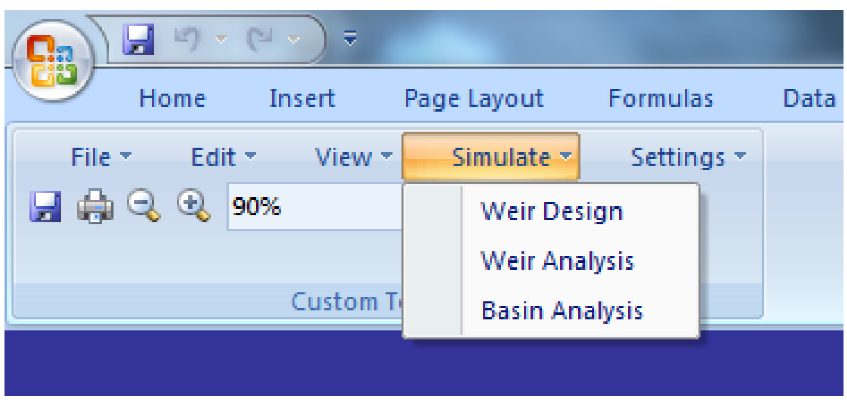
Graph

The **Graph** sub-menu displays the graph of the tabulated values on an active worksheet. For example, if the active view is that of the hydrograph table, BAS graphs the Time-Discharge relationship (hydrograph).



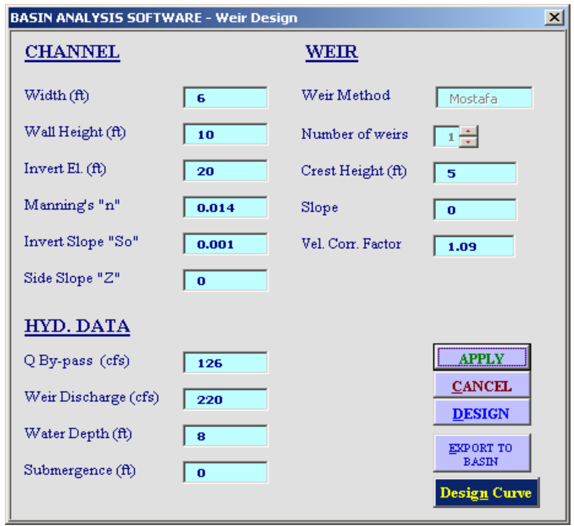
Simulate

The **Simulate** menu command provides access to the three BAS components: **Weir Design**, **Weir Analysis** and **Basin Analysis is**.



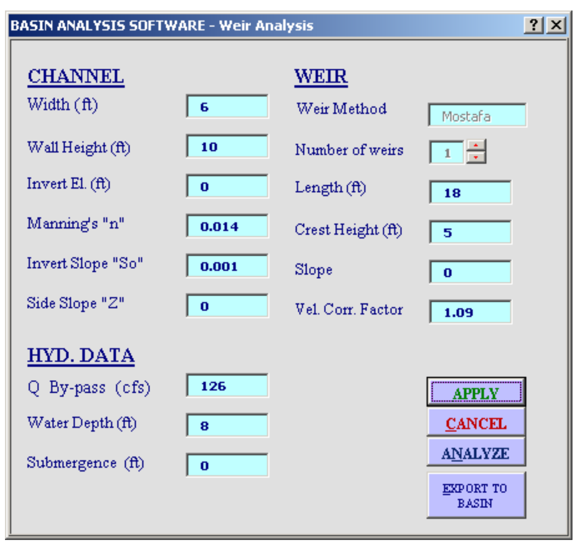
Weir Design

The **Weir Design** sub-menu displays an input form for the design of a side-weir. For a detailed discussion of this form, please read the Weir Design Chapter.



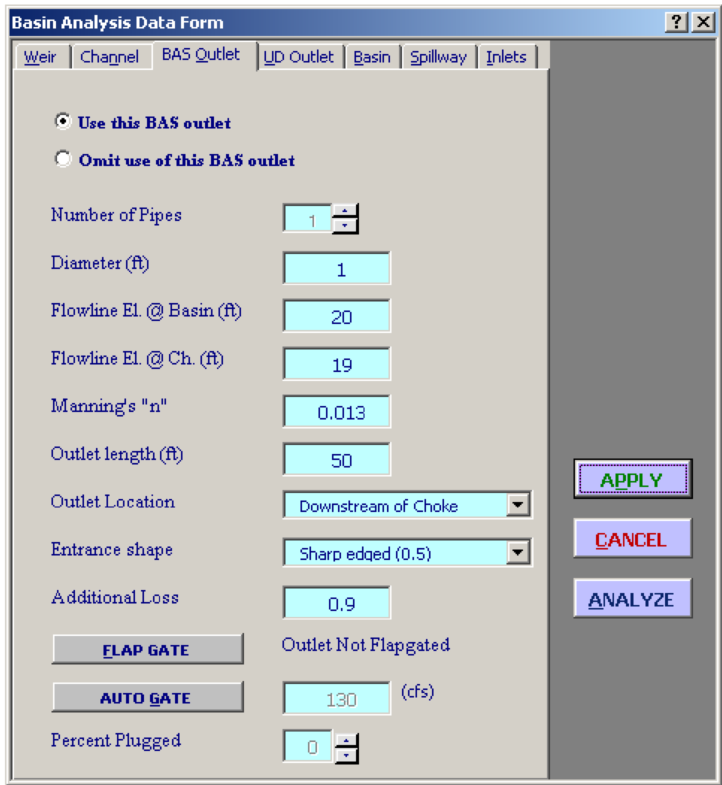
Weir Analysis

The **Weir Analysis** sub-menu displays an input form for the design of a side-weir. For a detailed discussion of this form, please read the Weir Design Chapter.



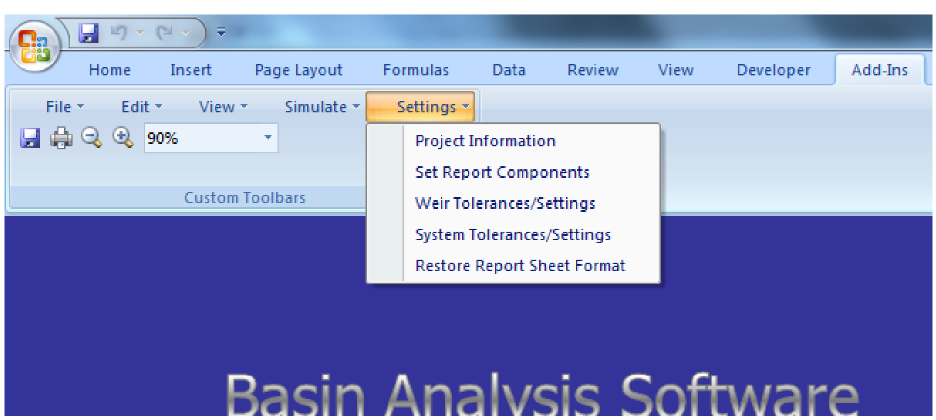
Basin Analysis

The **Basin Analysis** sub-menu displays the input form for the analysis of an offline retarding system. This form contains several pages (tabs) that are discussed in more detail in the Basin Analysis Chapter.



Settings

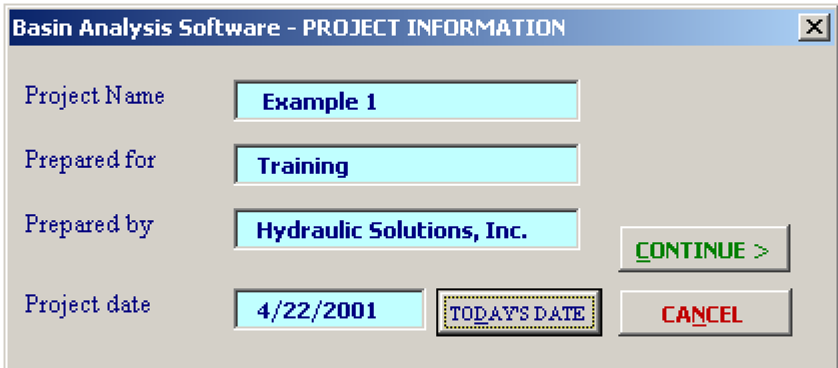
The **Settings** menu command provides access to all the BAS internal settings. Except for the Project Information sub-menu command, it is generally not recommended to modify any of the settings. In the event that the settings are



changed, pressing the Default button will return BAS to its original settings.

Project Information

The **Project Information** sub-menu displays a short form that prompts the user to identify specific information regarding an opened project. This information is later displayed on various BAS report sheets.



Report Components

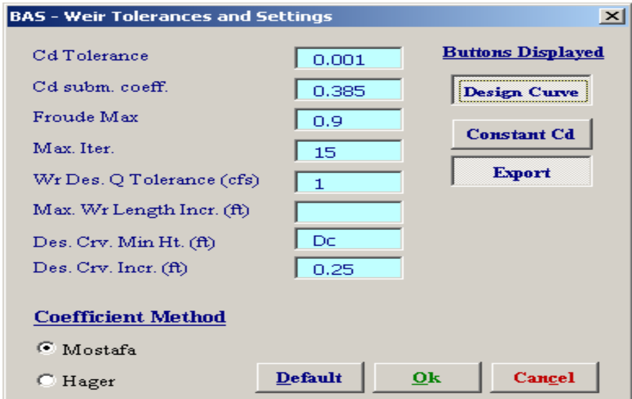
The **Report Components** sub-menu allows the user to specify which components of an analyzed project will be previewed or printed. The user may choose to preview or print directly from this form. Else, the report may be previewed or printed while in the BAS Main Page.

Basin & Weir Tolerances \ Settings

The **Basin & Weir Tolerances\Settings** sub-menu displays a form that includes various internal settings for BAS. For a detailed discussion of this form, see the Basin Analysis Settings in the Basin Analysis chapter. However, it is generally not recommended that any of the values be modified without consulting with Nadeem Majaj first. In the event that one or more settings are modified, it is possible to recover by pressing the Default button.

Weir Tolerances & Settings

The **Weir Tolerances & Settings** sub-menu displays a form that includes various internal settings for BAS. For a detailed discussion of this form, see the **Weir Analysis Settings** in the **Weir Analysis** chapter. However, it is generally not recommended that any of the values be modified without consulting with the software developer first. In the event that one or more settings are modified, it is possible to recover by pressing the Default button.



Restore Report Sheet Format

The **Restore Report Sheet Format** sub-menu displays a form that allows the user to select any of the BAS worksheets for re-formatting. The need to re-format may occur when the user has modified the formatting of a sheet sufficiently to disable BAS to function properly or to appropriately display the data and/or results.

Chapter 3 - WEIR DESIGN

The design and analyses of side-weirs is one of the most complex hydraulic challenges. Numerous methods and equations have been developed in the past century for such analyses. Often the results between one method and another vary significantly. It is always prudent to fully comprehend the original technical papers that develop the methodology for such analyses. Once the engineer chooses and utilizes a method that he/she feels is most applicable, it is strongly recommended that a physical model be constructed to verify and fine-tune the design. Methods within **BAS** are no exception to this philosophy. For an overview of the development of the side-weir analyses, see Appendix C of this manual.

The **Weir Design** component includes features that determine the length and height of a weir that is necessary to divert a user-specified flow into a basin. The user supplies the channel parameters, the desired diversion and the amount of submergence, if any, over the weir. Once the computations have been completed, BAS generates a comprehensive report of the weir design. This report includes the information supplied by the user (input data) as well as the results of the analysis (output data). Within the report, a table is presented which includes the water surface elevation, the depth and the Froude Numbers along the length of the weir. Detailed information is presented in the Weir Design Report section of this chapter.

In the event that the user-specified parameters do not result in the desired weir flow, BAS includes a statement in the report notifying the user that the design goals have not been met. The user may then modify the input to accomplish a successful design. Typical causes for such failures result from a low water surface in the channel at the downstream end of the weir or from a hydraulic jump occurring in the channel within the weir segment. In either case, the water surface dips below the weir crest and lengthening the weir would not result in any increase in weir flow.

BAS also includes a very useful feature, the **Design Curve** utility. This utility generates a table and graph of a series of weir lengths and heights that are necessary to convey a user-specified weir flow into a basin. This information enables the user to select a short weir with a minimum height or a very long weir with a high weir crest height. Once a set of weir design parameters are computed, the user may export the data to the Basin Analysis component for future use.

FEATURES:

The following is an overview of the features provided within the **Weir Design** component:

Spatially Varied Flow Analysis – The **Weir Design** component analyzes the water surface profile and discharge along a sharp crested side-weir in a subcritical prismatic channel. The computations utilize the classic spatially-varied-flow function as presented in Ven T. Chow's "Open Channel Hydraulics" and other hydraulic references.

$$\frac{dy}{dx} = \frac{S_o - S_f - \alpha Q q_* / gA^2}{1 - \alpha Q^2 / gA^2 D} \qquad \text{(Chow 1959)}$$

where Q = channel discharge, S_o = channel slope, S_f = friction slope, dy/dx = change in water surface with respect to distance along weir, D = channel depth, A =flow area,

g = acceleration of gravity and $\alpha =$ velocity correction factor

$$q_* = \frac{dQ}{dx} = (-C_d(y - y_w)^{1.5}) \qquad \text{(Mostafa 1987)}$$

and q_* = side-weir's unit discharge, C_d = side-weir's coefficient of discharge,

y = channel depth at weir, y_w = weir height

Coefficient of Discharge – Studies have shown that a side-weir's coefficient of discharge varies with the change in weir discharge and flow depth. In its default mode, BAS adjusts the coefficient of discharge to account for this variance. The user may also utilize a constant (user-specified) coefficient of discharge that may be derived from other sources. The following equation is the current default equation which provides an adjustment for the coefficient:

$$C_d = C' + 0.259C'(y - y_w)^2 / y^2 \quad \text{and} \quad C' = 0.60(Q_w / Q_o)^{0.167} \qquad \text{(Mostafa 1987)}$$

where Q_w = weir discharge, Q_o = upstream discharge and C' = Cd adjustment based upon weir flow rate to the by-pass flow rate and y (and y_w) are as defined above.

BAS also includes a correction coefficient relationship as provided by Hager (1987). For technical details and applicability of the equation, please review the referenced paper.

Double Weirs – BAS allows for the design or analysis of one or two side-weirs (both sides of channel) in a single analysis.

Sloping Weirs – The slope of the weir may vary from that of the channel's invert slope.

Weir Submergence – In some cases, the water surface in the retarding basin may exceed the weir crest elevation and therefore the side-weir becomes "submerged". BAS adjusts the coefficient of discharge (C_d) to account for such submergence. The following equation accomplishes the submergence adjustment:

$$\frac{C_{ds}}{C_d} = \left[1 - \left(\frac{H_1}{H}\right)^{1.5}\right]^{.35} \qquad \text{(Mostafa 1974)}$$

where C_{ds} = submerged-flow discharge coefficient, H_1 = submergence head, H = free head and C_d is as defined above

Flow Instability - In cases where the user-specified data results in an upstream weir discharge with its flow depth approaching critical depth, BAS terminates the computations and notifies the user that unstable flow has resulted at a specific location along the weir. The default setting for this condition is a Froude number of 0.90. However, this value may be reset to any Froude number between 0 and 0.95 by selecting **Settings\Weir Tolerances\Settings**.

Hydraulic Jumps - In the event that the channel's invert slope along the weir is supercritical while the flow regime at the downstream end of the weir is subcritical, BAS will compute the gradually varied water surface along the weir. BAS will then compare the Force (Pressure + Momentum) of the upstream supercritical flow with the Force at each station along the weir. If the Force of the upstream (supercritical) flow equals the Force in the channel (subcritical) along the weir at a specific location, a hydraulic jump is assumed to occur at that location and the "effective weir length" is limited to the location of the jump. The upstream supercritical flows are assumed to be uniform for this analysis.

Note: Should a hydraulic jump occur within the weir length, it is recommended that a re-design of the weir and channel be considered to eliminate a jump from forming within the channel reach that includes the weir.

Evaluation of Results – Once the design computations have been completed, BAS evaluates the results and notifies the user if the target weir discharge was not achieved.

Export of Channel and Weir parameters - Once the computations have been completed, the user has the option to export the channel and weir data to the **Basin Analysis** component for future basin analyses.

Design Curve – BAS generates a useful "Length vs. Height" relationship to assist in the selection of the proper design parameters. This design curve exhibits the range of weir lengths and heights that may be utilized for specific channel geometry and flow conditions.

Output Report - Detailed input and output data including displays of the weir and the water surface profile along the weir are automatically generated by BAS.

THE WEIR DESIGN FORM:

The **Weir Design** form displayed below requires input of all the essential information for a successful design computation. The **APPLY**, **CANCEL** and **DESIGN** buttons are displayed each time the form is called. The **DESIGN** button is grayed until the values provided on the form are acceptable and the **APPLY** button has been pressed.

The **Weir Design** component is relatively simple to use. However, to ensure proper input of the systems parameters, each input value and all the selection options are defined below.

CHANNEL

Width (ft):	The main channel's base width.
Wall Height (ft):	The main channel's wall height upstream and downstream of the side-weir.
Invert El. (ft):	The main channel's invert elevation at the downstream end of the weir.
Manning's "n":	The main channel's Manning's coefficient of friction.
Invert Slope "So":	The main channel's longitudinal invert slope along the weir.
Side Slope "Z":	The main channel's side-slope ratio.

WEIR

Weir Method:	Indicates the current specified weir coefficient method, i.e. Mostafa or Hager.
Number of weirs:	Two identical weirs may be modeled opposite each other.
Crest Height (ft):	The vertical distance from the channel's invert to the weir crest at the downstream end of the weir.
Slope:	The weir crest slope along the main channel.
Vel. Corr. Factor:	The velocity correction factor accounts for the variation of the velocity within a channel cross-section along the weir.

HYD. DATA

Q By-Pass (cfs):	The channel's by-pass flow just downstream of the side-weir.
Weir Discharge (cfs):	The target weir discharge for the design.

Water Depth (ft): The depth of water at the downstream end of the side-weir (corresponding to Q By-Pass (cfs).

Submergence (ft): The depth of submergence over the side-weir. This occurs when the basin's water surface exceeds the weir crest.

Constant Cd

The **Constant Cd** button is used when the user desires to use a user-specified coefficient of discharge for the design of the weir. This button as well as the other two buttons displayed at the lower right hand corner of the form are optional and may be turned on or off through the **Settings\Weir Tolerances_Settings** sub-menu.

Export to Basin Analysis

The **Export to Basin Analysis** button is used to transfer the channel and weir data to the **Basin Analysis** component for future computations.

Design Curve

The **Design Curve** feature is a very useful feature of BAS. When selected, BAS determines the range of weir crest lengths and heights that would result in a desired flow diversion over a weir. To utilize this feature, the **Weir Design** form must be completely filled in with the exception of the weir height field.

Note: if the Design Curve button is not displayed, go to **Settings\Weir Tolerances_Settings** and press the **Design Curve** button. This will enable the button to be displayed on the Weir Design form.

The default minimum weir crest height is automatically set at close to the critical depth at the upstream end of the weir. The default setting for this is a Froude Number of 0.90 (to change this setting, go to the **Settings\Weir Tolerances_Settings** menu command). The minimum weir height may also be reset to below the critical depth. On the other hand, the maximum weir crest height is the case where the desired weir flow is achieved yet no additional flows may be diverted over the weir by increasing the weir's length since the water surface profile dips below the weir crest.

A QUICK GUIDE TO RUNNING THE WEIR DESIGN:

The **Weir Design** component offers two methods for arriving at the desired weir lengths.

METHOD 1

Method 1 assumes that the weir crest height is known. BAS then calculates the appropriate weir length. The results of this calculation are very detailed and include a weir design table (see the Weir Design Example) as well as a view of the weir and its water surface profile. The following are the required steps to accomplish this method:

Enter the main channel's width, wall height, invert elevation, Manning's friction factor, invert slope and side-slope

Enter the side-weir's number of weirs, weir crest height, weir crest slope and the velocity correction factor

If the design is based upon a known coefficient of discharge, press the **Constant Cd** button and enter its value

Enter the hydraulic data for the system. This includes the Q By-pass, the desired weir discharge, the depth of water in the channel at the downstream end of the weir and the submergence, if any.

Press the **APPLY** button

Press the **DESIGN** button

METHOD 2

Method 2 assumes that the weir height and length are unknown. BAS calculates a series of weir lengths and heights that would result in the desired weir discharge. The following are the required steps to accomplish this method:

Enter the main channel's width, wall height, invert elevation, Manning's friction factor, invert slope and side-slope

Enter the side-weir's number of weirs, weir crest slope and the velocity correction factor

If the design is based upon a known coefficient of discharge, press the **Constant Cd** button and enter its value

Enter the hydraulic data for the system. This includes the Q By-pass, the desired weir discharge, the depth of water in the channel at the downstream end of the weir and the submergence, if any.

Press the **APPLY** button

Press the **DESIGN CURVE** button

Select a weir length and height from the resulting Design Curve table and input the values in the Weir Design form.

Press the **APPLY** button

Press the **DESIGN** button

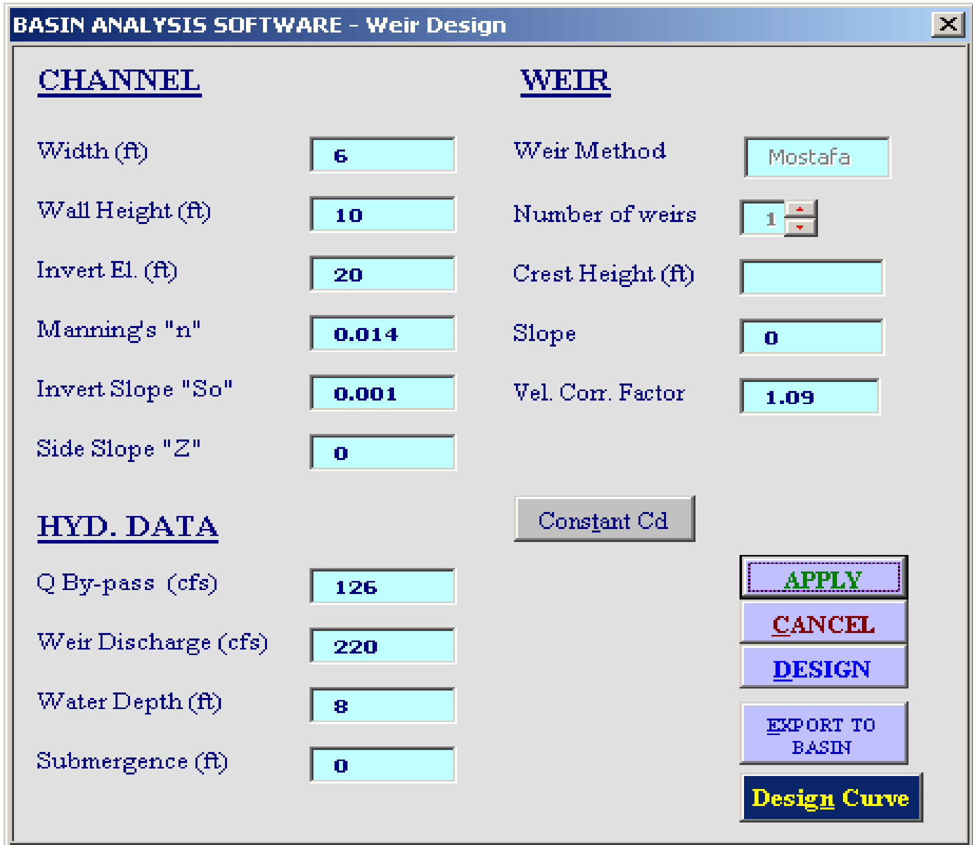
A WEIR DESIGN EXAMPLE:

In order to demonstrate the use of the **Weir Design** component, a detailed example is provided below. It is recommended to first utilize the **Design Curve** utility to determine the relationship between the weir length and height. The following is a sample problem along with its solution using BAS:

A development is proposed along a creek which is anticipated to increase the flows in the creek from 130.3 cfs to 346 cfs. Development conditions, in part, requires no increase in discharge in the downstream existing channel. Therefore, retarding was necessitated within the development. First, an online retarding basin was preliminary designed using state-of-the-art optimization software. The optimized system required an area of 2.66 acres in order to accomplish the desired diversion.

Offline retarding, when possible, is considerably more efficient in accomplishing the same amount of retarding. Therefore, BAS was utilized. It is not practical to include, herein, all of the considerations in the design of the main channel and the choke box. However, the values for these parameters are provided as input into the Weir Design form. The optimized system, using BAS required an area of 1.63 acres in order to accomplish the desired diversion. This is 39 percent less surface area than the optimized online retarding basin. Where land values are at a premium, this could translate into substantial cost savings in right-of-way. The following are the input parameters for the design of the side-weir structure. The complete retarding system analysis is presented in the Basin Analysis chapter.

Note: The weir crest height was not included since the first step will be to run the Design Curve utility.

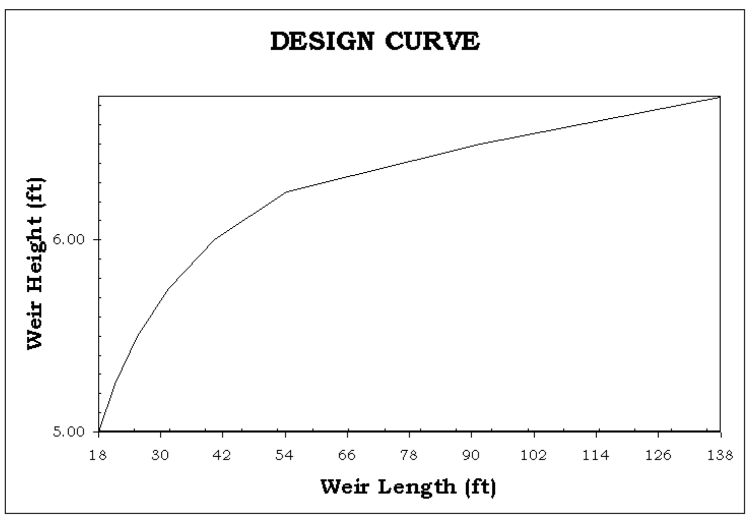


RUN THE DESIGN CURVE UTILITIY

Running establishes the range of weir lengths and heights that would accomplish the desired diversion. The selection of a particular combination is best done after running the Basin Analysis component for several combinations and determining which combination results in an optimized system. The following table resulted from this run:

DESIGN CURVE				
Weir Length (ft)	Weir Height (ft)	SYSTEM INFORMATION		
18.1	5.00	CHANNEL		
21.3	5.25	Width (ft)	=	6.00
25.6	5.50	Mann. (n)	=	0.014
31.5	5.75	Slope	=	0.00100
40.3	6.00	Side-Slope	=	0.00
54.3	6.25	Q-by pass (cfs)	=	126.0
91.7	6.50	Depth @ ds wr. (ft)	=	8.00
138.8	6.75			
		WEIR		
		No. Weirs	=	1
		Slope	=	0.00000
		Vel. Corr.	=	1.09
		Q-weir (cfs)	=	220.0
		Subm. (ft)	=	0.00

To aid the user in visualizing the length versus height relationship, BAS includes a graphing utility for this relationship. This may be accessed from the **View/Graph** sub-menu.



From the table and graph above, it was decided to select the values for the weir length and height to be 18.1 ft and 5.0 ft respectively.

Next, a weir height of 5.0 ft was entered and the **DESIGN** button was pressed. Although it was known that an 18.1 ft. long weir is needed, this step provides a detailed Weir Design report as shown below and verifies the weir length.

WEIR DESIGN REPORT:

The **Weir Design Report** is comprised of three blocks of data:

1. Input/Output table
2. Water surface profile data along the weir
3. Graph of the water surface data along the weir

Input/Output Table

This table is located directly above the water surface profile table as shown below.

Weir Design

CHANNEL DATA			WEIR DATA		
Q-Bypass (cfs)	=	126.0	Q-Weir (cfs)	=	220.8
Width (ft)	=	6.00	Crest Slope	=	0.00000
Longit. Slope	=	0.00100	Crest Length	=	18.1
Wall Height (ft)	=	10.00	Crest Height (ft)	=	5.00
Manning's(n)	=	0.014	Cd (Mostafa)	=	0.570
Water Depth (ft)	=	8.00	Subm. (ft)	=	0.00
Vel. Correction	=	1.09	Eff. Length (ft)	=	18.1
Invert El. (ft)	=	20.00	No. of weirs	=	1
Side Slope (z)	=	0.0			
Q-Upstream (cfs	=	346.8			

Input data comprises both columns with the exception of the Q-Upstream, Q-Weir and Crest Length fields (boxed). These values result from the computations and are derived from the more detailed water surface profile table below this section.

Water Surface profile along the weir

The detailed water surface profile table is presented below and is followed by a discussion of each table-column.

WEIR ANALYSIS

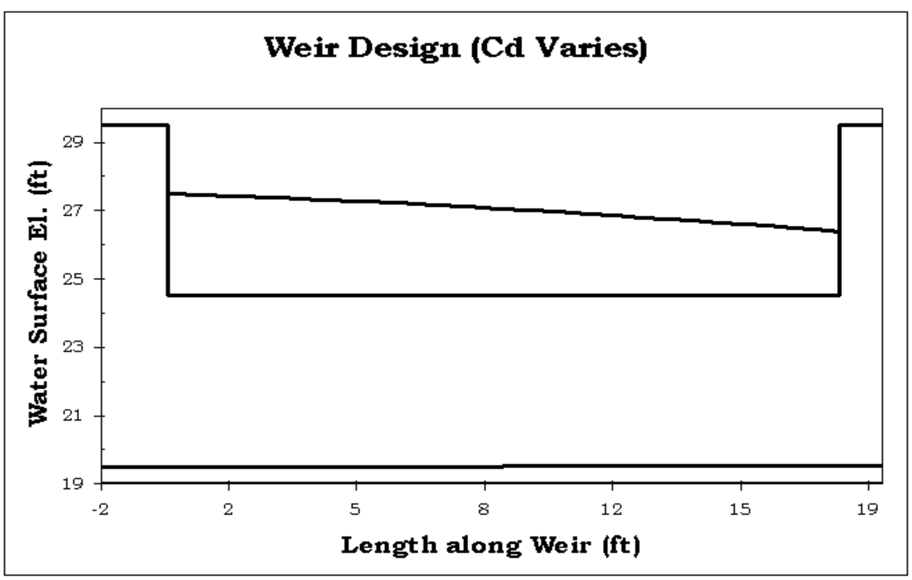
CHANNEL DATA			WEIR DATA		
Q-Bypass (cfs)	=	126.0	Q-Weir (cfs)	=	218.3
Width (ft)	=	6.00	Crest Slope	=	0.00000
Longit. Slope	=	0.00100	Crest Length	=	18.1
Wall Height (ft)	=	10.00	Crest Height (ft	=	5.00
Manning's(n)	=	0.014	Cd (Mostafa)	=	0.569
Water Depth (ft	=	8.00	Subm. (ft)	=	0.00
Vel. Correction	=	1.09	Eff. Length (ft)	=	18.1
Invert El. (ft)	=	0.00	No. of weirs	=	1
Side Slope (z)	=	0.0			
Q-Upstream (cl	=	344.3			

X along weir (ft)	Elmin (ft)	Depth (ft)	WS (ft)	Depth over Weir (ft)	Weir Flow (cfs)	Channel Flow (cfs)	Froude No.
0.0	0.00	8.00	8.00	3.00	0.00	126.00	0.16
0.8	0.00	7.97	7.97	2.97	12.48	138.48	0.18
2.2	0.00	7.91	7.91	2.91	21.90	160.38	0.21
3.5	0.00	7.85	7.86	2.86	17.90	178.28	0.24
4.6	0.00	7.80	7.80	2.81	15.97	194.25	0.26
5.6	0.01	7.75	7.75	2.75	14.39	208.65	0.28
6.6	0.01	7.69	7.70	2.70	13.12	221.77	0.31
7.5	0.01	7.64	7.65	2.65	12.06	233.84	0.33
8.4	0.01	7.59	7.59	2.59	11.16	245.00	0.34
9.2	0.01	7.53	7.54	2.54	10.38	255.37	0.36
10.0	0.01	7.48	7.49	2.49	9.69	265.06	0.38
10.8	0.01	7.43	7.44	2.44	9.07	274.14	0.40
11.6	0.01	7.38	7.39	2.39	8.52	282.66	0.41
12.3	0.01	7.32	7.34	2.34	8.02	290.68	0.43
13.0	0.01	7.27	7.28	2.29	7.56	298.24	0.45
13.7	0.01	7.22	7.23	2.23	7.14	305.38	0.46
14.4	0.01	7.17	7.18	2.18	6.75	312.13	0.48
15.1	0.02	7.12	7.13	2.13	6.38	318.51	0.49
15.8	0.02	7.07	7.08	2.08	6.04	324.55	0.51
16.4	0.02	7.01	7.03	2.03	5.72	330.28	0.52
17.0	0.02	6.96	6.98	1.98	5.42	335.70	0.54
17.7	0.02	6.91	6.93	1.93	5.14	340.84	0.55
18.1	0.02	6.88	6.89	1.89	3.42	344.26	0.56

X along weir (ft):	This is the distance along the weir starting at the downstream end (0.0).
Elmin (ft):	This is the main channel's invert elevation at station X.
Depth (ft):	This is the depth in the main channel at station X.
WS (ft):	This is the water surface in the main channel at station X.
Depth over Weir (ft):	This is the depth of flow over the weir at station X. It is the difference between the water surface and the weir crest elevation.
Weir Flow (cfs):	This is the weir discharge over the weir between station X and the previous station.
Channel Flow (cfs):	This is the net flow in the main channel between station 0 and station X.
Froude No.:	This is the Froude number in the main channel at station X.

Graph of the water surface data along the weir

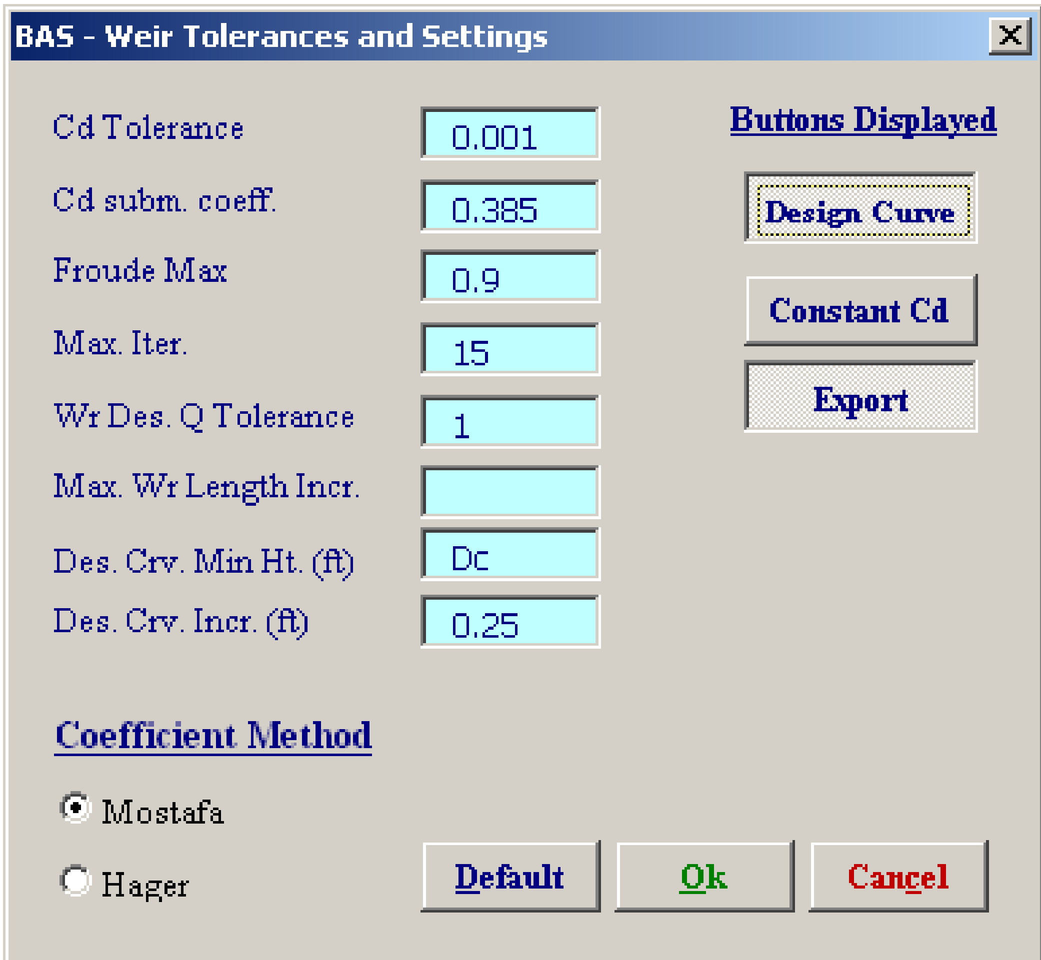
It is essential to view the water surface profile along the weir after each computation. To do so, select the **View\Graph** sub-menu command. Below is the graph for a weir design as seen in the **Preview** mode (black and white). On the screen, however, the graph of the weir and the water surface are presented in color. When finished previewing the graph, it is possible to either return to the table by pressing **View\Return to Table** or to go back to BAS's main sheet by pressing **View\Main Page**.



WEIR DESIGN (AND ANALYSIS) Tolerances and SETTINGS:

Several internal settings are provided within BAS for the design and analysis of side-weirs. These settings and tolerances provide options and limitations for the BAS program in order to avoid instabilities or excessive number if trials without convergence on a solution. However, it is strongly recommended that these settings not be modified without consultation with the Developer.

Below is a sample form and a description of each setting:



Cd Tolerance

In its default mode, BAS solves the spatially-varied-flow equation while including a varying coefficient of discharge in accordance with equations presented in the Features section of this chapter. This setting provides the maximum difference between one trial Cd value and the next. The default setting for this is 0.001

Cd. Subm. Coeff.

In the event that model studies are conducted which require adjustments of the power term in the Cd submergence correction term, this allows the user to utilize this adjustment. The default value is 0.385.

Froude Max.

This value sets the maximum Froude number allowed within the channel along the weir. The default value setting is 0.90. In the event that the user wishes to tolerate some instabilities along a weir, this value may be increased to a maximum of 0.95. On the other hand if the Froude number within the channel is not to exceed a certain value (less than 0.90), the user is capable of changing this limit in this field.

Max. Iter.

This sets the maximum number of iterations that BAS will perform while converging on a coefficient of discharge. The default value for this setting is 15.

Wr. Des. Q Tolerance

In the **Weir Design** mode, BAS determines the length of the weir needed to convey a specific flow over the weir. This tolerance sets the maximum difference between the desired weir flow and the calculated flow. Naturally, the smaller the tolerance, the longer BAS needs to converge on the solution.

Max. Weir Length Increment

This sets the upper limit of the computational increment along the side-weir. If left blank, BAS chooses the proper increment. However, for certain calculations where more accuracy is required, the setting may be reduced to a minimum of 0.5 foot.

Des. Crv. Min. Ht.

This sets the lower limit for the side-weir height during the **Design Curve** calculations. In its default mode, BAS automatically selects the critical depth for this limit. However, weir heights as low as 0.5 foot may be specified.

Des. Crv. Incr.

This sets the computational increment of weir height during the **Design Curve** calculations. In its default mode, BAS utilizes an increment of 0.25 foot.

Coefficient Method

This allows the user to select the weir discharge coefficient method. The default method is that of Mostafa.

DESIGN CURVE

In its default mode, BAS displays **DESIGN CURVE** button on the Weir Design form. However, if the user wishes, unselecting this button within the weir settings form will hide this button.

Constant Cd

In its default mode, BAS displays **Constant Cd** button on the Weir Design form. However, if the user wishes, unselecting this button within the weir settings form will hide this button.

EXPORT

In its default mode, BAS displays **EXPORT** button on the Weir Design form. However, if the user wishes, unselecting this button within the weir settings form will hide this button.

Chapter 4 - WEIR ANALYSIS

The design and analyses of side-weirs is one of the most complex hydraulic challenges. Numerous methods and equations have been developed in the past century for such analyses. Often the results between one method and another vary significantly. It is always prudent to fully comprehend the original technical papers that develop the methodology for such analyses. Once the engineer chooses and utilizes a method that he/she feels is most applicable, it is strongly recommended that a physical model be constructed to verify and fine-tune the design. Methods within **BAS** are no exception to this philosophy. For an overview of the development of the side-weir analyses, see Appendix C of this manual.

BAS's **Weir Analysis** component determines the amount of flow diverted over an existing weir. The user supplies the channel and weir parameters as well as the amount of submergence, if any, over the weir. Once the computations have been completed, BAS generates a comprehensive report of the weir analysis. This report includes the information supplied by the user (input data) as well as the results of the analysis within the report A table is presented which includes the water surface elevations, the depth and Froude Numbers along the length of the weir.

BAS determines an "effective weir length", which is the actual weir length used by the flow. There are two conditions which may occur when the full length of the weir is not utilized. In the event that a jump forms in the channel within the weir segment, the user is notified of this occurrence and the "effective weir length" is then limited to the location of the jump. Another case where the effective weir length is less than the user-specified length is when the water surface along the weir dips below the weir crest part way along the weir.

FEATURES:

The following is an overview of the features provided within the **Weir Analysis** component:

Spatially Varied Flow Analysis – The **Weir Analysis** component analyzes the water surface profile and discharge along a <u>sharp crested side-weir in a subcritical prismatic</u> channel. The computations utilize the classic spatially-varied-flow function as presented in Ven T. Chow's "Open Channel Hydraulics" and other hydraulic references.

$$\frac{dy}{dx} = \frac{S_o - S_f - \alpha Q q_* / gA^2}{1 - \alpha Q^2 / gA^2 D}$$ (Chow 1959)

where Q = channel discharge, S_o = channel slope, S_f = friction slope, dy/dx = change in water surface with respect to distance along weir, D = channel depth, A =flow area,

g = acceleration of gravity and $\alpha =$ velocity correction factor

$$q_* = \frac{dQ}{dx} = (-C_d(y - y_w)^{1.5})$$ (Mostafa 1987)

and q_* = side-weir's unit discharge, C_d = side-weir's coefficient of discharge,

y = channel depth at weir, y_w = weir height

Coefficient of Discharge – Studies have shown that a side-weir's coefficient of discharge varies with the change in weir discharge and flow depth. In its default mode, BAS adjusts the coefficient of discharge to account for this variance. The user may also utilize a constant coefficient of discharge that may be derived from other sources. The following equation provides an adjustment for the coefficient:

$$C_d = C' + 0.259C'(y - y_w)^2 / y^2 \quad \text{and} \quad C' = 0.60(Q_w / Q_o)^{0.167}$$ (Mostafa 1987)

where Q_w = weir discharge, Q_o = upstream discharge and C' = C_d adjustment based upon weir flow rate to the by-pass flow rate and y (and y_w) are as defined above.

BAS also includes a correction coefficient relationship as provided by Hager (1987). For technical details and applicability of the equation, please review the referenced paper.

Double Weirs – BAS allows for the design or analysis of one or two side-weirs (both sides of channel) in a single analysis.

Sloping Weirs – The slope of the weir may vary from that of the channel's invert slope.

Weir Submergence – In some cases, the water surface in the retarding basin may exceed the weir crest elevation and therefore the side-weir becomes "submerged". BAS adjusts the coefficient of discharge (C_d) to account for such submergence. The following equation accomplishes the submergence adjustment:

$$\frac{C_{ds}}{C_d} = \left[1 - \left(\frac{H_1}{H}\right)^{1.5}\right]^{.35}$$ (Mostafa 1974)

where C_{ds} = submerged-flow discharge coefficient, H_1 = submergence head, H = free head and C_d is as defined above

Flow Instability - In cases where the user-specified data results in an upstream weir discharge with its flow depth approaching critical depth, BAS terminates the computations and notifies the user that unstable flow has resulted at a specific location along the weir. The default setting for this condition is a Froude number of 0.90. However, this value may be reset to any Froude number between 0 and 0.95 by selecting **Settings\Weir Tolerances\Settings**.

Hydraulic Jumps - In the event that the channel's invert slope along the weir is supercritical while the flow regime at the downstream end of the weir is subcritical, BAS will compute the gradually varied water surface along the weir. BAS will then compare the Force (Pressure + Momentum) of the supercritical flow with the Force at each station along the weir. If the Force of the upstream (supercritical) flow equals the Force in the channel (subcritical) along the weir at a specific location, a hydraulic jump is assumed to occur at that location and the "effective weir length" is limited to the location of the jump. The upstream supercritical flows are assumed as uniform for this analysis.

Note: The location of the hydraulic jump is approximate. For an exact location of the jump, a physical model of the system is recommended.

Effective Weir Length - In the event that the water surface is reduced sufficiently to dip below the weir crest or if a hydraulic jump forms in the channel, no additional flow over the weir can be conveyed by extending the weir length past that point. Therefore, the "effective weir length" becomes the length along the weir from the downstream end to the location where the water surface dips below the crest.

Evaluation of Results – Once the design or analysis computations have been completed, BAS evaluates the results and notifies the user if the full weir length was utilized.

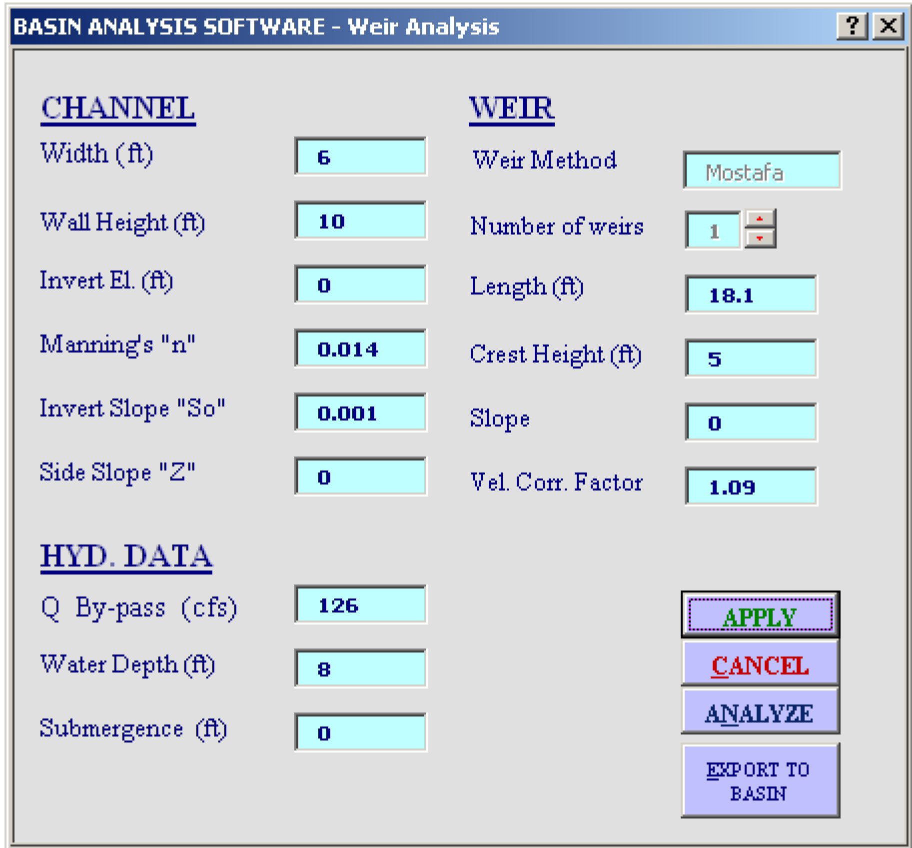
Export of Channel and Weir parameters - Once the computations have been completed, the user has the option of exporting the channel and weir data to the **Basin Analysis** component for future basin analyses.

Output Report - Detailed input and output data including color displays of the weir and the water surface profile along the weir are automatically generated by BAS.

THE WEIR ANALYSIS FORM:

The **Weir Analysis** form displayed below requires input of all the essential information for a successful analysis. The **APPLY**, **CANCEL** and **DESIGN** buttons are displayed each time the form is called. The **ANALYZE** button is grayed until the values provided on the form are acceptable and the **APPLY** button has been pressed.

The **Weir Analysis** component is relatively simple to use. However, to ensure proper input of the systems parameters, each input value and all of the selection options are defined below.



CHANNEL

Width (ft):	The main channel's base width.
Wall Height (ft):	The main channel's wall height upstream and downstream of the side-weir.
Invert El. (ft):	The main channel's invert elevation at the downstream end of the weir.
Manning's "n":	The main channel's Manning's coefficient of friction.
Invert Slope "So":	The main channel's longitudinal invert slope along the weir.
Side Slope "Z":	The main channel's side-slope ratio.

WEIR

Weir Method:	Indicates the current specified weir coefficient method, i.e. Mostafa or Hager.
Number of weirs:	Two identical weirs may be modeled opposite each other.

Crest Height (ft): The vertical distance from the channel's invert to the weir crest at the downstream end of the weir.

Slope: The weir crest slope along the main channel.

Vel. Corr. Factor: The velocity correction factor accounts for the variation of the velocity within a channel cross-section along the weir.

HYD. DATA

Q By-Pass (cfs) : The channel's by-pass flow just downstream of the side-weir.

Water Depth (ft): The depth of water at the downstream end of the side-weir (corresponding to Q By-Pass (cfs).

Submergence (ft): The depth of submergence over the side-weir. This occurs when the basin's water surface exceeds the weir crest.

Constant Cd

The **Constant Cd** button is used when the user desires to use a constant coefficient of discharge for the analysis of the weir. This button as well as the other button displayed at the lower right hand corner of the form are optional and may be turned on or off through the **Settings\Weir Tolerances_Settings** sub-menu.

Export to Basin Analysis

The **Export to Basin Analysis** button is used to transfer the channel and weir data to the Basin Analysis component for future computations.

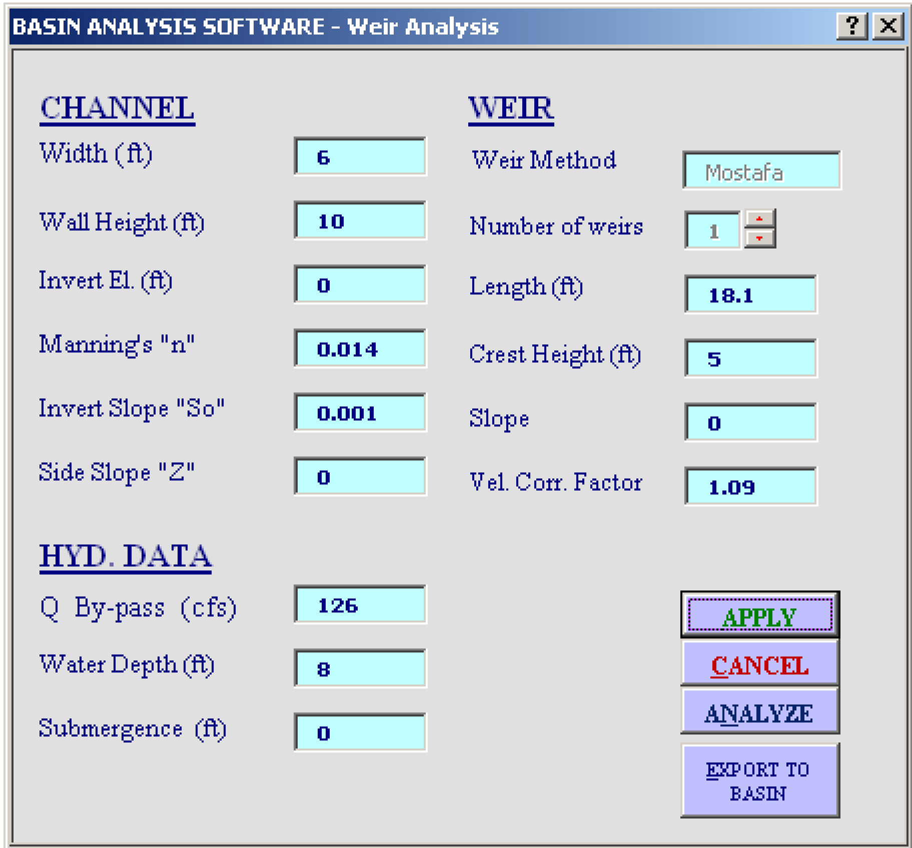
A QUICK GUIDE TO RUNNING THE WEIR ANALYSIS:

- Enter the main channel's width, wall height, invert elevation, Manning's friction factor, invert slope and side-slope
- Enter the side-weir's number of weirs, weir crest height, weir crest slope, length and the velocity correction factor
- If the analysis is based upon a known coefficient of discharge, press the **Constant Cd** button and enter its value
- Enter the hydraulic data for the system. This includes the Q By-pass, the depth of water in the channel at the downstream end of the weir and the submergence, if any.
- Press the **APPLY** button
- Press the **ANALYZE** button

A WEIR ANALYSIS EXAMPLE:

In order to demonstrate the use of the **Weir Analysis** component, a detailed example is provided below. The following sample problem is identical to that presented for the Weir Design section. However, in this case, the weir length is known and the weir discharge needs to be determined.

A development is proposed along a creek, which is anticipated to increase the flows in the creek from 130.3 cfs to 346 cfs. The conditions placed upon the development, in part, require no increase in discharge in the downstream existing channel. Therefore, retarding was necessitated within the development. After comparing the area required for an online retarding basin (2.66 acre-ft) to that of an offline retarding basin (1.63 acre-ft), the engineer decided to implement an offline system. To accomplish the diversion, an 41-foot long, six-foot high side-weir was designed along a 6-foot wide channel. However, initially an 18.1-foot long, five-foot high weir was used. To verify the function of the side-weir, the BAS **Weir Analysis** component is utilized. All of the system's parameters are presented in the **Weir Analysis** form below:



Next the **APPLY** button and the **ANALYZE** button were pressed and a **Weir Analysis** report was generated.

WEIR ANALYSIS REPORT:

BAS uses the same report for both the **Weir Design** as well as the **Weir Analysis** components. For details, see the Weir **Design** section.

WEIR ANALYSIS

CHANNEL DATA			WEIR DATA		
Q-Bypass (cfs)	=	126.0	Q-Weir (cfs)	=	218.3
Width (ft)	=	6.00	Crest Slope	=	0.00000
Longit. Slope	=	0.00100	Crest Length	=	18.1
Wall Height (ft)	=	10.00	Crest Height (f	=	5.00
Manning's(n)	=	0.014	Cd (Mostafa)	=	0.569
Water Depth (ft	=	8.00	Subm. (ft)	=	0.00
Vel. Correction	=	1.09	Eff. Length (ft)	=	18.1
Invert El. (ft)	=	0.00	No. of weirs	=	1
Side Slope (z)	=	0.0			
Q-Upstream (cl	=	344.3			

X along weir (ft)	Elmin (ft)	Depth (ft)	WS (ft)	Depth over Weir (ft)	Weir Flow (cfs)	Channel Flow (cfs)	Froude No.
0.0	0.00	8.00	8.00	3.00	0.00	126.00	0.16
0.8	0.00	7.97	7.97	2.97	12.48	138.48	0.18
2.2	0.00	7.91	7.91	2.91	21.90	160.38	0.21
3.5	0.00	7.85	7.86	2.86	17.90	178.28	0.24
4.6	0.00	7.80	7.80	2.81	15.97	194.25	0.26
5.6	0.01	7.75	7.75	2.75	14.39	208.65	0.28
6.6	0.01	7.69	7.70	2.70	13.12	221.77	0.31
7.5	0.01	7.64	7.65	2.65	12.06	233.84	0.33

WEIR ANALYSIS (AND DESIGN) SETTINGS:

BAS uses the same settings and tolerances for both the **Weir Design** as well as the **Weir Analysis** components. For details, see the **Weir Design** section.

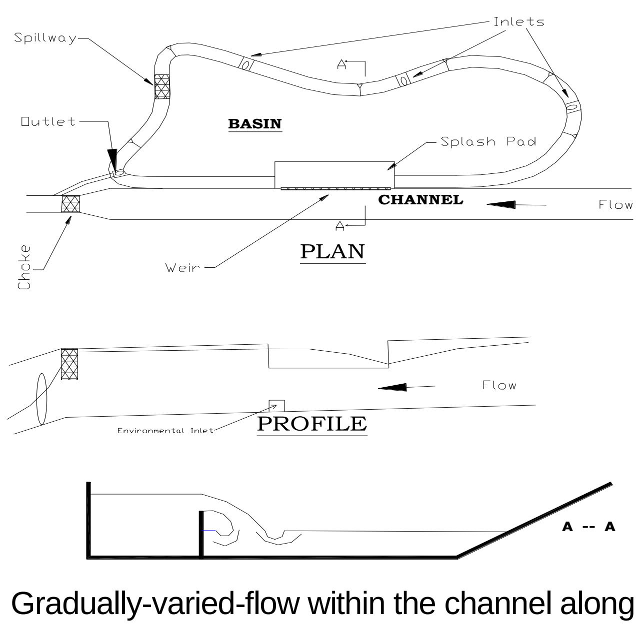
Chapter 5 - BASIN ANALYSIS

The design and analyses of side-weirs is one of the most complex hydraulic challenges. Numerous methods and equations have been developed in the past century for such analyses. Often the results between one method and another vary significantly. It is always prudent to fully comprehend the original technical papers that develop the methodology for such analyses. Once the engineer chooses a method that he/she feels is most applicable, it is strongly recommended that a physical model be constructed to verify and fine-tune the design. Methods within **BAS** are no exception to this philosophy. For an overview of the development of the side-weir and other basin appurtenances analyses, see Appendices C through E of this manual.

The Basin Analysis component is by far the most dynamic component of the software. A complete analysis of an offline system including the channel, side-weir, outlet(s), inlet(s) and spillway is performed for each hydrograph time-step. This type of analysis is termed quasi-unsteady since, although it is time dependent, it treats each time-step as a steady flow increment.

BASIN ANALYSIS OVERVIEW

A typical offline retarding system usually includes the components presented in the figure below. An overview of the Basin Analysis features within BAS is also presented below the figure.



- Gradually-varied-flow within the channel along the weir.

- Spatially varied flow over a sharp-crested weir (or two) adjacent to a subcritical prismatic channel.
- Basin outlet flows – This is a classic pipe outlet where Inlet control, outlet control, pressure, orifice and open-channel flows are analyzed. Several options are available for enhancing the analysis of the outlet's hydraulics. The user may specify Flapgates, which allow the water to flow out of but not into the basin. The user may also specify an automatically operated gate that optimizes the functioning of the basin's outlet. This type of gate, named **Auto Gate** in BAS, reduces the flows of the outlet when a targeted channel flow downstream of the outlet is exceeded. This gate gradually chokes the outlet flows or allows for more flows depending upon sensors placed in the downstream channel. Finally, the outlet may be analyzed for partial or full blockage due to debris accumulation or if the gate is mechanically controlled.
- User-defined outlet flows – There may be cases where the basin's outlet involves complexities which are beyond the capabilities of BAS. Or, the basin may include an outlet shape that is not round (pipe). The User-defined outlet provides an ability for the user to include a stage-discharge relationship for outflows. Hand calculations, nomographs or other hydraulic software may be used in constructing this relationship.
- Pump outlet flows – There may be cases where a pump is needed to assist in the draining of the basin or even as the primary (and only) outlet. The user may specify the maximum pump flow as well as the "on" and "off" elevations. The pumped flows may be directed back to the downstream channel (at the outlet location) or away from the system.
- Spillway flow – If the water surface in the basin exceeds a user specified spillway elevation, spillway flows are calculated and included in the systems calculations. The user may utilize an Ogee-crest or a Sharp-crest spillway.
- Environmental Inlet Flows – BAS allows the channel wall along the weir to includes an opening for inletting (or outletting) flows into (or out of) the basin. Such openings may be utilized for water quality purposes. Obviously, such openings also allow a portion of the lower stages of the hydrograph to also enter the basin and thus deplete a portion of the available flood storage. When the opening is kept small, a basin which combines environmental as well as flood protection purposes, may be designed. BAS includes the effects of such an inlet in each of the hydrograph time-steps.
- Inlets – Often, inflows from storm drains into a side-basin are ignored to facilitate the basin's analysis. Consequently, these seemingly small flows over several hours amount to a significant volume requirement. BAS allows for the inclusion of 3 such inlets into a basin. In order to

realistically model their function, the user may input a complete hydrograph for each inlet. In addition, if the surface area of a basin is large, the user may opt to model the precipitation hydrograph over the basin as one of the inlets.

These quasi-unsteady calculations optimize the hydraulics of the basin-channel system and should result in a very realistic understanding of the system's operation.

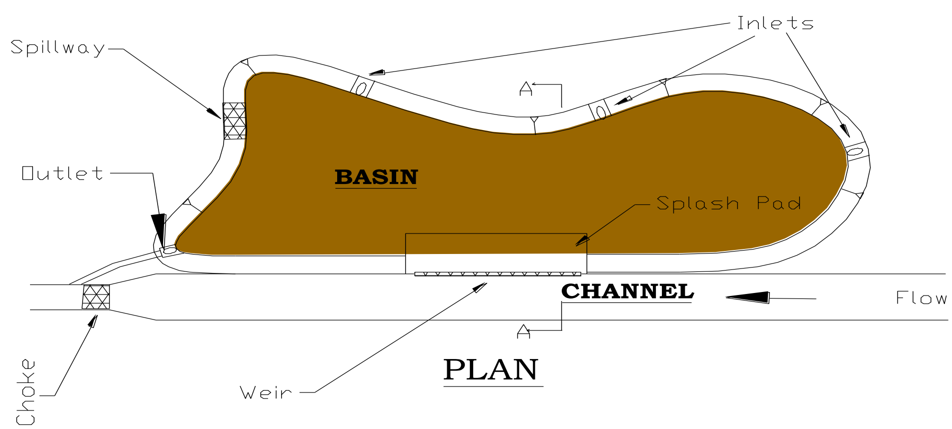
BASIN ANALYSIS FEATURES:

Types of Weir Flow

The spatially-varied-flow features presented in the **Weir Design** and **Weir Analysis** chapters are utilized for the **Basin Analysis** component and will not be presented in detail herein. However, there are several types of flow which may occur in a channel and its side-weir during a hydrograph cycle. The following briefly describes each weir flow type:

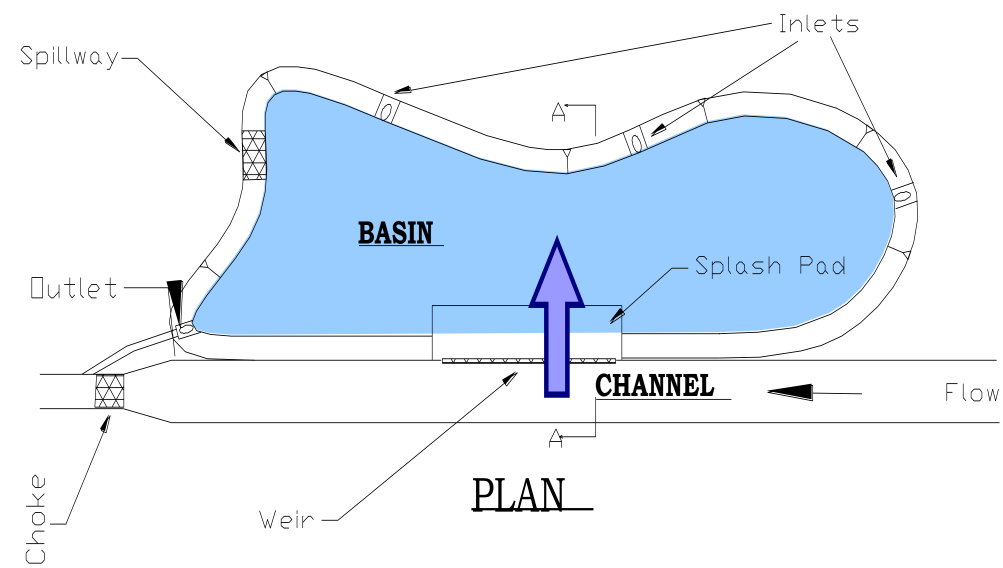
NO FLOW INTO BASIN

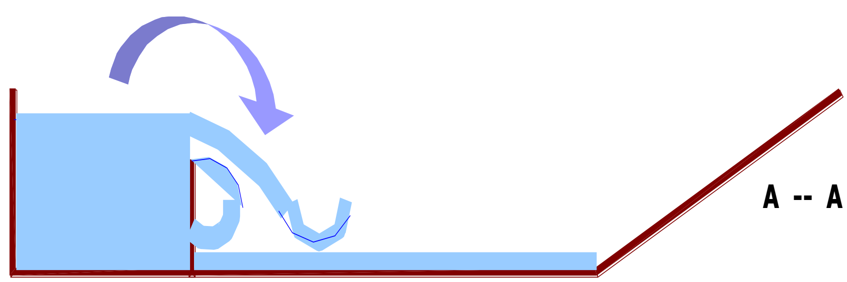
The water surface elevations in the basin and channel are below that of the weir crest and no flow occurs over the weir. BAS calculates the gradually varied water surface in the channel that is used to determine the flow, flow direction and the hydraulic controls of the outlet(s).



UNSUBMERGED-FLOW INTO THE BASIN

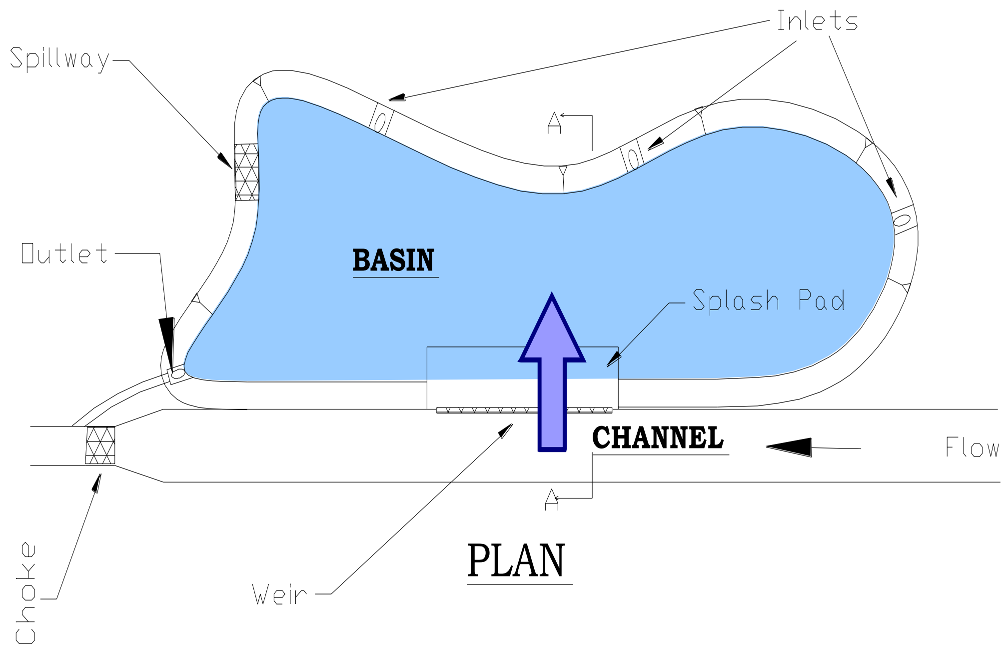
Unsubmerged-flow – Water flows from the channel and into the basin where the water surface elevation in the basin is below the weir crest elevation. BAS's **Weir Analysis** component is accessed during this condition.





SUBMERGED-FLOW INTO THE BASIN

Submerged-flow – Water flows from the channel and into the basin where the water surface elevation in the basin is above the weir-crest elevation (submerged crest). BAS's Weir Analysis component is accessed during this condition.



UNSUBMERGED REVERSE FLOW (Into Channel)

Unsubmerged Reverse Flow – Water flows from the basin and into the channel where the water surface elevation in the channel is below the weir crest. The weir analysis in this case utilizes the standard (not spatially varied) weir flow function with the water surface in the basin assumed horizontal. The following formula is used:

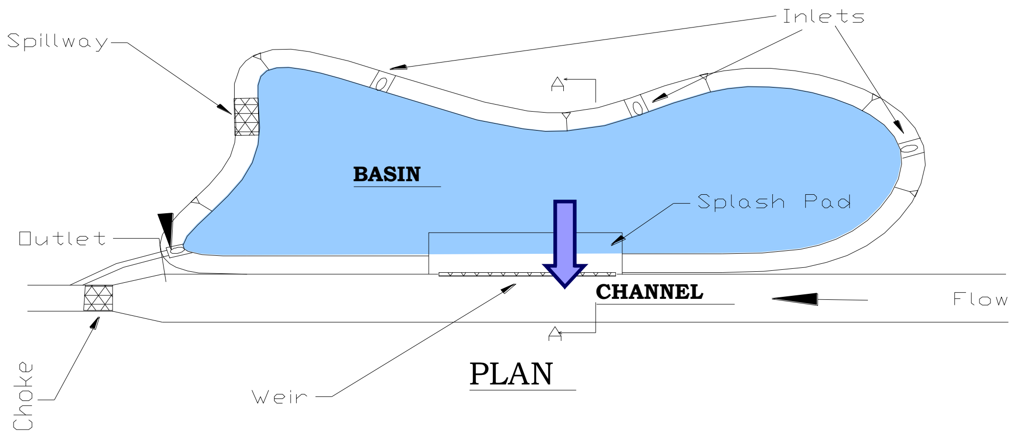
$$Q = CLH^{3/2}$$ (Brater & King 1963)

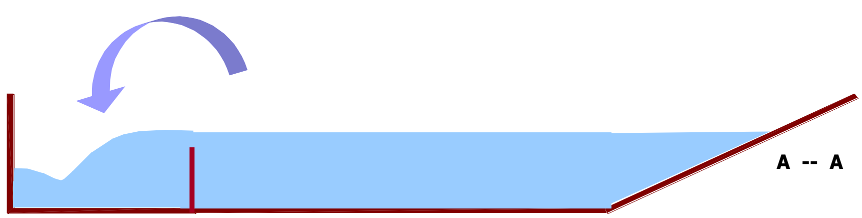
where

$$C = 3.22 + 0.44(H/Y_w)$$ (Brater & King 1963)

and L = weir Length, H = flow depth over weir, Y_w = weir height and

C = coefficient of discharge





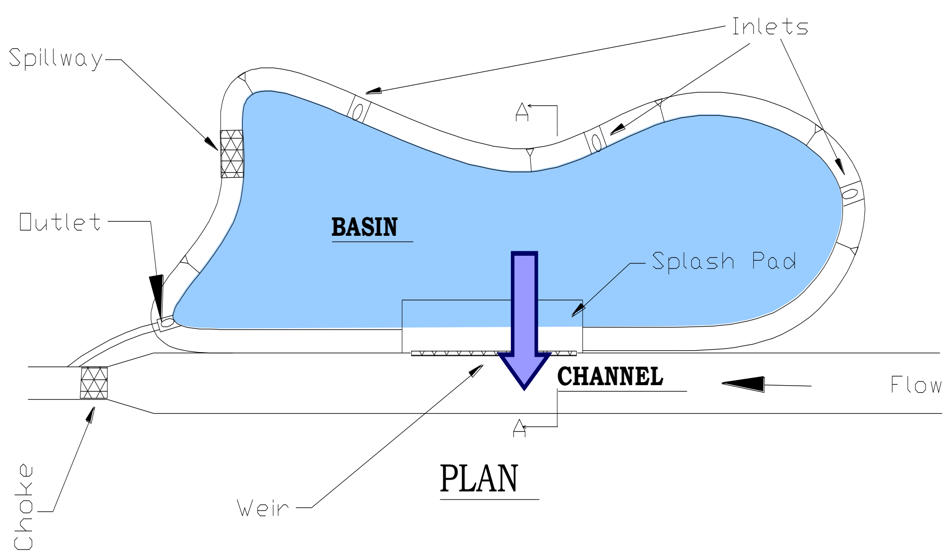
SUBMERGED REVERSED FLOW (Into the channel)

Submerged Reverse Flow - Water flows from the basin and into the channel where the water surface elevation in the channel is above the weir crest elevation (submerged crest). The weir analysis in this case utilizes the same equation as that utilized in the unusbmerged reverse flow with the following correction to account for submergence:

$$\frac{Q}{Q_1} = \left[1 - \left(\frac{H_2}{H_1}\right)^{1.5}\right]^{0.385}$$

(Brater & King 1963)

Where Q=submerged discharge, Q_1=free discharge, H_1=free head, H_2=submerged head



Basin Outlet

The outlet of a basin plays a major role in the determination of the volume stored in the basin. BAS provides several options for outlet analyses that are provided below.

The outlet's hydraulic analyses are based upon established hydraulic theory of culvert flows and no detailed discussion of the theories is presented herein. The United States Bureau of Reclamation's Design of Small Dams text includes such basic theory.

In general, the outlet may flow under pressure or as an open-channel depending upon the water surfaces in the basin and the channel. The hydraulic control may occur at the inlet or at the outlet where the inlet may at times function as an orifice. Flow at the downstream end of the outlet may cross through critical as it outlets into the channel. In such a situation (where the outlet's slope is mild), the hydraulic control would be at critical depth of the outlet. For steep sloped outlets, where the flow is assumed to flow supercritical and the depth at the basin (end of outlet) is less than 1.2*Diameter, the control occurs at the inlet (approx. 0.72*Dc). For a general understanding of outlets, see Appendix D of this manual.

The following is an overview of the options and features included basin outlet analysis:

Multiple Pipes

The basin may include up to 10 outlet pipes. The pipes will be identical and are all described in the same form.

Location

The confluence location of the outlet with the main channel, in most cases, will be downstream of the channel's choke as shown in the figure presented at the beginning of this chapter. However, there may be cases where that is not feasible. BAS provides the user an option to locate the outlet either downstream or upstream of the channel's choke section. This option is provided on the **Basin Analysis \Outlets** tab.

Flow Direction

The flow through the outlet may be directed into the basin or into the channel depending upon their respective water surface elevations at each hydrograph time increment.

Hydraulic Control

Inlet and outlet hydraulic controls are analyzed at each hydrograph time increment.

Pressure or Open-Channel Flow

The flow in the outlet may be pressure flow or open-channel flow depending upon the relationships between the water surface elevations in the basin and channel, and the outlet's soffit elevation.

Flapgates

In some cases, flap-gates may significantly reduce the storage volume requirements by preventing reverse-flows into the basin through the outlet. Typically such flows occur when the basin is empty and the channel flow depth is rising above the invert elevation of the basin. The user has the option to include flap-gates in the analysis by pressing the Flapgates button.

Pluggage

At times, debris may accumulate either in the main channel at the basin's outlet or within the basin at the outlet. Such accumulation of debris reduces the outlet's capacity and will consequently result in a larger retarding volume requirement for the basin. It is advisable to check the effect of pluggage on the required basin volume. BAS allows the user to choose pluggage values between 0 and 100 percent by pressing the Percent Plugged button.

Entrance Configuration

The entrance configuration of the outlet may be either well rounded or sharp edged. Either configuration may be selected in order to better approximate the energy losses. The smooth edged entrance utilizes a minor loss coefficient of 0.2 while the sharp edged entrance utilizes a minor loss coefficient of 0.5.

Additional Losses

The outlet pipe(s) may include other energy losses such as those associated with bends, manholes, etc. The user has the option to include "Additional Loss Factors" in the analysis by inputting the net loss factor in this field.

Auto Gate

The outlet may include an automatically operated gate. Electronic sensors in the main channel would typically operate such gate. The purpose of this gate is to reduce the outlet(s) flows if the downstream channel flows ever exceed a specified maximum flow. For example, if the channel flow-bypassing the weir is 4300 cfs and the desired channel flow downstream of the outlet is 4400 cfs, the outlet gate partly closes to only allow 100 cfs maximum flow to pass through the outlet. Further, if the weir's by-pass flow exceeds 4400 cfs, the outlet's gate completely closes preventing flows from neither exiting nor entering the basin through the outlet. The Auto Gate feature allows only flows to exit the basin. That is, it automatically operates as a one-way valve, much like a flapgate.

UD Outlet

This option includes the two capabilities below. Only one of those may be selected at a time.

1. **User-Defined** Outflow – There may be cases where the basin's outlet involves complexities which are beyond the capabilities of BAS. Or, the basin may include an outlet shape that is not round (pipe). In this, the user chooses to utilize a specific outlet stage-discharge relationship. Hand calculations, nomographs or other hydraulic software may be used in constructing this relationship This outlfow device may be set at a different elevation than the standard pipe outlet.
2. **Pump** Outflow - There may be cases where a pump is needed to assist in the draining of the basin or even as the primary (and only) outlet. In this, the user chooses to utilize a pump instead of the User-Defined outflow. The user then chooses the water surface elevations for the "Pump On" and Pump Off" along with a maximum pump discharge. The pumped flows may be directed back to the downstream channel (at the outlet location) or away from the system. The latter assumes flows are directed out of the basin to another impoundment or water course and are completely lost to the system.

Inlets

The basin may receive flows from other sources besides the side-weir and reverse-flow in the outlet(s). These additional flows usually originate from tributary facilities flowing directly into the basin or from precipitation over the basin. Up to three hydrographs are allowed for such flows into the basin.

Environmental Inlet

Another type of inlet, which BAS allows for inclusion into the analysis, is a window opening in the channel wall separating the basin and the channel. This low-flow "Environmental Inlet" which acts as an orifice, allows for the entry of the main channel's low-flows into the basin for water quality purposes.

Spillway

- Most basins are required to include an emergency spillway. BAS considers the overall hydraulic balance of the system that includes the effects of flows exiting the system over the emergency spillway. The user may utilize an Ogee-crest or a Sharp-crest spillway. It should be noted that flows over the spillway are not returned to the downstream channel and are assumed lost to the system. For a general understanding of spillways, see Appendix E of this manual.

Basin Geometry

The user typically provides the stage-volume relationship. This relationship is critical in the basin storage calculations and for determining if the weir becomes submerged or if the weir flow reverses into the channel. To simplify preliminary design in the planning stages, the user may specify a rectangular base with sloping banks. The following describes each basin geometry scenario:

- User Defined: This is a water surface versus volume relationship in the basin. This data may be arrived at by use of terrain-modeling software or any other convenient means.
- Rectangular: This shape is recommended for planning purposes only. The user needs to supply the length and width of the base, the basin's side-slopes and the basin's base-elevation.

Output Report

BAS generates a comprehensive report of the routing analysis. The report includes all input data and resulting calculations in tabular and graphical form. The user may choose to print portions of the output for evaluation of an analysis or design.

BASIN ANALYSIS SETTINGS:

The Basin Analysis Settings are limitations utilized within BAS in order to efficiently arrive at a solution without instabilities in the calculations.

Note: It is highly recommended that the settings do not get modified without consultation with the Developer. In the event that settings are modified and the user wishes to return to the original settings, pressing the **Default** button will accomplish the restoration of the settings.

The following is a brief description of each setting:

WEIR

Modify Weir Settings

Parameters within the WEIR category are only displayed in this form. To modify any of the parameters, press this button to reach the Weir Tolerances and Settings form. Another way to reach the same form is to press **Settings/Weir Tolerances/Settings** sub menu. Definitions of parameters are included in *Chapter 2 – Weir Design (and Analysis) Settings*.

SUBM. WEIR

Q Tolerance (% Qu/s)

This setting provides the maximum difference between the actual upstream flow (inflow) and the calculated flow during submergence of the weir. The default setting for this is .02% of the upstream flow.

Q adjustment (% Q dif.)

During the calculations of the submerged weir, numerous trials are computed by BAS. Each trial adjusts the flow by this percentage. The default value is 10%.

Max Iterations

Should BAS perform an excess number of calculations during the submergence routine, this sets an upper limit on the number of trials. The default value is 100 trials.

W.S. Balanced @ submerged

Instabilities in the calculations may occur when the water surface in the basin approaches that of the channel. This instability occurs in a situation where the water surface in the channel exceeds that of basin. BAS calculates a weir discharge for this case. However, as the volume of this discharge is added to the basin's volume, the water surface elevation in the basin will exceed the channel. Since this occurs within the same time increment, BAS will keep cycling without converging on a solution. To overcome that, a tolerance for the water surface difference between the channel and the basin is set here. When this difference is less than this specified limit, BAS will set the water surface equal to each other and the calculations resume to the next time increment. The default value for this setting is 0.001 foot.

REV. FLOW

Q Tolerance (% Qu/s)

This setting provides the maximum difference between the actual upstream flow (inflow) and the calculated flow during reversed flows over the weir. The default setting for this is 0.5% of the upstream flow.

Q adjustment (% Q dif.)

During the calculations of the reversed weir flow, numerous trials are computed by BAS. Each trial adjusts the flow by this percentage. The default value is 0.5%.

Max Iterations

Should BAS perform an excess number of calculations during the reverse-flow routine, this sets an upper limit on the number of trials. The default value is 500 trials.

Q Tol. For No Flow (%Q u/s)

Instabilities in the calculations may occur when the water surface in the basin approaches that of the channel. This instability occurs in a situation where the water surface in the basin exceeds that of the channel. BAS calculates a reverse weir discharge for this case. However, as the volume of this discharge is subtracted from the basin's volume, the water surface elevation in the basin becomes lower than that of the channel. Since this occurs within the same time increment, BAS will keep cycling without converging on a solution. To overcome that, a tolerance for the water surface difference between the channel and the basin is set here. When this difference is less than this specified limit, BAS will set the water surfaces equal to each other and the calculations resume to the next time increment. The default value for this setting is 0.4% of the upstream discharge.

W.S. Balanced @ submerged

Instabilities in the calculations may occur when the water surface in the basin approaches that of the channel. This instability occurs in a situation where the water surface in the basin exceeds that of the channel. BAS calculates a reverse weir discharge for this case. However, as the volume of this discharge is subtracted from the basin's volume, the water surface elevation in the basin becomes lower than that of the channel. Since this occurs within the same time increment, BAS will keep cycling without converging on a solution. To overcome that, a tolerance for the water surface difference between the channel and the basin is set here. When this difference is less than this specified limit, BAS will set the water surfaces equal to each other and the calculations resume to the next time increment. The default value for this setting is 0.001 foot.

BASIN

Target WS Change

The value in this field is used during the insertion of intermediate hydrograph points prior to the analysis of the system. This interpolation helps ensure that the time increments used in the analysis are not excessive. Thus, this contributes substantially to the stability of BAS. The default value for this setting is 0.1 foot.

Target dQ @ Outlet (cfs)

The value in this field is also used during the insertion of intermediate hydrograph points prior to the analysis of the system. This interpolation helps ensure that the time increments used in the analysis are not excessive. Thus, this contributes substantially to the stability of BAS. The default value for this setting is 0.1 cfs.

HYDROGRAPH

Max. Time Incr. (hrs)

Similar to the BASIN settings above, setting an upper limit on the hydrograph's time increment reduces the possibility of instabilities in the calculations. The default value for this setting is 0.25 hours.

Max. Q chg. (% Q u/s)

Similar to the Max**. Time Incr** setting above, setting an upper limit on the difference in discharge between one hydrograph time-step and the next, reduces the possibility of instabilities in the calculations. The default value for this setting is 1% of the upstream discharge.

PARTIAL RESULTS

In the event that the computations stop due to deficient basin volume or insufficient discharge values in the rating curves at the weir and/or the outlet, choosing this option prompts BAS to compile a report which includes all the calculations up to the point the program stops.

Interpolate Hydrograph (Max)

In its default mode, BAS includes routines which insert hydrograph time increments at critical hydraulic times which enable more accurate modeling of a system. At times, such interpolation may add excessive points which may exceed the number of hydrograph points allowed. Un-checking this box, precludes such interpolation.

BUTTONS DISPLAYED

DESIGN CURVE

In its default mode, BAS displays the **DESIGN CURVE** button on the Weir Design form. However, if the user wishes, unselecting this button may hide it.

Constant Cd

In its default mode, BAS displays **Constant Cd** button on the **Weir Design** and **Weir Analysis** forms. However, if the user wishes, unselecting this button may hide it.

EXPORT

In its default mode, BAS displays **Export to Basin** button on the **Weir Design** and **Weir Analysis** forms. However, if the user wishes, unselecting this button may hide it.

Chapter 6 - DEVELOPING A BASIN ANALYSIS MODEL

The design and analyses of side-weirs is one of the most complex hydraulic challenges. Numerous methods and equations have been developed in the past century for such analyses. Often the results between one method and another vary significantly. It is always prudent to fully comprehend the original technical papers that develop the methodology for such analyses. Once the engineer chooses and utilizes a method that he/she feels is most applicable, it is strongly recommended that a physical model be constructed to verify and fine-tune the design. Methods within **BAS** are no exception to this philosophy.

QUICK GUIDE TO A SUCCESSFUL BAS RUN

Pre-BAS Analyses

Prior to beginning any basin analysis, the following data is required and needs to be generated separately from BAS:

- Inflow hydrograph in the main channel at the upstream end of the weir. This hydrograph is the result of a hydrological evaluation of the watershed upstream of the retarding basin.
- Stage-discharge relationship in the main channel at the downstream end of the weir. This relationship is the result of a hydraulic analysis of the channel system downstream of the weir.
- Stage-discharge relationship in the main channel at the basin's outlet.
- Stage-volume relationship in the basin.
- Other inflow hydrographs into the basin.

BAS Analysis

Below is a brief overview of the steps required to successfully analyze a BAS project.

Create a new BAS project file by selecting the **File/New Project (Template)** menu command

Run the **Design Curve** Utility to arrive at a range of weir heights and lengths that result in the desired flow reduction in the main channel.

Choose a weir height and length from the **Design Curve** table.

Select the **Edit/Main Hydrograph** command and enter the channel's hydrograph in the table

Select the **Edit/Q-WS @ Weir** command and enter the channel's stage-discharge relationship at the downstream end of the weir

Select the **Edit/Q-WS @ Outlet** command and enter the channel's stage-discharge relationship at basin's outlet. This data is needed only when the outlet is located downstream of the channel's choke

Select the **Edit/WS-Vol** command and enter stage-volume relationship at the basin

Select **Simulate/Basin Analysis** and display the Basin Analysis input form

Select the **Weir** tab and enter the weir parameters data

Select the **Channel** tab and enter the main channel parameters

Select the **Outlet** tab and enter the basin outlet's parameters

Select the **Spillway** tab and enter the emergency spillway parameters (if included in the system)

Select the **UD-Outlet** tab and select either a User-Defined outlet or a pump. Then enter the required parameters.

Select the **Inlets** tab and enter the basin's other Inlets' hydrograph data (if included in the system)

Within the **Inlets** tab, enter the **Environmental Inlet**'s data (if included in the system)

Select the **Apply** button to accept the data

Select the **Analyze** button to run BAS

Note: Depending upon the duration of the hydrograph as well as the size of the basin, the analysis may absorb several minutes. A longer period is required when the basin reaches its capacity and its water surface submerges the weir crest. Similarly, when the flow has reversed its direction over the weir (from basin to channel), and the water surface in the channel is above the weir crest, the calculations require more iterations and thus more calculation time.

DETAILED GUIDE TO A SUCCESSFUL BAS RUN

The following sections describe in detail the steps required to start a new project for the BAS phase of the analysis:

Step 1 - Create a new project file

From the main menu select **File\New Project (Template)**. It is recommended that this option be selected instead of the Create New Project command. The latter absorbs several minutes as VBA and Excel create the required sheets and format their interior.

Step 2 – Run the Design Curve utility

In order to run the **Basin Analysis** feature, a weir length and height needs to be determined. Initially, the user will not have this information. BAS's **Design Curve** utility provides the user with a range of weir lengths and heights, which achieve the desired peak, flow reduction. To run this utility, select **Simulate/Weir Design** menu command. Then, enter the miscellaneous parameters needed for this component (for details, please see the **Weir Design** chapter). Note: the weir height may be left blank at this point in time since BAS will determine a series of weir lengths and heights that may be utilized.

Step 3 – Choose a weir length and height

Step 2 provides the Design Curve table with a series of weir lengths and heights that may be utilized. However, at this point, the user will not be able to determine the optimal parameters. This is accomplished later after running BAS for several combinations of these parameters. In general, it is common to begin with the highest weir value as this will result in the smallest volume requirement in the basin. However, such lengths are also excessive and may be quite costly. Cost considerations as well as the hydraulic behavior of the system will eventually yield an optimum system design.

Step 4 - Enter the main channel hydrograph

BAS requires the main channel hydrograph at the upstream end of the weir to be supplied. This data consists of the Time (in hours) and the Discharge (in cubic feet per second) for a specific time-period. The main channel hydrograph table may accessed by either selecting, from the main menu, the **Edit/Hydrograph** Data or by selecting the **Edit Hydrograph** from the **Basin Analysis/Channel** tab.

Step 5 - Enter the main channel Stage-Discharge at downstream of weir

BAS requires the main channel stage-discharge at the downstream end of the weir to be supplied. This table represents the data needed in order to compute various hydraulic parameters in the channel, along and over the weir. The data consists of a series of water surface Elevations (in feet) and their corresponding discharges (in cubic feet per second). It is recommended that several sets of stage-discharge values be provided at a maximum water surface elevation interval of 0.5-feet. Or, the rating table may start at a discharge of 5 percent of the proposed by-pass discharge at the downstream end of the weir and move up by five percent increments up to 105 percent. The data may be constructed through the use of any acceptable hydraulic model such as the Corps of Engineers' HEC-2 or HECRAS. Accuracy in this table is paramount for the accurate simulation of the weir hydraulics. This table may accessed by either selecting the **Edit/Q-WS @ Weir** or by selecting the **Edit Q-WS @ Weir** from the **Basin Analysis/Channel** tab.

Step 6 - Enter the main channel Stage-Discharge at the basin's Outlet

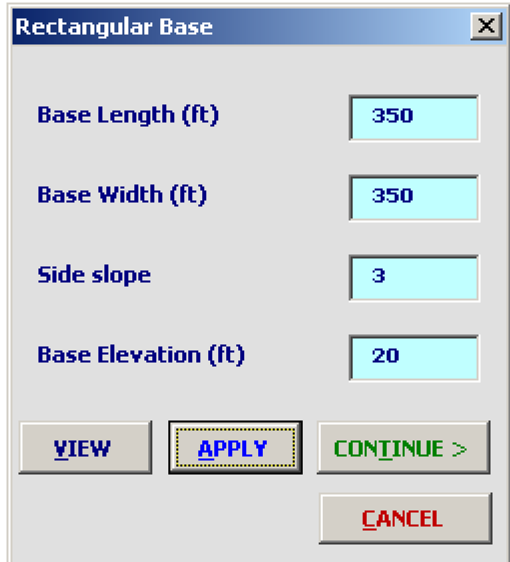
BAS requires the main channel stage-discharge at the basin's outlet to be supplied. This data consists of a series of water surface elevations (in feet) and their corresponding discharges (in cubic feet per second). Similar to the data supplied for the Stage-Discharge at the downstream end of the weir, this data is also calculated separately and then supplied to BAS. For example, the channel system downstream of the outlet may be modeled using HECRAS, HEC-2, WSPG or any other software (or hand calculations) that is capable of determining the stage for a specific discharge. Typically one gradually-varied-flow model is constructed which begins at the downstream end of the side-weir and ends at an appropriate control location downstream of the basin's outlet confluence with the main channel.

This table may accessed by either selecting the **Edit/Q-WS @ Outlet** or by selecting the **Edit Q-WS @ Outlet** from the **Basin Analysis/Channel** tab.

Step 7 – Enter the stage-volume relationship at the basin

BAS requires all pertinent basin stage-volume data to be supplied. For preliminary design, the user may choose to utilize a rectangular basin. The input requirements for this case is minimized to avoid time consuming data input. However, once a basin configuration has been decided upon, precise stage-volume data would yield more accurate system analysis results. The data input is described in more detail below. The starting water surface elevation field (on the **Basin Analysis** input form) is not a required input field. If left blank, BAS assumes a completely empty basin and places the lowest basin elevation in the field.

- Rectangular - Selecting this toggle-button assumes a rectangular-based basin. The data required for this option includes: the basin's base width and length, side-slopes and the base elevation (ft). This form may accessed by selecting Rectangular and then Edit Rectangular Basin from the Basin Analysis/Basin tab.



- User Defined - When selecting this toggle-button, BAS requires input of a series of Elevations (in feet) and their corresponding volumes (in acre-feet). This table may accessed by either selecting the **Edit/WS-Vol** or by selecting the **Edit User Defined Basin** from the **Basin Analysis/Basin** tab.

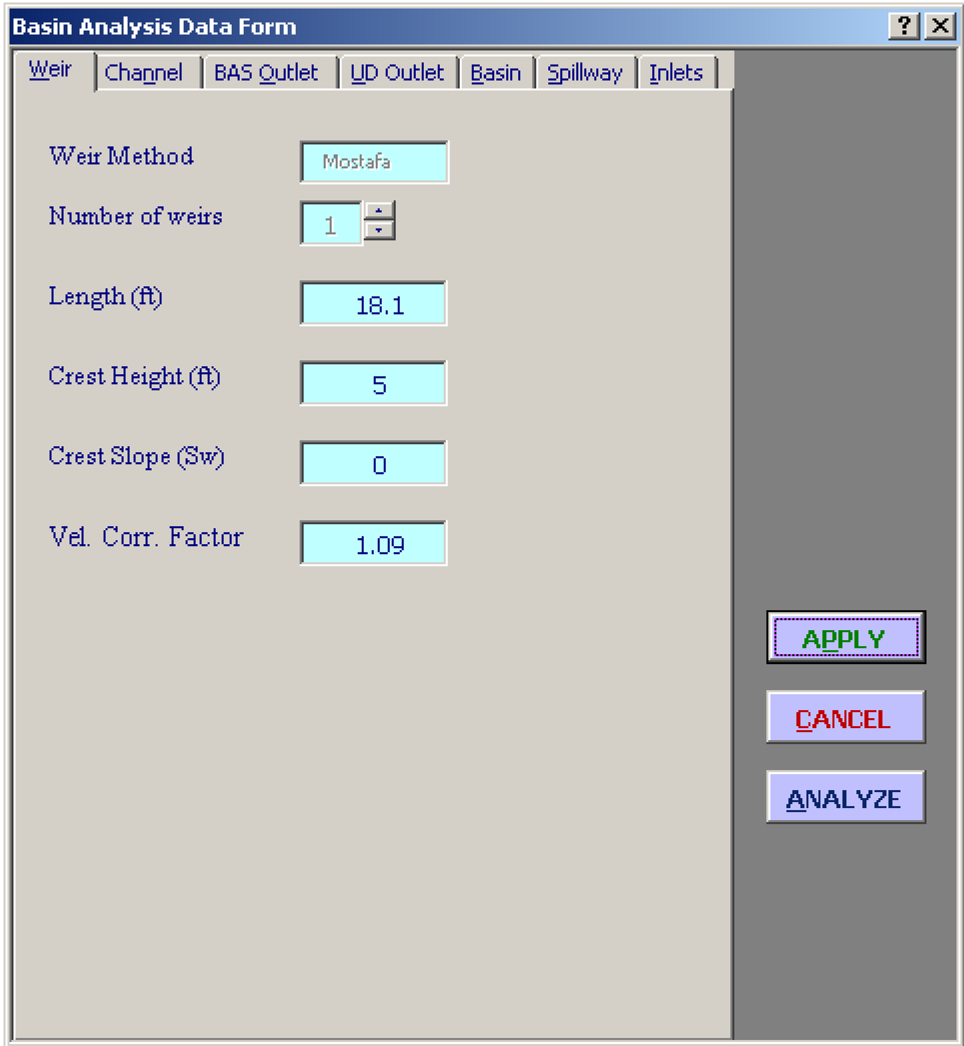
Step 8 – Display the Basin Analysis form

Select the **Simulate/Basin Analysis** menu command to display the **Basin Analysis** input form. This will be utilized to input the remaining data and analyze the system.

Step 9 - Enter the weir's data

BAS requires all pertinent side-weir data to be supplied. This data includes:

Number of Weirs:	One or two side-weirs may be utilized
Length (ft):	The side-weir's crest length along the channel
Crest Height (ft):	The height of the side-weir's crest above the channel invert at the downstream end of the weir
Crest Slope:	The slope of the side-weir's crest
Velocity Corr. Factor:	This is the velocity correction factor for the channel along the weir

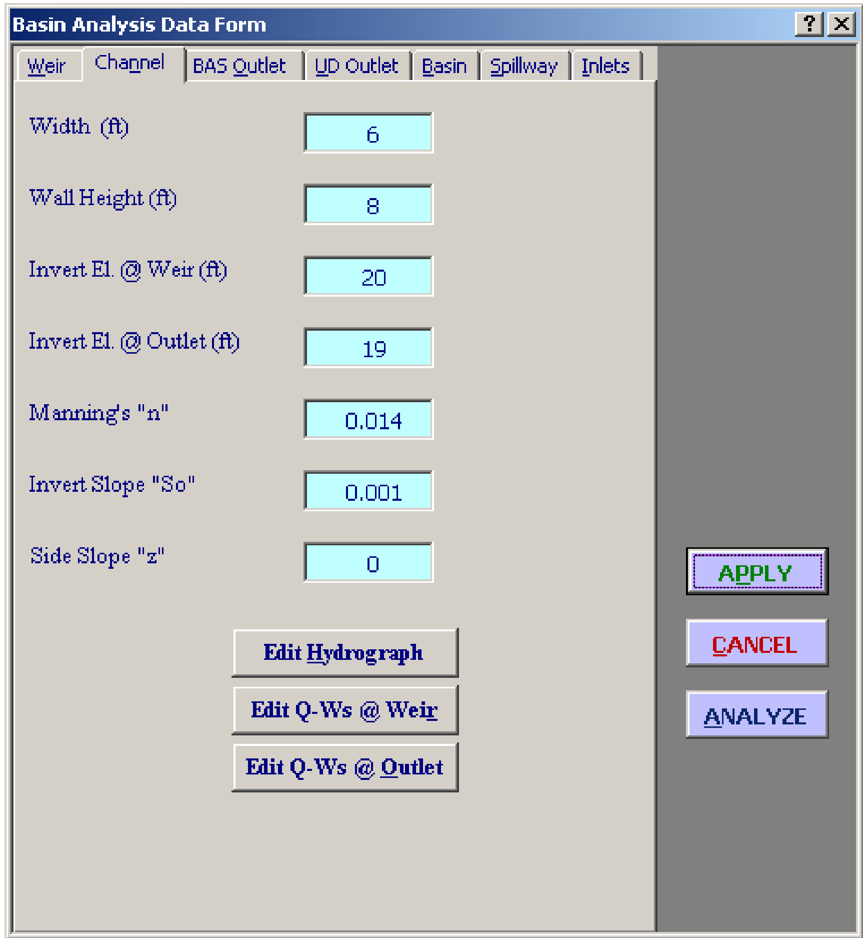


Step 10 - Enter the channel's parameters

BAS requires all pertinent channel data to be supplied. This data includes:

Weir Method :	This displays the current setting for weir coefficient calculations. To change this, go to **Settings/Weir Tolerances/Settings**
Channel Width (ft) :	The width of the channel along the side-weir
Channel Wall Height (ft) :	The wall height above the invert
Invert El. @ Weir (ft) :	The channel invert elevation at the downstream end of the side-weir
Invert El. @ Outlet (ft) :	The channel invert elevation at its confluence with the basin's outlet

Manning's "n":	The channel's Manning's coefficient of friction
Invert Slope (So):	The channel's invert slope along the side-weir
Side Slope "z":	The channel's side-slope ratio (run to rise)
Edit Hydrograph	This button provides access to the inflow hydrograph data entry table. This is the same location as that accessed when pressing **Edit/Main Hydrograph**.
Edit Q-Ws @ Weir	This button provides access to the stage-discharge (at downstream end of the side-weir) data entry table. This is the same location as that accessed when pressing **Edit/Q-WS @ Weir**.
Edit Q-Ws @ Outlet	This button provides access to the stage-discharge (at downstream end of the side-weir) data entry table. This is the same location as that accessed when pressing **Edit/Q-WS @ Outlet**.



Step 11 - Enter the basin outlet's parameters

BAS includes three types of outlet devices. The standard outlet requires all pertinent basin outlet data to be supplied. The User-Defined outflow is possible by selecting the UD Outlet form. In this form, two option are available. The first is a User-Defined stage-discharge table and the second is a Pump. Below is a brief description of the parameters needed for each.

BAS Outlet (Standard)

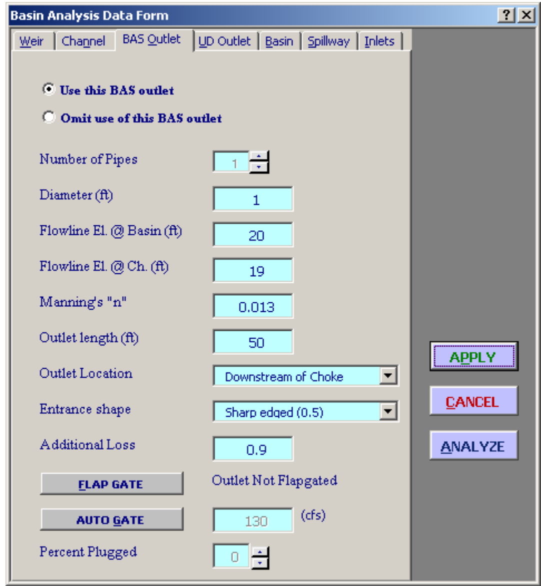
The standard BAS outlet requires the following:

- Number of Pipes: BAS accepts a maximum of 10 outlet pipes within the basin system. Outlet pipes will all have the same geometric and hydraulic parameters.
- Diameter (ft): This is the diameter of the outlet pipe.

- Flowline El. @ Basin (ft): This is the outlet's flowline elevation at the basin end.
- Flowline El. @ Ch. (ft): This is the outlet's flowline elevation at the confluence with the main channel. This elevation must be greater than the channel's invert elevation at this location.
- Manning's "n": This is the Manning's friction coefficient for the outlet pipe
- Outlet Length (ft): This is the length of the outlet pipe
- Outlet Location: The location of the outlet may be upstream or downstream of the main channel's choke. If the outlet is located upstream of the channel's choke, the stage-discharge relationship at the outlet is assumed equal to the stage-discharge relationship at the downstream end of the weir.

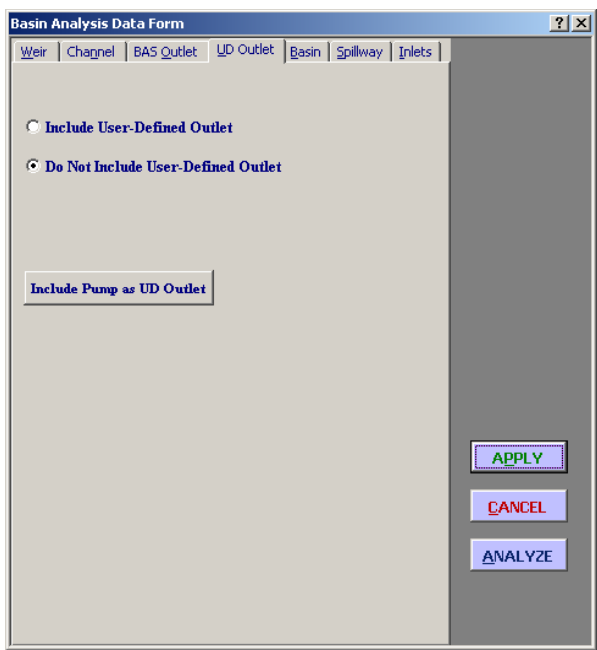
Note: It is recommended to locate the outlet downstream of the choke in order to more efficiently drain the basin and prevent unnecessary backflows into the basin from the channel.

- Entrance Shape: The entrance shape of the outlet may be either sharp or smooth edged.
- Additional Loss: This is the summation of all other minor loss coefficients within the outlet system.
- Flap-gate If the outlet system includes a flap-gate, select this button.
- Auto Gate If the outlet system includes an automatic gate (operated by mechanical or electronic sensors), select this button and enter the maximum desired flow in the main channel downstream of the outlet.
- Percent Plugged It is possible to examine the effect of outlet plugging on the system's performance by clicking (up or down) the percentage of outlet plugging. A 0% plugged outlet passes 100% of the flows into or out of the basin. A 30% pluggage factor allows the outlet to pass 70% of the flows.



UD Outlet

In its default state, no User-Defined outlets are selected. The form below displays this state. The user has a choice between a User-Defined Outlet and Pump.



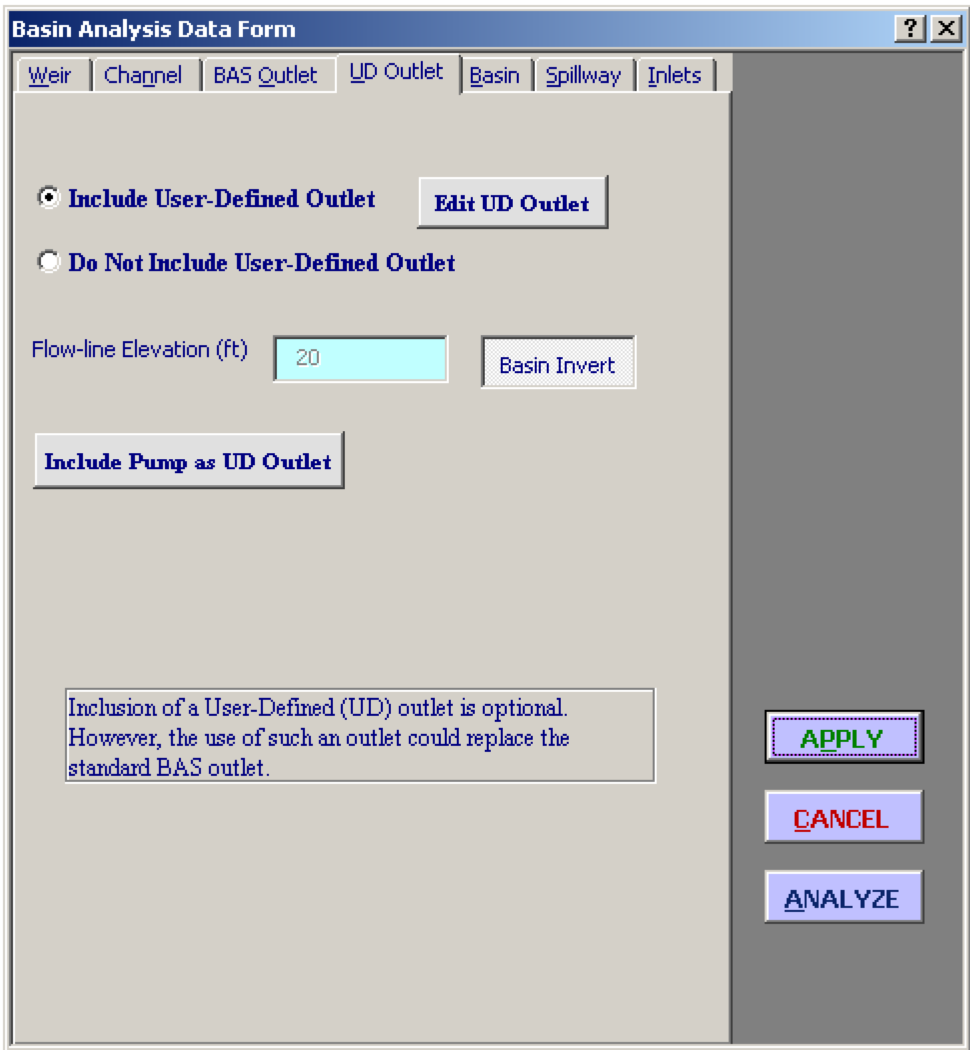
User-Defined Outlet

When this option is selected the form below is displayed. Parameters needed are:

Flow-line Elevation (ft) – This sets the elevation at which the outlet begins to discharge flows out of the basin. It may be higher or equal to the basin invert. If left blank, BAS automatically selects the minimum invert elevation of the basin.

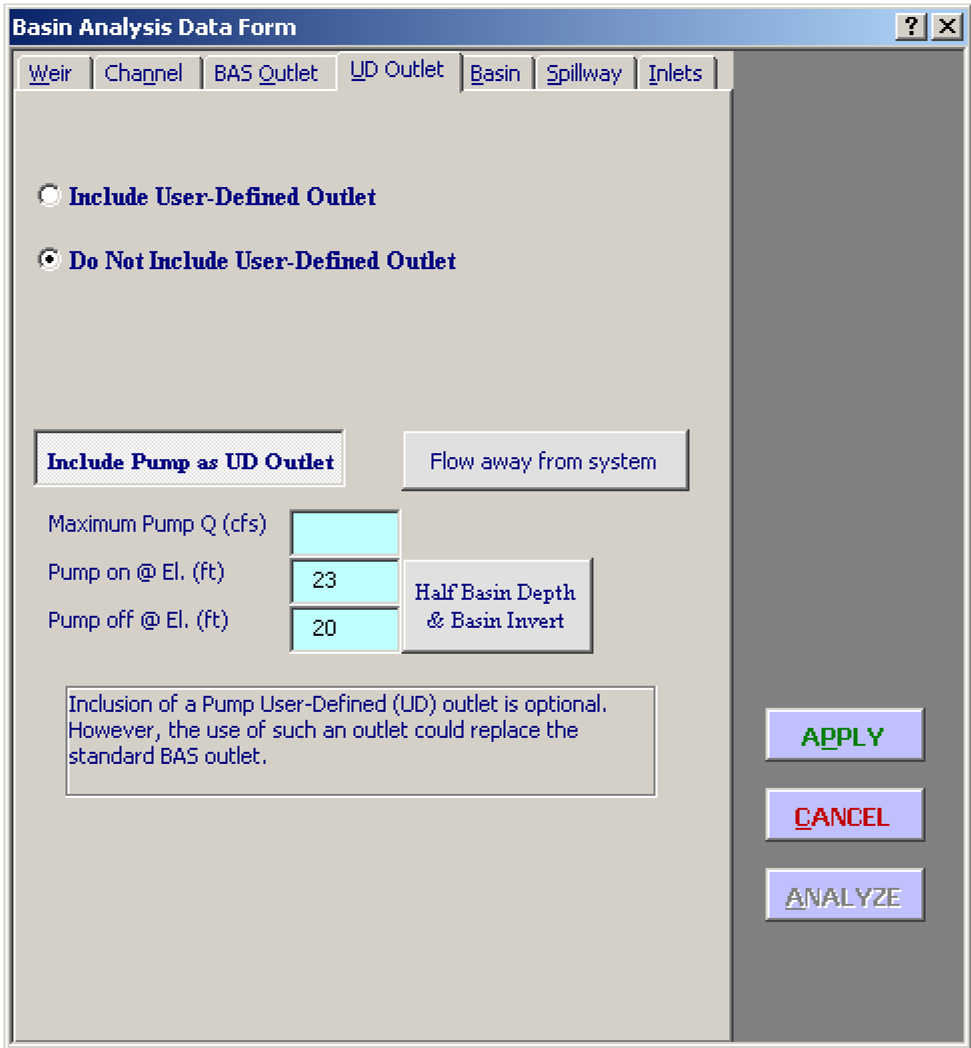
Basin Invert – This button is a convenient way to insert the lowest elevation in the basin.

Edit UD Outlet – This button allows the user to enter the outflow stage-discharge table.



Pump Outlet

When the "Include User-Defined Outlet" option is selected, the form below is displayed. Note that the user may utilize either the User-Defined outflow relationship or a pump. When a pump is selected, the following parameters are needed:



Flow away from system – This button allows the user to direct the outflow completely away from the basin/channel system (flow lost). Alternately, when this button is re-pressed, the button will display Flow to channel. This directs the flow back to the channel at the location of the outlet.

Pump on @ El. (ft) – This sets the water surface elevation within the basin which triggers the pump to be activated.

Pump off @ El. (ft) – This sets the water surface elevation within the basin which triggers the pump to be shut off.

To assist the user with selecting the pump "on" and 'off", a button is available which selects an "on" position (elevation) at halfway depth within the basin (halfway from basin invert to weir crest). In this, the lowest basin elevation is selected as the "off" elevation.

Step 12 – Enter Emergency Spillway parameters

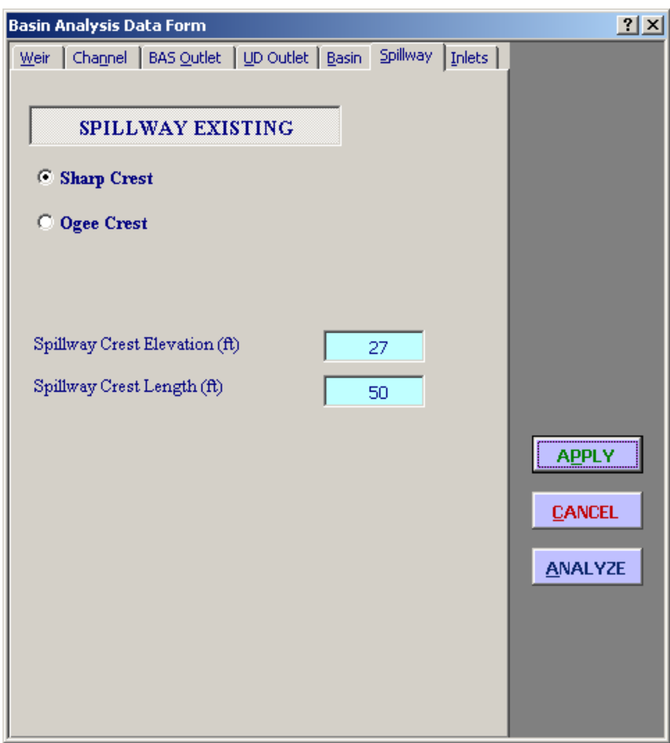
BAS does not require Spillway data to be supplied. However, if a spillway exists, the user may choose a sharp-crested or an ogee crested spillway. The ogee crested spillway requires the basin's invert elevation at the spillway for more accurate determination of the coefficient of discharge. The following parameters must be included:

Select the **Spillway Existing** button

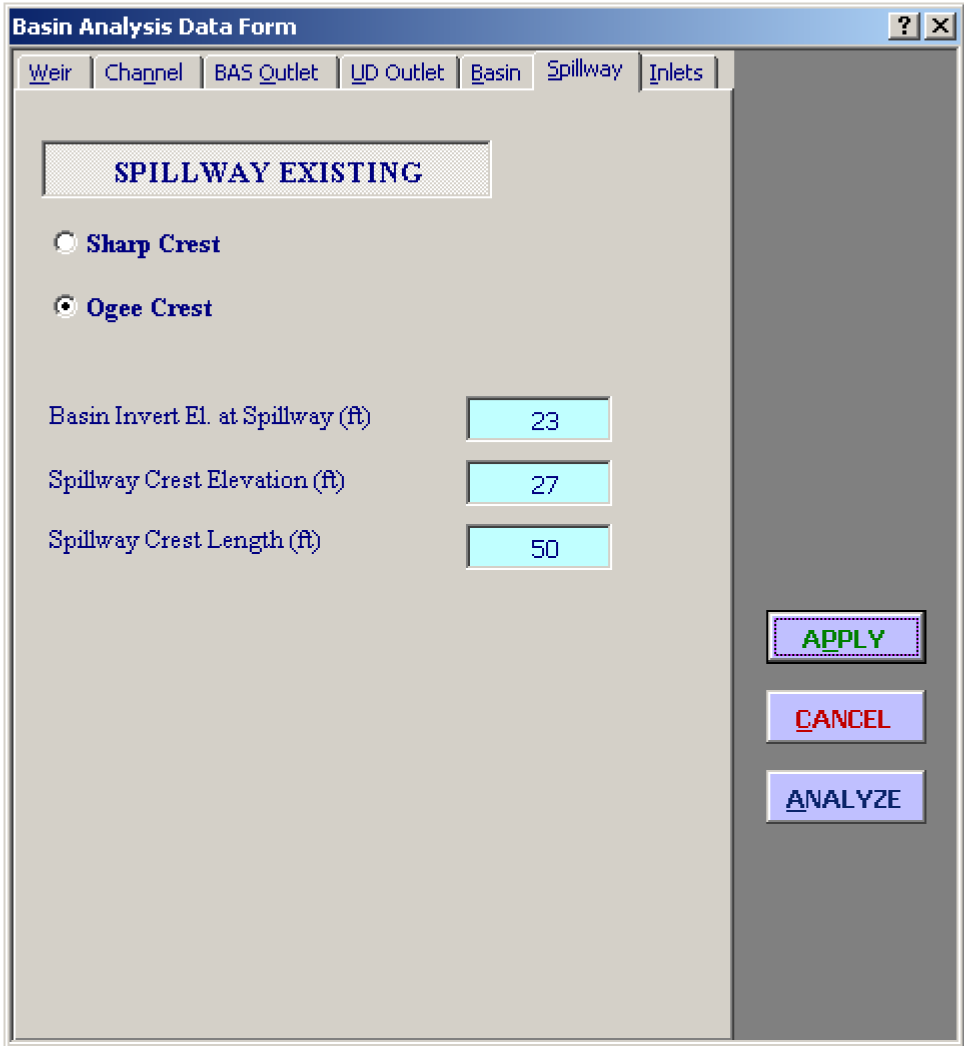
Select **Sharp Crest** or **Ogee Crest**

Enter Spillway Crest Elevation (ft)

Enter Spillway Crest Length (ft)



Note the additional information item needed for the Ogee Crest spillway (Basin Invert Elevation at Spillway).



Step 13 - Enter the basin's (other) Inlets' hydrograph data

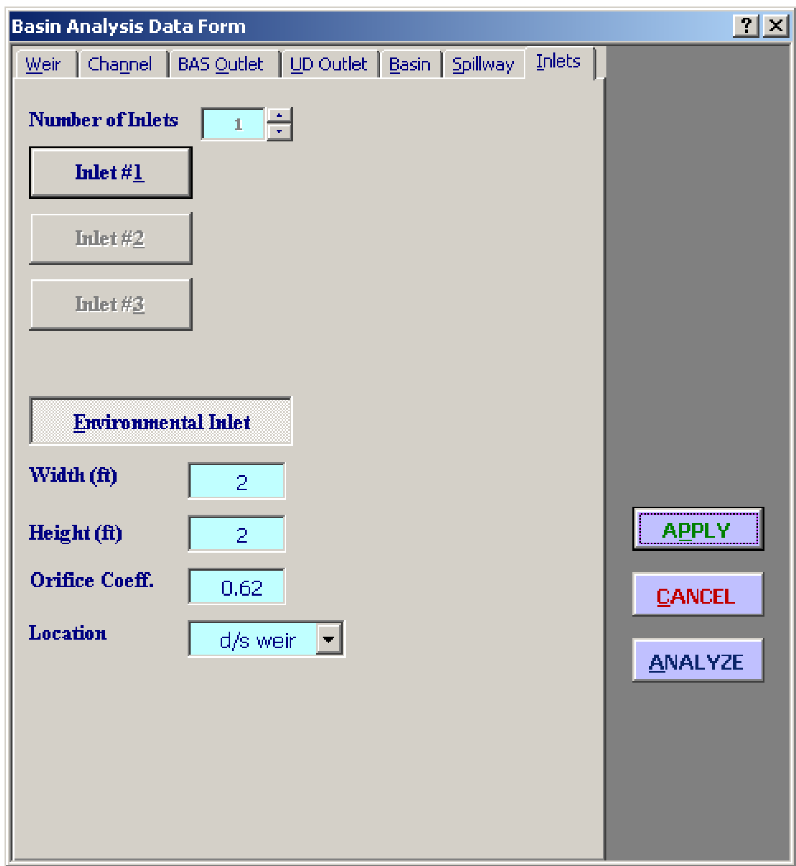
BAS does not require Inlets' data to be supplied. However, if Inlets exists within the basin system, the following parameters must be included:

- Select the number of inlets
- Press each Inlet's button to enter its hydrograph data
- Similar to the main channel's hydrograph input, enter Time (hours) and Discharge (cubic feet per second) for each time increment of the Inlet's hydrograph.

Step 14 - Enter the Environmental Inlet's parameters

BAS does not require Environmental Inlet data to be supplied. However, if such an inlet exists within the basin system, the following parameters must be included:

Width (ft) :	Enter Environmental Inlet's opening width (along the channel)
Height (ft) :	Enter Environmental Inlet's opening height with respect to the channel's invert
Orifice Coefficient:	Enter the opening's (orifice) contraction coefficient
Location:	Select the location of the Environmental Inlet. The inlet may be located at the upstream, downstream or midway at the side-weir



Step 15 – Accept the data

Once all the data has been entered, press the **Apply** button to accept and transfer the data to the project file.

Step 16 – Analyze the retarding system

Press the **Analyze** button to begin the system's analysis. The first step that BAS performs in the analysis is to check the input for missing or erroneous data. If there is any unusable or missing data, BAS displays a report containing such errors. The user then proceeds to repair the listed errors and then summons the **Basin Analysis** component once more.

BAS undergoes several preparatory calculation steps prior to beginning the routing of the hydrograph. In general, the BAS calculations conclude rather quickly (a few minutes). However, should the flow over the side-weir become submerged (i.e. the water surface in the basin submerges the weir crest), the calculations will take significantly longer in order to balance the hydraulics at each time-step. To provide the user information regarding the status of the calculations, BAS displays (at the bottom-left of the screen) a status bar which indicates the current time-step as well as the weir and outlet flows. In addition, the type of weir flow and the basin's volume are also indicated on the status bar.

Note: Depending upon the duration of the hydrograph as well as the size of the basin, the analysis may absorb several minutes. A longer period is required when the basin reaches its capacity and its water surface submerges the weir crest. Similarly, when the flow has reversed its direction over the weir (from basin to channel), and the water surface in the channel is above the weir crest, the calculations require more iterations and thus more calculation time.

A BASIN ANALYSIS EXAMPLE:

In order to demonstrate the use of the **Basin Analysis** component, a detailed example is provided below. It is recommended to review and practice using the **Weir Design** and **Weir Analysis** components. The following sample problem is a continuation of the example presented in the **Weir Design** and **Weir Analysis** components chapters:

A development is proposed along a creek which is anticipated to increase the flows in the creek from 130.3 cfs to 346 cfs. The development conditions placed upon the development, in part, requires no increase in discharge in the downstream existing channel. Therefore, retarding was necessitated within the development. First, an online retarding basin was preliminary designed using state-of-the-art optimization software. The optimized system required an area of 2.66 acres in order to accomplish the desired diversion.

Offline retarding, when possible, is considerably more efficient in accomplishing the same amount of retarding. Therefore, BAS was utilized. It is not practical to include, herein, all of the considerations in the design of the main channel and the choke box. However, the values for these parameters are provided as input into the **Basin Analysis** form. The optimized system, using BAS required an area of 1.63 acres in order to accomplish the desired diversion. This is 39 percent less surface area than the optimized online retarding basin. Where land values are at a premium, this could translate into a substantial cost savings in right-of-way.

Referring to the steps outlined in the "Detailed Guide to a successful BAS Run" section, the following is presented:

- Step 1 – Step 3: these steps were accomplished in the **Weir Design** and **Weir Analysis section.**
- <u>Step 4 – Enter the main channel hydrograph</u>: The same hydrograph data used in the design of the online system was input in the Main Hydrograph form (menu item **Edit/Main Hydrograph**)

File Edit View Simulate Settings

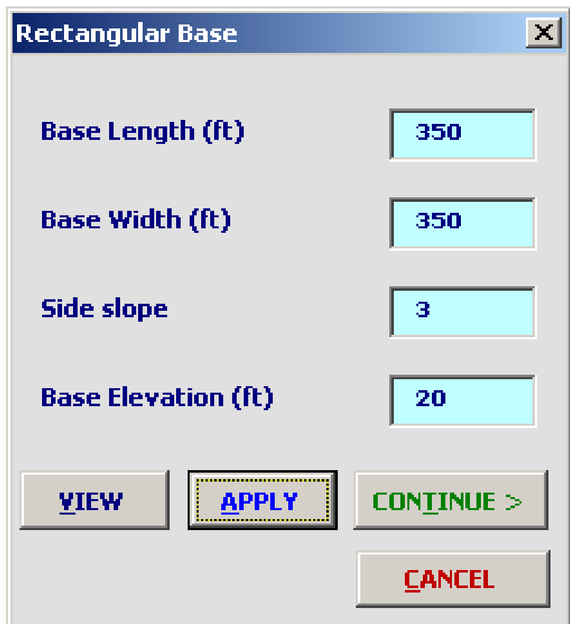
100%

U/S HYDROGRAPH

Time (hrs)	Discharge (cfs)
0.000	0.0
0.100	3.0
0.200	14.0
0.300	29.0
0.400	46.0
0.500	84.0
0.600	101.0
0.700	130.0
0.800	173.0
0.900	229.0
1.000	252.0
1.100	308.0
1.200	346.0
1.300	324.0
1.400	281.0
1.500	179.0
1.600	144.0
1.700	132.0
1.800	108.0
1.900	98.0
2.000	85.0
2.100	69.0
2.200	52.0
2.300	42.0
2.400	30.0
2.500	20.0
2.600	13.0
2.700	4.0
2.800	0.0
6.200	0.0

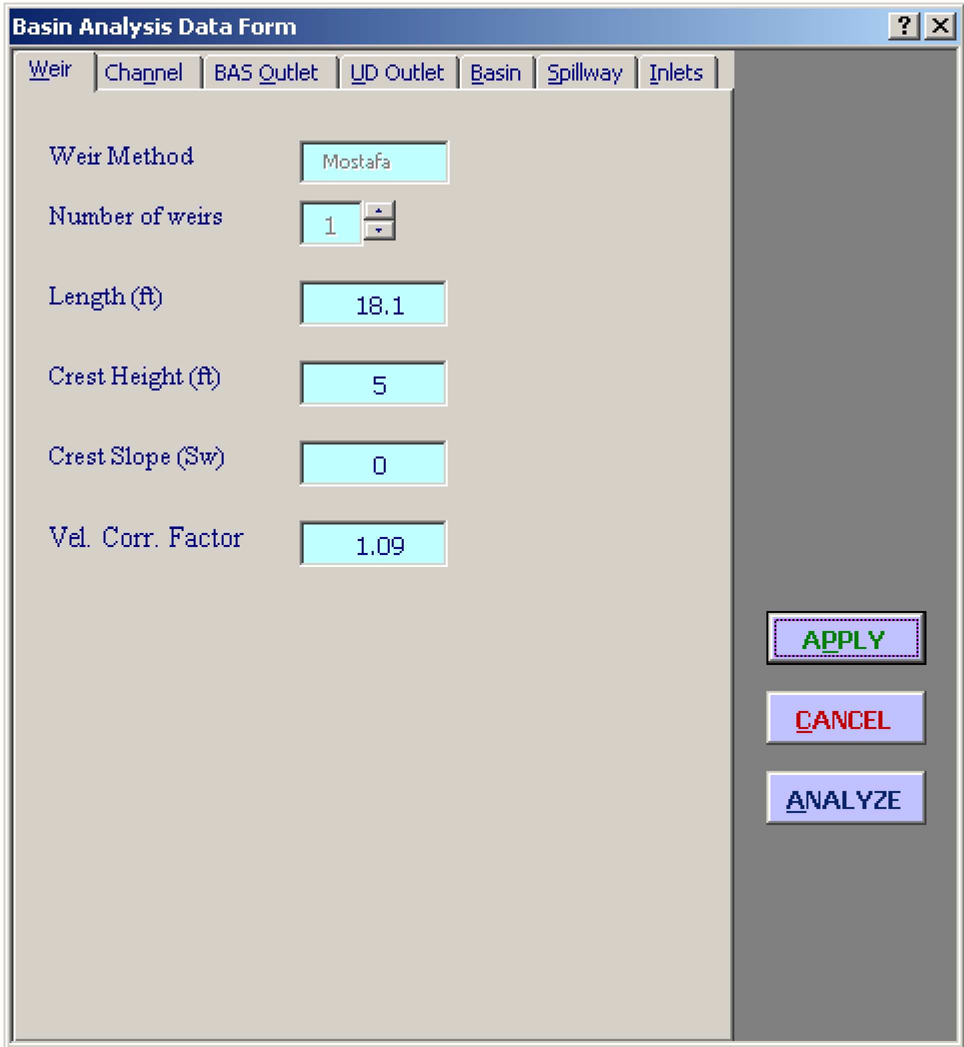
- Step 5 – Enter the main channel Stage-Discharge at downstream of weir: This step provides BAS with the results of gradually-varied-flow calculations at the downstream end of the weir using HECRAS.

- Step 6 - Enter the main channel Stage-Discharge at basin's outlet : This step provides BAS with the results of gradually-varied-flow calculations (HECRAS) at the outlet/channel confluence.
- Step 7 – Enter the Stage-volume relationship at the basin: This step provides BAS with the relationship of the stage to volume within the basin. Since the basin is at a preliminary design level at this point, it is simpler to choose the Rectangular Basin option. To do so, select **Simulate/Basin Analysis** menu. Once the Basin Analysis form is displayed, select **Rectangular** and press **Edit Rectangular Basin Data**.

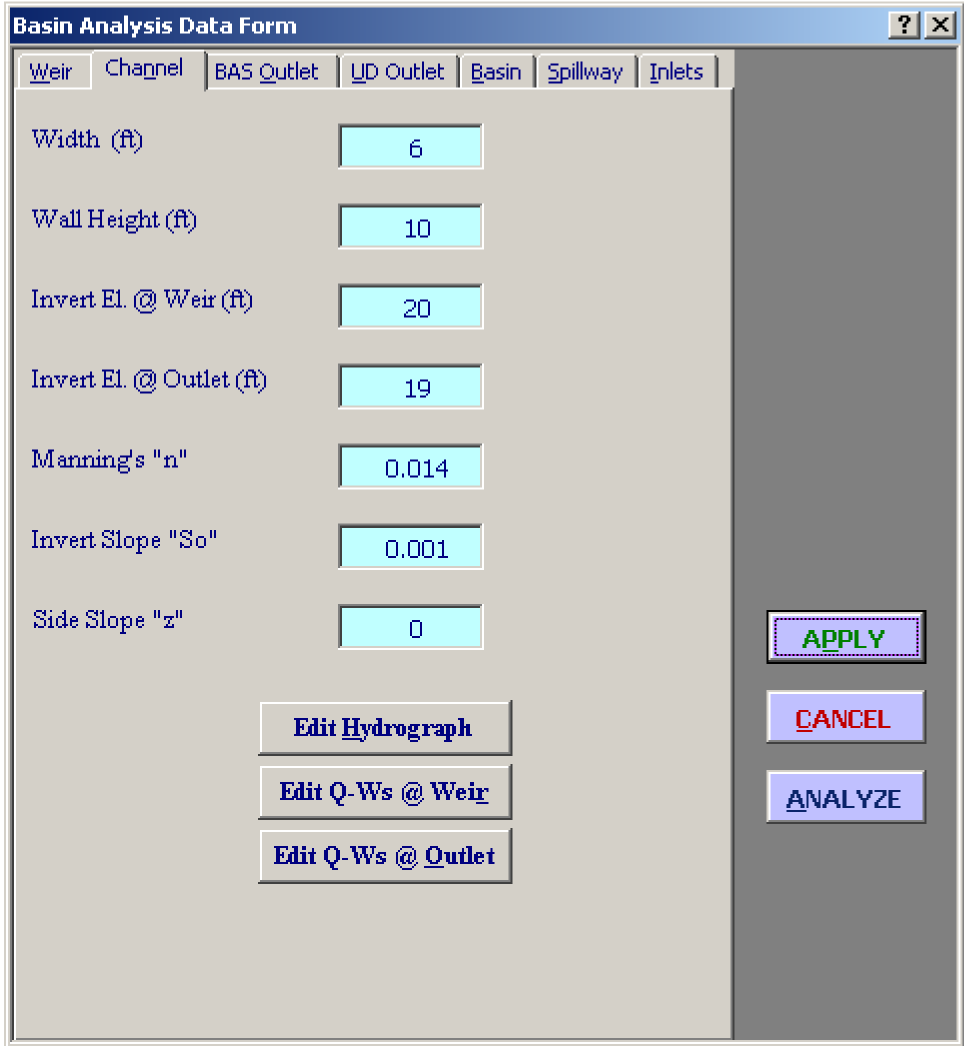


- Step 8 – Display the Basin Analysis form: This step is needed if the previous step was accomplished without displaying the Basin Analysis form (through the **Edit/Basin Volume menu**). Initially, enter dimensions for a relatively large basin. This will ensure that sufficient storage volume is available to complete the analysis. Refinements will be accomplished later to reduce the size. The following data should be entered:

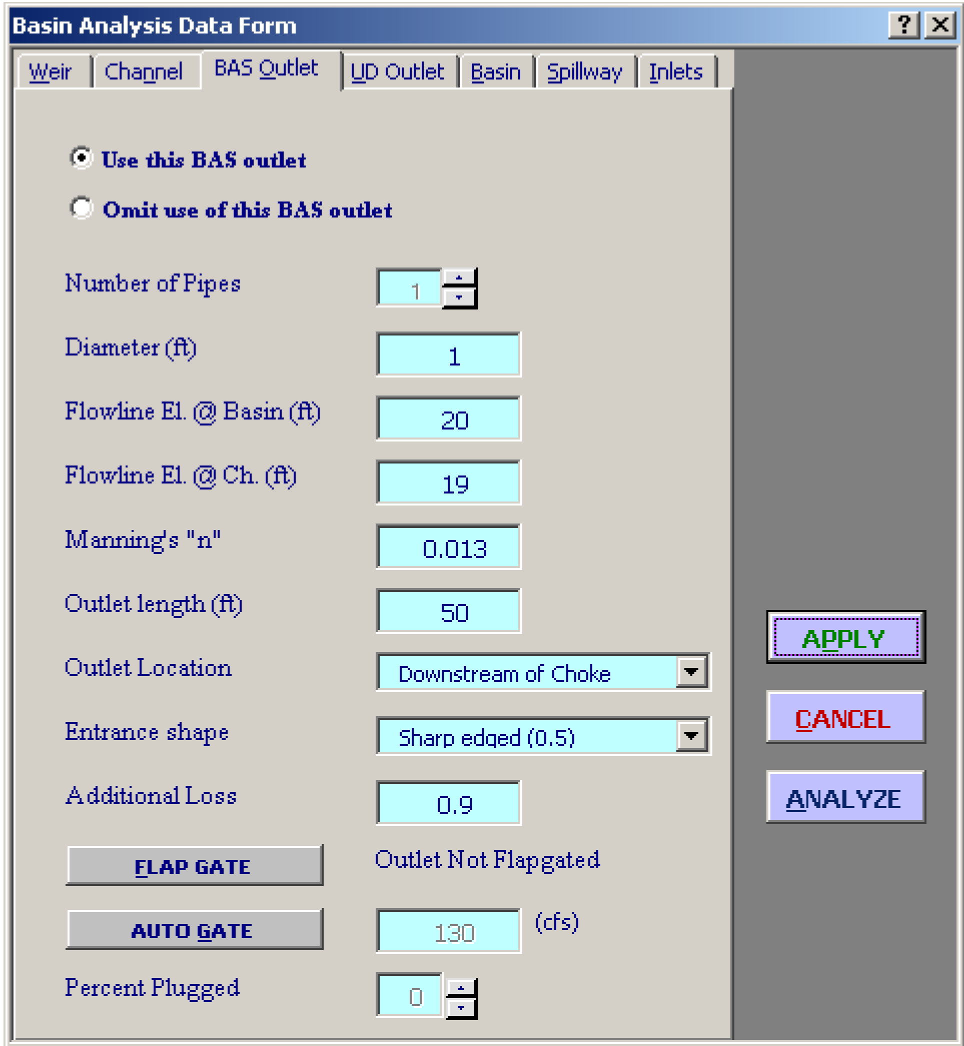
- Step 9 – Enter the weir's data:



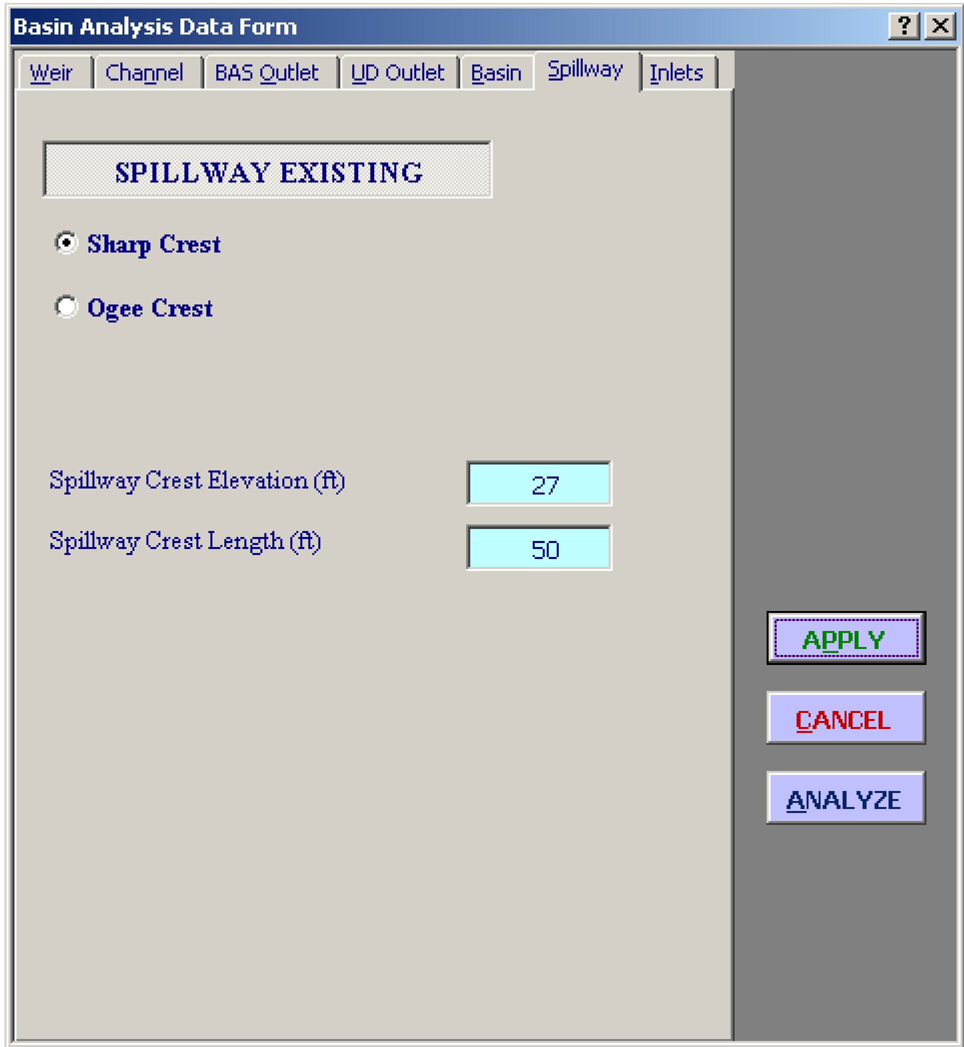
- Step 10 – Enter the channel's parameters:



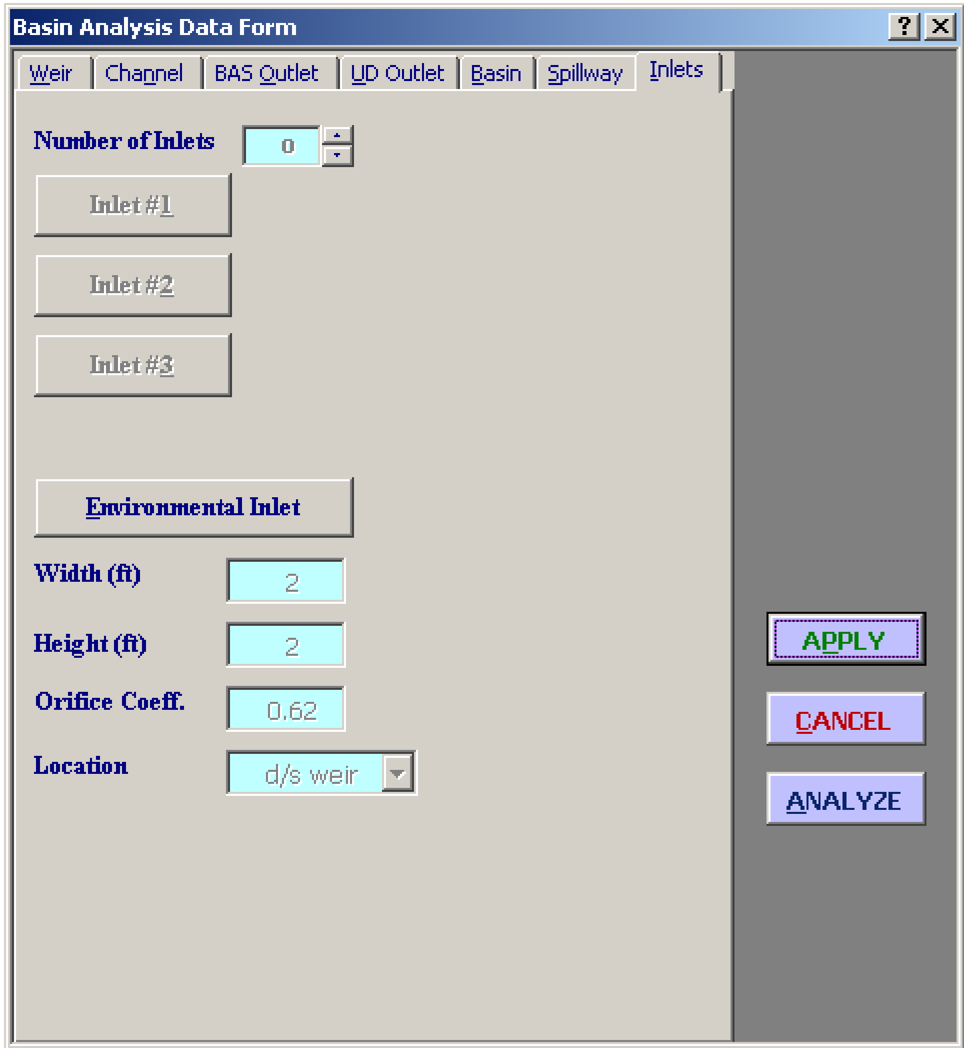
- Step 11 – Enter the basin outlet's parameters:



- <u>Step 12 – Enter the basin spillway's parameters</u>:



- Step 13 and Step 14 – Enter the basin's (other) Inlets' hydrograph data and the Environmental Inlet's parameters :



- Step 15 – Accept the data: Press **APPLY**
- Step 16 – Analyze the retarding system: Press **ANALYZE**

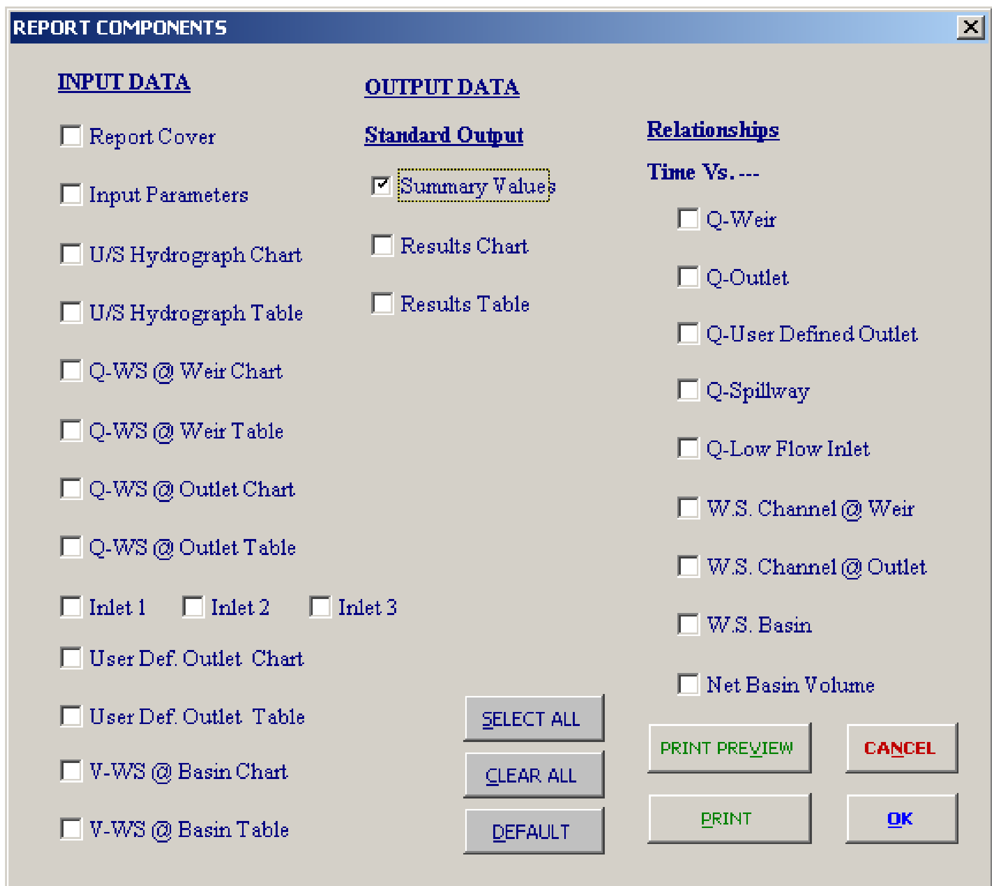
Once the calculations have been completed, the resulting Basin Analysis table is displayed. At this point, perform a cursory review of the results.

Critical values to check are:

- Maximum channel discharge at outlet - this value needs to be less than the 130 cfs maximum allowable value for the downstream channel system.

- Water surface in the basin – an efficiently designed basin system would utilize the available basin volume up to the weir crest elevation.
- System Efficiency – an optimum system would have an efficiency higher than 90 percent.

The most direct way to check these parameters is by reviewing the summary sheet. To do so, press **Settings/Report Components** menu. Once the form is displayed, clear all items on the form and check **Summary Values.**



Finally, press **PRINT PREVIEW**. The following is the displayed form:

SUMMARY OF RESULTS

CHANNEL

Max. Discharge @ d/s weir (cfs) = 125.6	Inflow Hydrograph Volume (Ac-Ft)	= 27.2
Max. Discharge @ u/s weir (cfs) = 346.0	Downstream Hydrograph Volume (Ac-Ft)	= 22.7
Max. Discharge @ Outlet (cfs) = 136.0		

WEIR

Max. Discharge into Basin (cfs) = 220.4	Weir Flow (Into Basin) Volume (Ac-Ft)	= 10.2
Max. Discharge out of Basin (cfs) = 0.0	Weir Flow (Out of Basin) Volume (Ac-Ft)	= 0.0

OUTLET(s)

Max. Discharge Into Basin(cfs) = 8.5	Outlet Flow(Into Basin) Volume (Ac-Ft)	= 0.4
Max. Discharge Out of Basin(cfs) = -24.0	Outlet Flow(Out of Basin) Volume (Ac-Ft)	= -6.2

SPILLWAY

Max. Discharge Out of Basin(cfs) = 0.0	Spillway Flow Volume (Ac-Ft)	= 0.0

BASIN — **SYSTEM EFFICIENCY** = 71.70

Max. Water Surface El. (ft)	= 22.99
Max. Volume (Ac-ft)	= 9.965

INLET 1

Max. Discharge (cfs) = 0.0	Inlet#1 Flow Volume (Ac-Ft)	= 0.0

INLET 2

Max. Discharge (cfs) = 0.0	Inlet#2 Flow Volume (Ac-Ft)	= 0.0

INLET 3

Max. Discharge (cfs) = 0.0	Inlet#3 Flow Volume (Ac-Ft)	= 0.0

LOW-FLOW INLET

Max. Discharge Into Basin(cfs) = 0.0	Low-Flow Inlet(Into Basin) Volume (Ac-Ft)	= 0.0
Max. Discharge Out of Basin(cfs) = 0.0	Low-Flow Inlet(Out of Basin) Volume (Ac-Ft)	= 0.0

USER-DEFINED OUTFLOW

Max. Discharge Out of Basin(cfs) = 0.0	User Defined Outflow Volume (Ac-Ft)	= 0.0

As can be seen from the above, the system is not very efficient. This is mostly due to the excessive volume allotted. Therefore, the next run (run 2) includes a smaller outlet at 1.5 feet diameter. The table below shows the discharge for the modified system has been reduced and the efficiency increased. The following run (run 3) utilized an outlet diameter of 1.0 foot. Now, the discharge in the downstream channel has been further reduced, and the efficiency increased. Although the system has been improved, further optimization could be accomplished. The 4th through the 6th runs decreased the basin's area to 240 ft by 300 ft. The use of flapgates (run 7) did not reduce the basin's area. The use of the Autogate utility (run 9), reduced the basin area to 230 ft by 300 ft. Finally, using the Autogate utility while submerging the weir (run 14) resulted in an optimum basin area of 180 ft by 300 ft with an efficiency of 97.00%.

Runs 16 through 27 utilized a higher and longer weir, 6 ft and 41 ft respectively. As can be seen from the table below, the optimum basin (with submergence) has a 160 ft by 300 ft base area without the use of flapgates or Autogates. Their use, as can be seen, does not contribute to the optimization of this system.

In summary, the higher (and longer weir) is much more efficient than the lower and shorter weir. Naturally, the basin and outlet configuration play a major role in the efficiency of the system as a whole.

Run	Basin Area	Outlet Diameter	Weir length/ht	Basin Elev.	Qch at Outlet	Flapgate	Auto Gate	Env. Inlet	Efficiency
1	350x350	2.0	18.1/5	22.99	**136.0**	No	No	No	72.70
2	350x350	1.5	18.1/5	23.01	129.0	No	No	No	75.40
3	350x350	1.0	18.1/5	23.03	126.2	No	No	No	75.50
4	300x300	1.0	18.1/5	23.98	127.6	No	No	No	84.75
5	250x300	1.0	18.1/5	24.66	128.3	No	No	No	91.50
6	240x300	1.0	18.1/5	24.83	128.4	No	No	No	93.15
7	240x300	1.0	18.1/5	24.80	128.4	Yes	No	No	93.15
8	240x300	2.0	18.1/5	24.56	130.0	Yes	Yes	No	92.40
9	230x300	2.0	18.1/5	24.73	130.0	Yes	Yes	No	94.15
10	220x300	2.0	18.1/5	24.92	130.0	Yes	Yes	No	96.05
11	210x300	2.0	18.1/5	25.12 **S**	130.0	Yes	Yes	No	96.85
12	200x300	2.0	18.1/5	25.33 **S**	130.0	Yes	Yes	No	96.90
13	190x300	2.0	18.1/5	25.57 **S**	130.0	Yes	Yes	No	96.95
14	180x300	2.0	18.1/5	25.83 **S**	130.0	Yes	Yes	No	**97.00**
15	175x300	2.0	18.1/5	25.96 **S**	130.1	Yes	Yes	No	96.75
16	230x300	1.0	41/6	24.87	128.4	No	No	No	88.75

17	200x300	1.0	41/6	25.48	128.9	No	No	No	91.80
18	190x300	1.0	41/6	25.72	129.1	No	No	No	93.75
19	185x300	1.0	41/6	25.85	129.1	No	No	No	94.85
20	170x300	1.0	41/6	26.24 **S**	129.4	No	No	No	96.30
21	165x300	1.0	41/6	26.39 **S**	129.5	No	No	No	96.30
22	160x300	1.0	41/6	26.54 **S**	129.6	No	No	No	**96.35**
23	155x300	1.0	41/6	26.70 **S**	**132.1**	No	No	No	95.4
24	155x300	1.0	41/6	26.66 **S**	**132.1**	Yes	No	No	95.65
25	155x300	1.0	41/6	26.66 **S**	**130.5**	Yes	Yes	No	96.35
26	155x300	3.0	41/6	26.49 **S**	**130.3**	Yes	Yes	No	97.65

UNDERSTANDING A BASIN ANALYSIS REPORT

The Basin Analysis Report is a very useful feature of BAS. Summary and detailed results may be generated which allows the user to develop a true sense of understanding for the system being analyzed.

Displaying and printing the Basin Analysis Report

In order to enable viewing and/or printing a basin analysis report, the basin analysis report's components must be selected first. To do so, select the **Settings/Report Components** sub-menu. Select the desired report components from the following form and press the **OK** button.

Overview of the Basin Analysis Report Components

Report Cover

The report's cover sheet utilizes information that was provided to BAS upon creation of the data file. Selecting the **Settings/Project Information** command will modify this information.

FLOW-BY RETARDING BASIN ANALYSIS

Example 1

Prepared for

Training

Prepared by

Hydraulic Solutions, Inc.

1/1/2001

Input Parameters

This table includes all the single-parameter input values of the project file. Hydrographs and rating tables are presented in the following report sections. To include this information in the BAS report, go to the **Settings/Report Components** and select the Input Parameters check box.

INPUT PARAMETERS

CHANNEL

Width (ft)	=	6.00
Wall Height (ft)	=	8.00
Invert El. @ d/s Wr. (ft)	=	20.00
Manning's Friction Factor	=	0.014
Invert Slope	=	0.00100
Invert El. @ Outlet (ft)	=	19.00
Side Slope (Z)	=	0

BASIN

Starting Water Surface El. (ft)	=	20.00
Geometry	=	Rectangular
Length (ft)	=	350
Width (ft)	=	350
Side-slopes	=	3
Base elev. (ft)	=	20

SPILLWAY

Spillway Crest Elevation (ft)	=	27.00
Spillway Length (ft)	=	50.00

WEIR

Length (ft)	=	18.10
Crest Height (ft)	=	5.00
Crest Slope	=	0.00000
Vel. Corr. Factor	=	1.09
Number of Weirs	=	1
Weir Method	=	Mostafa

INLET(s)

Number of Inlets	=	0

USER-DEFINED OUTFLOW

Actuated	=	FALSE
Flow Line El.	=	20

OUTLET OUTLET ACTIVATED

Diameter (ft)	=	2.00
Length (ft)	=	50.00
Invert El. @ Basin (ft)	=	20.00
Invert El. @ Channel (ft)	=	19.00
Manning's Friction (n)	=	0.013
Entrance Loss Coeff.	=	0.50
Additional Losses Coeff.	=	0.9
Outlet location	=	DS Choke
Number of conduits	=	1
Flapgates	=	NFG
Percent Plugged	=	0

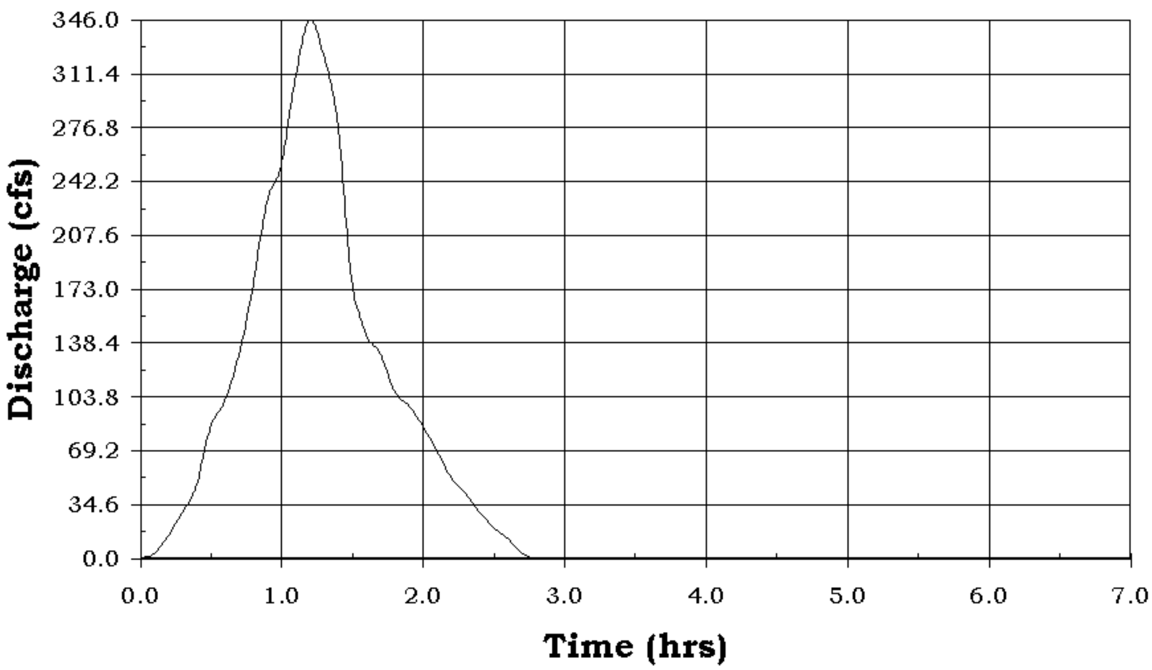
U/S Hydrograph Table and Chart

This table represents the upstream (inflow) hydrograph data of the system. The BAS report may include both the Time-Discharge table as well as the hydrograph chart. To include this information in the BAS report, go to the **Settings/Report Components** and select the **U/S Hydrograph Table** and **Chart** check boxes.

Example 1

U/S HYDROGRAPH	
Time (hrs)	Discharge (cfs)
0.000	0.0
0.100	3.0
0.200	14.0
0.300	29.0
0.400	46.0
0.500	84.0
0.600	101.0
0.700	130.0
0.800	173.0
0.900	229.0
1.000	252.0

Upstream Hydrograph

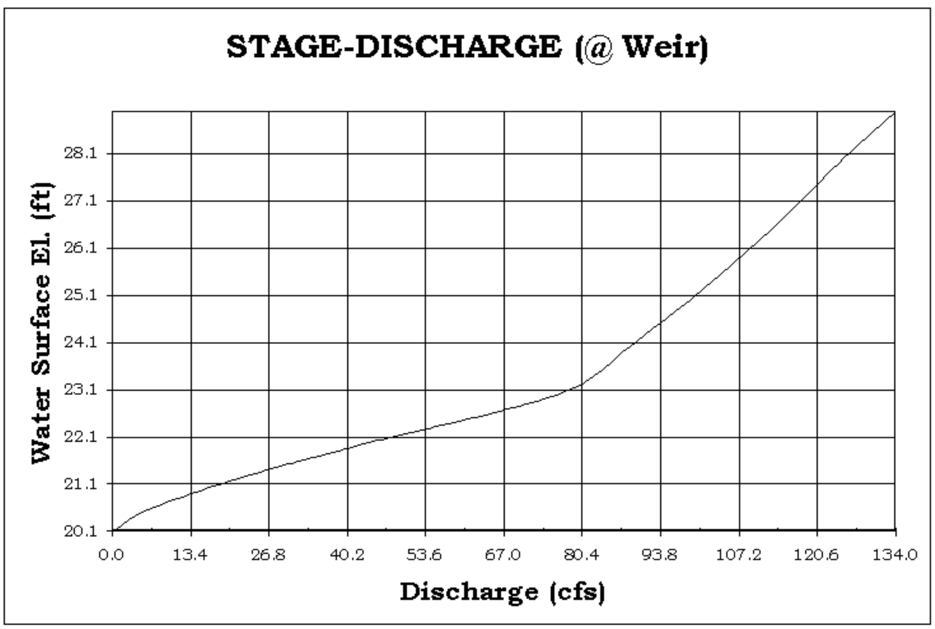


Q-WS @ Weir Table and Chart

This table represents the main channel's hydraulic rating relationship relationship (discharge vs. water surface) at the downstream end of the weir. The BAS report may include both the table as well as the chart. To include this information in the BAS report, go to the **Settings/Report Components** and select the **Q-WS @ Weir Table** and **Chart** check boxes.

Example 1

Q-WS @ ds Weir	
Discharge (cfs)	W.S. El (ft)
0.4	20.10
5.7	20.50
16.6	21.00
30.2	21.50
36.5	21.70
45.5	22.00
76.8	23.00
88.7	24.00
99.1	25.00
108.6	26.00
117.3	27.00
125.4	28.00
134.5	29.00

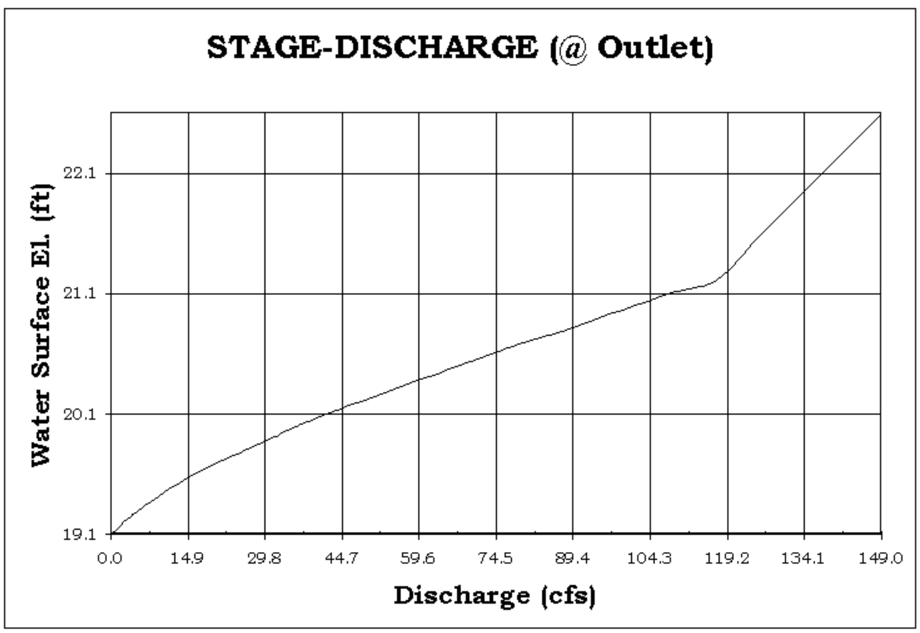


Q-WS @ Outlet Table and Chart

This table represents the main channel's hydraulic rating relationship (discharge vs. water surface) at the downstream end of the weir. The BAS report may include both the table as well as the chart. To include this information in the BAS report, go to the **Settings/Report Components** and select the **Q-WS @ Outlet Table** and **Chart** check boxes.

Example 1

Q-WS @ d/s Outlet	
Discharge (cfs)	W.S. El (ft)
0.4	19.10
5.7	19.30
16.6	19.60
30.2	19.87
36.5	20.00
45.5	20.15
76.8	20.64
88.7	20.80
99.1	20.96
108.6	21.10
117.3	21.21
125.4	21.56
133.5	21.91
141.6	22.26
149.7	22.61



V-WS @ Basin Table and Chart

The user may choose a table which represents the basin's volume relationship to its water surface elevations. This table is similar to other tables such as the hydrograph table. However, the user may also choose a rectangular shaped basin. If this is used, the dimensions of the rectangular basin are presented in the Input Parameters report component.

Inlet #1, #2 and #3 Tables and Charts

These tables represent any inflow hydrographs into the basin from other sources besides the weir. The BAS report may include both the Time-Discharge tables as well as the hydrograph charts. The data is presented identically to that for the Main Hydrograph table and chart. To include this information in the BAS report, go to the **Settings/Report Components** and select the **Inlet (#) Table** and **Chart** check boxes.

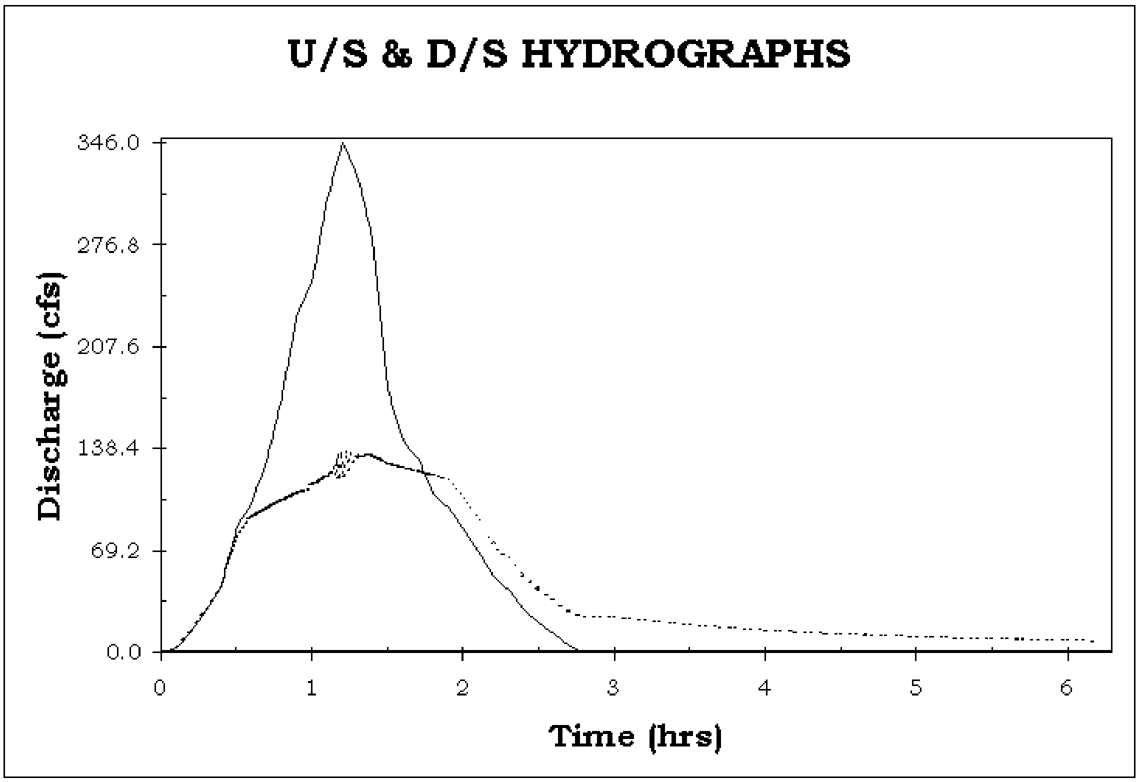
Summary Values

This table represents an overview of the BAS results. Maximum and other critical values are displayed. To include this information in the BAS report, go to the Settings/Report Components and select the Summary Values Table check box. Note: the SYSTEM EFFICIENCY presented midway in the table below represents a general guide to the hydraulic efficiency of the system. This value is arrived at by dividing the hydrograph volume at the peak flow (horizontal line) in the downstream channel by the total retarded volume. Ideally, if one could cut the hydrograph horizontally at the desired downstream peak flow, a 100% efficient hydraulic system would result.

CHANNEL					
Max. Discharge @ d/s weir (cfs)	=	125.6	InflowHydrograph Volume (Ac-Ft)	=	27.2
Max. Discharge @ u/s weir (cfs)	=	346.0	Downstream Hydrograph Volume (Ac-Ft)	=	22.9
Max. Discharge @ Outlet (cfs)	=	136.1			
WEIR					
Max. Discharge into Basin (cfs)	=	220.4	Weir Flow(Into Basin) Volume (Ac-Ft)	=	10.2
Max. Discharge out of Basin (cfs)	=	0.0	Weir Flow(Out of Basin) Volume (Ac-Ft)	=	0.0
OUTLET(s)					
Max. Discharge Into Basin(cfs)	=	8.4	Outlet Flow(Into Basin) Volume (Ac-Ft)	=	0.4
Max. Discharge Out of Basin(cfs)	=	-24.3	Outlet Flow(Out of Basin) Volume (Ac-Ft)	=	-6.4
SPILLWAY					
Max. Discharge Out of Basin(cfs)	=	0.0	Spillway Flow Volume (Ac-Ft)	=	0.0
BASIN			**SYSTEM EFFICIENCY**	=	72.75 Percent
Max. Water Surface El. (ft)	=	23.08			
Max. Volume (Ac-ft)	=	9.932			
INLET 1					
Max. Discharge (cfs)	=	0.0	Inlet#1 Flow Volume (Ac-Ft)	=	0.0
INLET 2					
Max. Discharge (cfs)	=	0.0	Inlet#2 Flow Volume (Ac-Ft)	=	0.0
INLET 3					
Max. Discharge (cfs)	=	0.0	Inlet#3 Flow Volume (Ac-Ft)	=	0.0
LOW-FLOW INLET					
Max. Discharge Into Basin(cfs)	=	0.0	Low-Flow Inlet(Into Basin) Volume (Ac-Ft)	=	0.0
Max. Discharge Out of Basin(cfs)	=	0.0	Low-Flow Inlet(Out of Basin) Volume (Ac-Ft)	=	0.0
USER-DEFINED OUTFLOW					
Max. Discharge Out of Basin(cfs)	=	0.0	User Defined Outflow Volume (Ac-Ft)	=	0.0

Results (Hydrograph) Chart

This chart exhibits both the upstream hydrograph (inflow) as well as the resulting hydrograph at the outlet in the main channel. To include this information in the BAS report, go to the **Settings/Report Components** and select the **Results Chart** check box.



Results Table

This table includes all the details of the system's calculations. To include this information in the BAS report, go to the **Settings/Report Components** and select the **Results Table** check box.

BASIN ROUTING RESULTS

Time (hrs)	Q ch. @ us Weir (cfs)	Q Weir (cfs)	Discharge Coeff.	Q ch. @ ds Weir (cfs)	Q Outlet (cfs)	Outlet control/type	Q ch. @ Outlet (cfs)	WS @ ds Weir (ft)	WS @ us Weir (ft)	WS @ Outlet (ft)	WS @ Basin (ft)	Vol. Change (Ac-ft)	Net Vol. (AF)	Weir Flow Type
1.024	265.2	145.8	0.550	119.4	4.6	Open/Out	114.8	27.26	26.58	21.18	20.90	0.075	2.909	Unsubmerged
1.029	268.5	148.8	0.551	119.7	4.5	Open/Out	115.1	27.29	26.59	21.18	20.92	0.063	2.972	Unsubmerged
1.035	271.8	151.9	0.552	119.9	4.5	Open/Out	115.4	27.32	26.60	21.19	20.95	0.078	3.050	Unsubmerged
1.041	275.1	154.9	0.553	120.2	4.4	Open/Out	115.8	27.35	26.62	21.19	20.97	0.079	3.129	Unsubmerged
1.047	278.4	158.0	0.554	120.4	4.3	Open/Out	116.1	27.38	26.63	21.19	21.00	0.080	3.209	Unsubmerged
1.053	281.6	160.9	0.554	120.7	4.2	Open/Out	116.4	27.41	26.64	21.20	21.02	0.082	3.291	Unsubmerged
1.059	284.9	164.0	0.555	120.9	4.1	Open/Out	116.8	27.45	26.66	21.20	21.05	0.083	3.375	Unsubmerged
1.065	288.2	167.0	0.556	121.2	3.9	Open/Out	117.2	27.48	26.67	21.21	21.07	0.085	3.459	Unsubmerged
1.071	291.5	170.1	0.557	121.4	3.8	Open/Out	117.6	27.51	26.68	21.22	21.10	0.086	3.546	Unsubmerged
1.076	294.8	173.1	0.558	121.7	3.7	Open/Out	117.9	27.54	26.70	21.23	21.12	0.073	3.619	Unsubmerged
1.082	298.1	176.2	0.558	121.9	3.7	Open/Out	118.2	27.57	26.71	21.25	21.15	0.089	3.708	Unsubmerged
1.088	301.4	179.2	0.559	122.2	3.5	Open/Out	118.6	27.60	26.72	21.26	21.18	0.091	3.799	Unsubmerged
1.094	304.7	182.3	0.560	122.4	3.4	Open/Out	119.0	27.63	26.74	21.28	21.21	0.092	3.891	Unsubmerged
1.100	308.0	185.3	0.561	122.7	3.3	Open/Out	119.4	27.66	26.75	21.29	21.24	0.094	3.984	Unsubmerged
1.109	311.5	188.6	0.562	122.9	3.1	Open/Out	119.8	27.70	26.77	21.31	21.28	0.143	4.127	Unsubmerged
1.118	314.9	191.7	0.562	123.2	1.9	Open/Out	121.3	27.73	26.78	21.33	21.32	0.144	4.271	Unsubmerged
1.127	318.4	194.9	0.563	123.5	0.4	Open/Out	123.1	27.76	26.79	21.39	21.37	0.145	4.416	Unsubmerged
1.136	321.8	198.1	0.564	123.7	2.1	Open/Out	121.7	27.79	26.81	21.47	21.42	0.149	4.565	Unsubmerged
1.145	325.3	201.3	0.565	124.0	-1.1	S/Prs/Out	125.1	27.83	26.82	21.41	21.46	0.149	4.714	Unsubmerged
1.155	328.7	204.5	0.566	124.2	3.9	Open/Out	120.4	27.86	26.83	21.56	21.52	0.172	4.886	Unsubmerged
1.164	332.2	207.7	0.567	124.5	-5.8	S/Prs/Out	130.3	27.89	26.85	21.35	21.56	0.150	5.036	Unsubmerged
1.173	335.6	210.8	0.567	124.8	6.2	Open/Out	118.5	27.92	26.86	21.78	21.61	0.161	5.198	Unsubmerged
1.182	339.1	214.1	0.568	125.0	-8.4	S/Prs/Out	133.5	27.96	26.88	21.27	21.66	0.153	5.351	Unsubmerged
1.191	342.5	217.2	0.569	125.3	7.0	Open/Out	118.3	27.99	26.89	21.92	21.71	0.167	5.517	Unsubmerged
1.200	346.0	220.4	0.570	125.6	-9.7	S/Prs/Out	135.2	28.02	26.90	21.27	21.76	0.157	5.674	Unsubmerged
1.214	342.9	217.6	0.569	125.3	6.5	Open/Out	118.8	27.99	26.89	21.97	21.84	0.259	5.933	Unsubmerged
1.229	339.7	214.6	0.568	125.1	-11.0	S/Prs/Out	136.1	27.96	26.88	21.26	21.92	0.252	6.186	Unsubmerged
1.243	336.6	211.8	0.568	124.8	1.4	Prs/Out	123.4	27.93	26.87	22.01	21.99	0.247	6.432	Unsubmerged
1.257	333.4	208.8	0.567	124.6	-10.6	S/Prs/Out	135.2	27.90	26.85	21.46	22.07	0.229	6.662	Unsubmerged
1.271	330.3	205.9	0.566	124.4	-4.4	S/Prs/Out	128.8	27.87	26.84	21.97	22.14	0.233	6.895	Unsubmerged
1.286	327.1	203.0	0.565	124.1	-9.6	S/Prs/Out	133.8	27.84	26.83	21.70	22.21	0.240	7.135	Unsubmerged
1.300	324.0	200.1	0.565	123.9	-8.0	S/Prs/Out	131.9	27.81	26.82	21.91	22.28	0.222	7.357	Unsubmerged
1.308	320.7	197.1	0.564	123.6	-9.8	S/Prs/Out	133.4	27.78	26.80	21.83	22.32	0.124	7.481	Unsubmerged
1.315	317.4	194.0	0.563	123.4	-9.5	S/Prs/Out	132.8	27.75	26.79	21.89	22.35	0.107	7.588	Unsubmerged
1.323	314.1	191.0	0.562	123.1	-10.1	S/Prs/Out	133.2	27.72	26.78	21.87	22.39	0.120	7.707	Unsubmerged

The following provides a brief description of the Results Table columns (Note: the example does not include all the options available in BAS and; therefore, the table above may not display such columns):

Time (hrs): Time values shown in this column as bold fonts represent the original (input) hydrograph time values. The non-bolded values are BAS interpolated values.

Qch @ us Weir (cfs): Discharge values shown in this column represent the discharge values at the very upstream end of the weir (inflow hydrograph).

Q Weir (cfs): Positive discharge values in this column represent flows into the basin while negative values represent flows out (reverse-flow) of the basin.

Discharge Coeff: Discharge coefficient values shown in this column are the coefficients used in the analysis.

Qch @ ds Weir (cfs): Discharge values shown in this column represent the discharge values at the downstream end of the weir.

Q Spillway (cfs): Discharge values shown in this column represent the discharge values over the basin's spillway.

Q Outlet (cfs): Discharge values shown in this column represent the discharge values through the basin's outlet. Positive values in this column represent flows into the basin while negative values represent flows out of the basin.

Outlet control/type: This column provides information regarding the flow-hydraulics through the basin's outlet. The following are the various types of flows through the outlet:

- No Flow: No flow occurs through the outlet.
- M/Open/Out: The slope of the outlet is mild. The flow through the outlet is open-channel and the control is downstream at the channel.
- M/Open/Out/Dc: The slope of the outlet is mild. The flow through the outlet is open-channel and the control is downstream at critical depth.
- M/Prs/Out: The slope of the outlet is mild. The flow through the outlet is under pressure and the control is downstream at the channel.
- M/Orf/In: The slope of the outlet is mild. The hydraulic control is at the inlet and is orifice flow.
- S/Open/In/Dc: The slope of the outlet is steep. The flow through the outlet is open-channel and the control is at the inlet at critical depth.
- S/Prs/Out: The slope of the outlet is steep. The flow through the outlet is under pressure and the control is at the outlet (at the channel).
- S/Prs/In: The slope of the outlet is steep. The flow through the outlet is pressurized and the control is at the inlet (at the basin).
- S/Orf/In: The slope of the outlet is steep. The hydraulic control is at the inlet and is orifice flow.

Q inlet #1 (cfs): Discharge values shown in this column represent the discharge values entering the basin from sources other than the weir. Such sources may be storm drains, weirs from adjacent basins or rainfall over the basin site.

Q inlet #2 (cfs): Discharge values shown in this column represent the discharge values entering the basin from sources other than the weir. Such sources may be storm drains, weirs from adjacent basins or rainfall over the basin site.

Q inlet #3 (cfs): Discharge values shown in this column represent the discharge values entering the basin from sources other than the weir. Such sources may be storm drains, weirs from adjacent basins or rainfall over the basin site.

Low-Flow Inlet (cfs): Discharge values shown in this column represent the discharge values entering or exiting the basin through the Environmental Inlet. Positive discharge values in this column represent flows into the basin while negative values represent flows out of the basin.

Qch @ Outlet (cfs): Discharge values shown in this column represent the discharge values in the main channel downstream of the confluence with the basin's outlet.

WS @ ds Weir (ft): Values shown in this column represent the channel's water surface elevation values at the downstream end of the weir.

WS @ us Weir (ft): Values shown in this column represent the channel's water surface elevation values at the upstream end of the weir.

WS @ Outlet (ft): Values shown in this column represent the channel's water surface elevation values at the basin's outlet.

WS @ Basin (ft): Values shown in this column represent the basin's water surface elevation values.

Vol. Change (Ac-ft): Values shown in this column represent the volume change within the basin for the current time interval (from the previous time-increment to the current one).

Net Vol. (AF): Values shown in this column represent the net volume within the basin at the current time step.

Weir Flow Type: This column includes information regarding the flow hydraulics at the weir. The following are the various types of flows at the weir:

No Flow: No flow occurs at the weir.

Submerged: The flow at the weir is into the basin and is submerged (the water surface elevation in the basin exceeds the weir crest elevation).

Unsubmerged: The flow at the weir is into the basin and is not submerged (the water surface elevation in the basin is below the weir crest elevation).

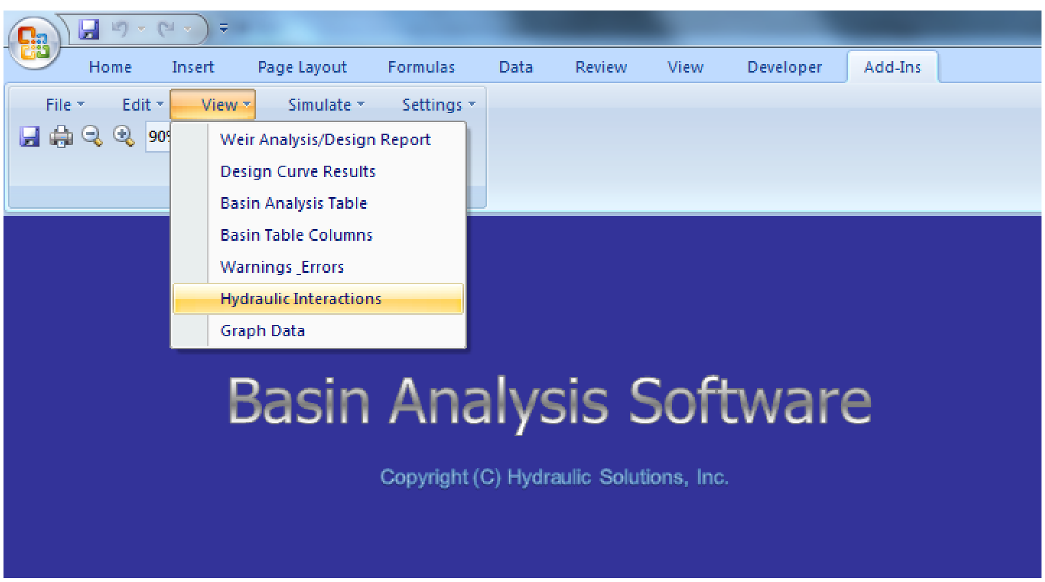
WS Balanced: The water surface elevation in the channel equals the water surface elevation in the basin and no flow occurs at the weir.

Subm. Rev. : The flow at the weir is out of the basin (into channel) and is submerged (the water surface elevation in the channel exceeds the weir crest elevation).

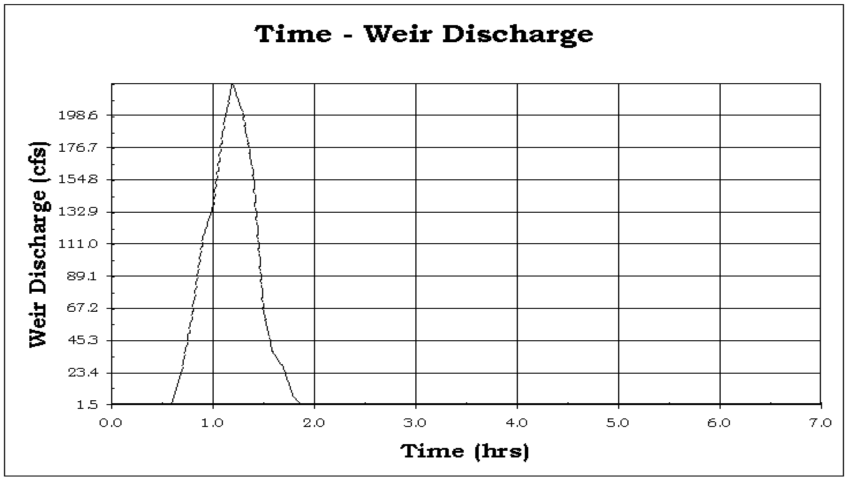
Unsubm. Rev. : The flow at the weir is out of the basin (into channel) and is not submerged (the water surface elevation in the channel is below the weir crest elevation).

OPTIONAL BASIN ANALYSIS OUTPUT - RELATIONSHIPS

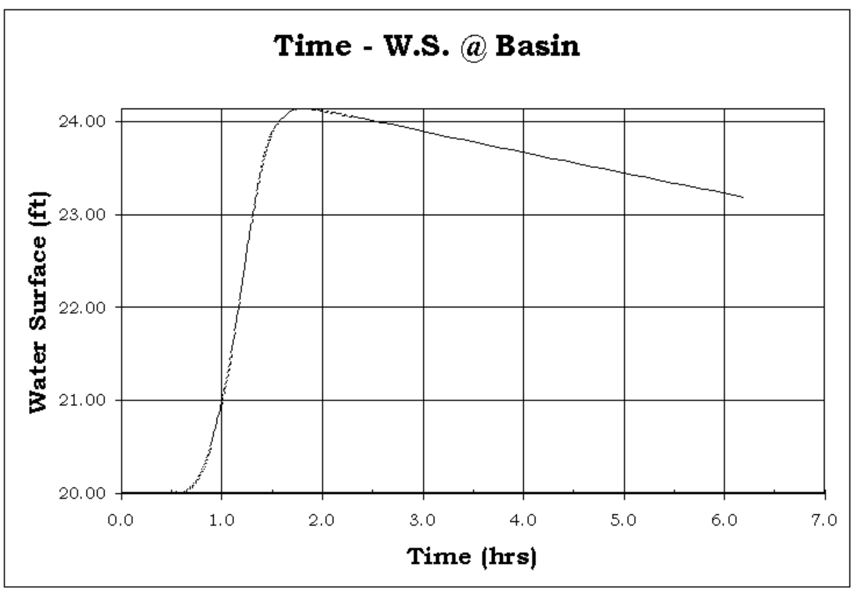
BAS includes a very useful informational tool for the user. To realize a thorough understanding of the basin/channel function during the hydrograph cycle, the user may access the Relationships utility and view various hydraulic behaviors of the system. For example, the user may wish to view the functioning of the weir during the design storm or the behavior of the outlet, etc. To view this information, go to the **View/Hydraulic Interactions** command and select the desired relationship. Only two relationships are provided below for demonstration purposes.



Time Versus Weir Discharge



Time versus W.S.@ Basin



APPENDIX A - BASICS OF HYDRAULICS FOR FLOOD CONTROL

TYPES OF FLOW

- Pressure Flow fills conduit
- Open-channel Water surface exposed to atmospheric pressure
- Laminar Layers of fluid slide over each other in smooth parallel paths
- Turbulent Vortices and rotation of fluid particles occur
- Steady Discharge constant with respect to time
- Unsteady Discharge varies with respect to time
- Continuous Discharge constant with respect to distance
- Discontinuous Discharge varies with respect to distance (spatially varied)
- Uniform Flow depth constant with respect to distance
- Varied Flow depth varies with respect to distance

Rapidly-varied-flow (RVF) - Depth changes abruptly over a short distance i.e. hydraulic jump, hydraulic drop, free outfall, etc..

Gradually Varied Flow (GVF) - Depth changes gradually over a long distance resulting in water surface profile curves

GRADIENTS

- Channel Gradient (S_o) — Physical slope of channel
- Hydraulic Gradient (S_{HGL}) — Slope of the hydraulic grade line (HGL)
- Energy Gradient (S_f) — Slope of the energy grade line (EGL)

CONSERVATION PRINCIPLES

Conservation of Mass

According to this principle, the total mass in a system is conserved.

Mass $_{in}$ = Mass $_{out}$ resulting in $Q_1 = Q_2$ where Q is the discharge in cubic feet per second and is equal to VA, then $V_1A_1=V_2A_2$

Conservation of Momentum

The definition of this principle is best phrased by Newton as, “The change in momentum per unit time in the body of flowing water is equal to the resultant of all external forces acting on the body”.

$M_2 - M_1$ = all forces acting on body from 1 to 2 = Pressure, Weight, Friction and any others

$$M_2 - M_1 = \Delta M = \sum F$$

$$\sum F = -F_f + W\sin\theta + P_1 - P_2 + others$$

$$M = \frac{\gamma QV}{g}$$

$$P = \gamma \bar{Z} A$$

$$W = \gamma b \bar{d} L$$

$$F_f = \gamma h_{lf} b \bar{d}$$

$$\therefore \frac{\gamma Q(V_2 - V_1)}{g} = -F_f + W\sin\theta + P_1 - P_2 + others$$

Conservation of Energy

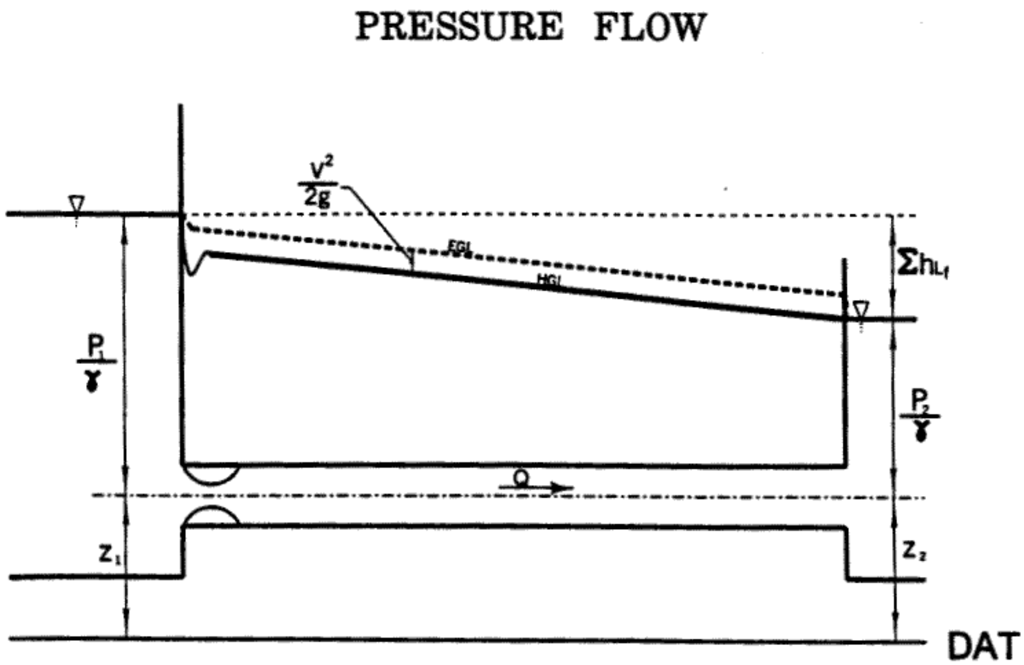
According to this principle, the total energy in a system is conserved.

Energy at section 1 = Energy at section 2 + Losses

Bernoulli eq. $\frac{P_1}{\gamma} + Z_1 + \frac{V_1^2}{2g} = \frac{P_2}{\gamma} + Z_2 + \frac{V_2^2}{2g} + \sum h_l$ (PIPE FLOW)

$$d_1 + Z_1 + \frac{V_1^2}{2g} = d_2 + Z_2 + \frac{V_2^2}{2g} + \sum h_l \quad \text{(OPEN-CHANNEL)}$$

$$\sum h_l = \sum Losses = \sum Minor + \sum Major = h_{lm} + h_{lf}$$



Major Losses

Major losses result from the dissipation in the flow's energy due to the friction between the fluid and its flow boundary. The three most common equations are Manning, Darcy-Weisbach and Hazen-Williams. Manning's may be used for Open-channel and Pipe Flow (water), Darcy-Weisbach is best used for pipe flow (all fluids) and Hazen Williams is best used for pipe network analyses (water). The latter is not applicable to this manual and will not be mentioned further. All equations result from the basic form $h_{lf} = KV^N$, where $h_{lf} = S_f L$

$$h_{lf} = \frac{n^2 L V^2}{2.208 R^{4/3}} \quad \text{(Manning for Open-channel and Pipe Flow)}$$

$$h_{lf} = \frac{f L V^2}{2Dg} \quad \text{(Darcy-Weisbach for Pipe Flow)}$$

Note: f varies with temperature, viscosity, velocity, absolute roughness and pipe diameter

Minor Losses

Minor losses result from either a change in the magnitude of the velocity or a change in the direction of flow. Typical minor losses in a storm drain are entrance, exit, bends, transitions, junctions and manholes. A typical minor loss function is

hlm = $\frac{K_m V^2}{2g}$

Transition Loss

Typically are equal to the change in velocity head from one flow section to the next multiplied by a coefficient.

For Contractions, **hlm** = $K_c(\frac{V_2^{\ 2}}{2g} - \frac{V_1^{\ 2}}{2g})$ where K_c varies from 0.1 for gradual contractions to 0.5 for abrupt contractions.

For Expansions, **hlm** = $K_{\exp}(\frac{V_1^{\ 2}}{2g} - \frac{V_2^{\ 2}}{2g})$ for K_{exp} values, see Appendix D (Outlet Conduits)

Junction Loss

$$P_2 + M_2 = P_1 + M_1 + M_3 Cos\theta + P_i + P_w - P_f$$

Manhole Loss

Manhole Loss **hlm** = $K_{mh}\frac{V^2}{2g}$ where $K_{mh} = 0.05$

Bend Loss

Bend Loss $h_{lb} = \frac{K_b V^2}{2g}$

Where $K_b = 0.25\sqrt{\frac{\theta}{90}}$ or other relationships provided in Appendix D (Outlet Conduits) may be used.

Angle Point Loss

Angle Point Loss **hlm** = $K_{an}\dfrac{V^2}{2g}$ where $K_{an} = 0.02$

Flap-gate Loss

Flap-gate Loss $h_{lm} = \left(8^{\frac{-1.15V_{pipe}}{\sqrt{D}}}\right)\dfrac{V^2}{2g}$

Other minor loss factors are presented in Appendix D (Outlet Conduits).

UNIFORM FLOW

Uniform flow occurs when the flow depth is constant with respect to distance. This depth is called *Normal Depth* (d_n)

Energy Gradient (EGL slope, S_f) = Hydraulic Gradient (Water Surface slope, S_{HGL}) = Channel Gradient (Channel slope S_0)

Energy Gradient is typically calculated using the Manning's equation.

Manning

$$h_{lf} = \frac{n^2LV^2}{2.208R^{4/3}}$$

then $S_{lf} = \dfrac{h_{lf}}{L}\dfrac{n^2V^2}{2.208R^{4/3}}$

Note: *n* may vary with flow depth and/or flow regimes (soft channels). For composite sections, calculate an $n_{composite}$

$$n_{composite} = \left[\frac{P_{w1}n_1^{3/2} + P_{w2}n_2^{3/2} + P_{wi}n_i^{3/2}}{P_{W\,Total}}\right]^{2/3}$$

COMPUTATION OF NORMAL DEPTH (d_n)

Normal depth is the depth at which Uniform Flow occurs. There are several methods for computing Normal Depth (dn). The classic method utilizes King's "Handbook of Hydraulics" tables. Computer use replaced the traditional methods of either manual computation or using the above mentioned tables. All methods solve the basic friction loss equations.

$$h_{lf} = \frac{n^2 L V^2}{2.208 R^{4/3}}$$

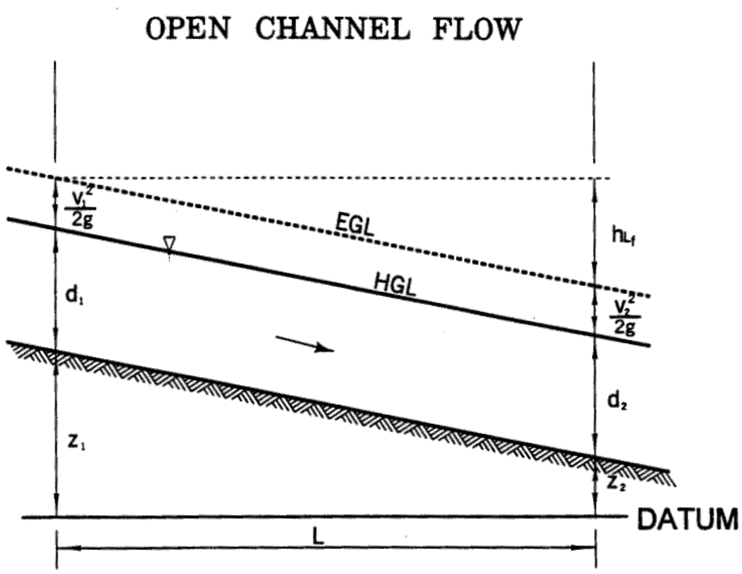
$$S_f L = \frac{n^2 L Q^2}{2.208 A^2 R^{4/3}}$$

$$S_f = S_0$$

re-arranging

$$AR^{2/3} = \frac{nQ}{1.49 {S_0}^{1/2}}$$

$(bd_n + zd_n{}^2)\left[\frac{(bd_n + zd_n{}^2)}{b + 2d_n\sqrt{1+z}}\right]^{2/3} = \frac{nQ}{1.49 S^{1/2}}$ Knowing n,Q,S,z and b, can solve (Iteratively) for d_n



FROUDE No. (F_r) & FLOW REGIMES

Froude No. is a dimensionless number that relates the average velocity of flow to the speed of a fluid wave.

Froude No. = $\frac{V}{\sqrt{g d_h}}$, where d_h = hydraulic depth = $\frac{A}{T}$ and T = Top width at water surface

Flow in an open-channel may be any one of the following:

Critical	F_r =1.0	$V=V_c$	Unstable Flow
Subcritical	F_r < 1.0	$V<V_c$	For F_r =0.9-1.0, Flow is Unstable
Supercritical	F_r >1.0	$V>V_c$	For F_r =1.0-1.2, Flow is Unstable

Note: As shown in the table, design should avoid the unstable Froude No. range of $0.9 < F_r < 1.2$ (small change in energy results in large change in depth)

CRITICAL DEPTH (d_c)

- Critical depth is the depth at which the velocity is critical (equals the speed of a fluid wave)
- Critical Depth is a function of the discharge (Q) and the cross-sectional geometry of flow area
- At Critical Depth, specific energy and specific force are minimum
- At Critical Depth, the Froude No.(F_r) = 1.0 = $\frac{V}{\sqrt{gd_h}}$, $d_h = \frac{A}{T}$ and T = Top width at water surface
- Critical depth for a rectangular cross-section is arrived at by the following derivation:

F_r = 1.0 = $\frac{V}{\sqrt{gd_h}}$, $d_h = \frac{A}{T}$ and T = Top width at water surface then

$$d_h = depth = dc$$

$$1 = \frac{V}{\sqrt{gdc}}$$

$$V = \sqrt{gdc}$$

$V^2 = gdc$ since $V^2 = \frac{Q^2}{A^2}$ and $A^2 = b^2 dc^2$

then $\frac{Q^2}{b^2 dc^2} = gdc$

rearranging $dc^3 = \frac{Q^2}{gb^2}$

finally $dc = \sqrt[3]{\frac{Q^2}{gb^2}}$

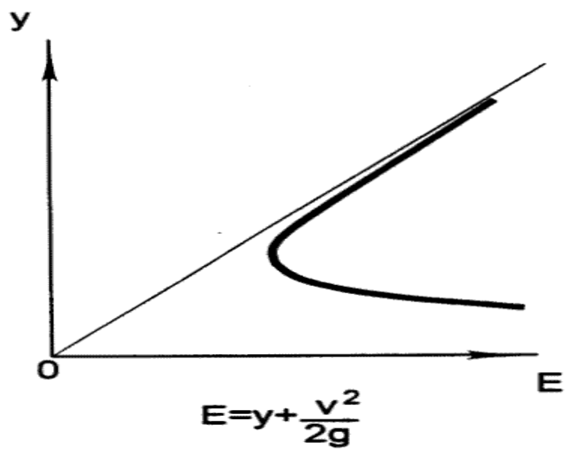
Critical depth may be beneficial in the following ways:

- Flow measurement
- Hydraulic Control
- Flow regime determination

SPECIFIC ENERGY (E)

Energy per pound of water at any section measured with respect to invert (Depth vs. Energy)

- The Specific Energy Curve shows the relationship between E and d
- The Specific Energy Curve shows the relationship between flow regimes (discussed below).
- The Specific Energy Curve shows the relationship between varying width channels.
- The depths at the same specific energy are called *Alternate Depths*
- Specific Energy = E = d + $\frac{V^2}{2g}$



Specific–energy curve

SPECIFIC FORCE (F)

Force per pound of water at any section measured with respect to invert (Depth vs. Force)

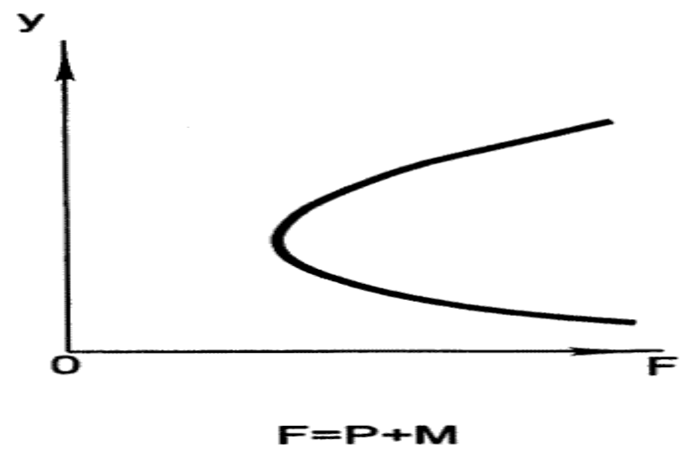
- The Specific Force Curve shows the relationship between F and d
- The Specific Force Curve shows the relationship between flow regimes (discussed below).
- The depths at the same specific force are called *Sequent (or Conjugate) Depths*
- Specific Force = Pressure + Momentum = F

$$P = \gamma \bar{z} A$$

where

$$M = \frac{\gamma Q V}{g}$$

F = $\gamma \bar{Z} A + \frac{\gamma Q V}{g}$



Specific-force

VARIED FLOW

Flow depth varies with respect to distance. In general, water surface profile approaches normal depth asymptotically and critical depth vertically.

RAPIDLY VARYIED FLOW (RVF)

Depth varies abruptly over a short distance i.e. hydraulic jump, hydraulic drop, free outfall, etc..

Hydraulic Jump (HJ)

The following are characteristics of hydraulic jumps.

- Sudden rise in depth due to a flow regime change from supercritical to subcritical.
- Involve a large loss of energy through the dissipation of turbulent underflows.
- The depth before the HJ is always less than the depth after.
- The Force before the HJ is equal to the Force after the HJ.

Practical Application for Flood Control - The HJ may be applied in order to dissipate the energy of the flow and reduce scour and degradation. Other benefits that are non-flood control related are: a) raise water surface to a higher level for water supply and irrigation, b) mix chemicals, c) aerate water, etc..

Detrimental Effects - a) Highly turbulent and will cause scour in earthen channels, b) the rise in water surface may cause flooding, c) extremely dangerous if one is trapped in it.

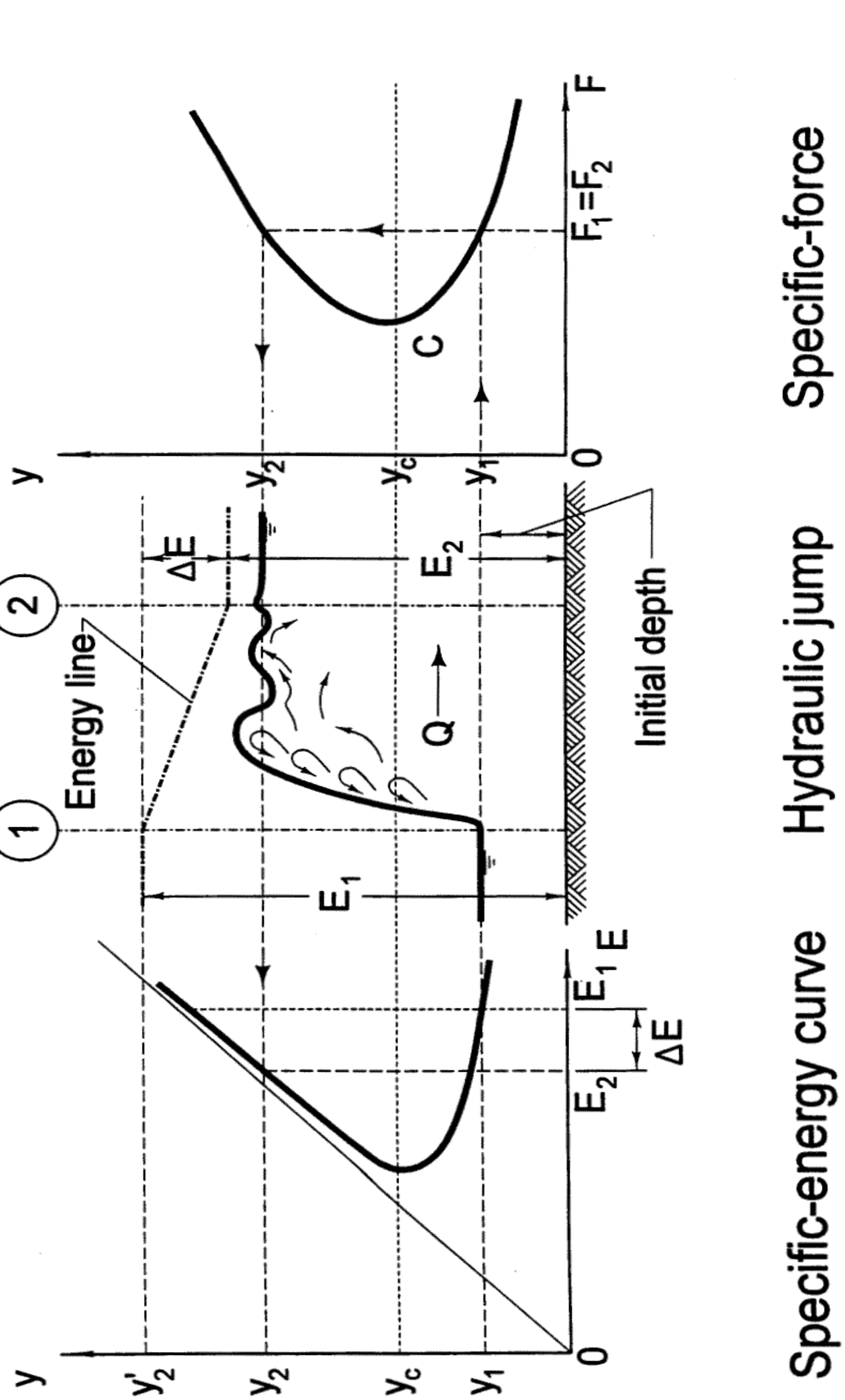


Figure above from Chow (1959)

Since the force before the HJ equals the force after, a relationship between the depth before (d_{us}) and the depth after (d_{ds}) may be derived. These depths are called *Sequent (or Conjugate)* depths

$$d_{us} = \frac{d_{ds}}{2}\left(\sqrt{1+8F^{2}{}_{r\,ds}} \;-1\right)$$

and

$$d_{ds} = \frac{d_{us}}{2}\left(\sqrt{1+8F^{2}{}_{r\,us}} \;-1\right)$$

Hydraulic Drops

Depth of flow decreases rapidly in short distance

Depth at edge is ~70% of critical depth

Critical Depth occurs at a distance of 3-4dc from brink

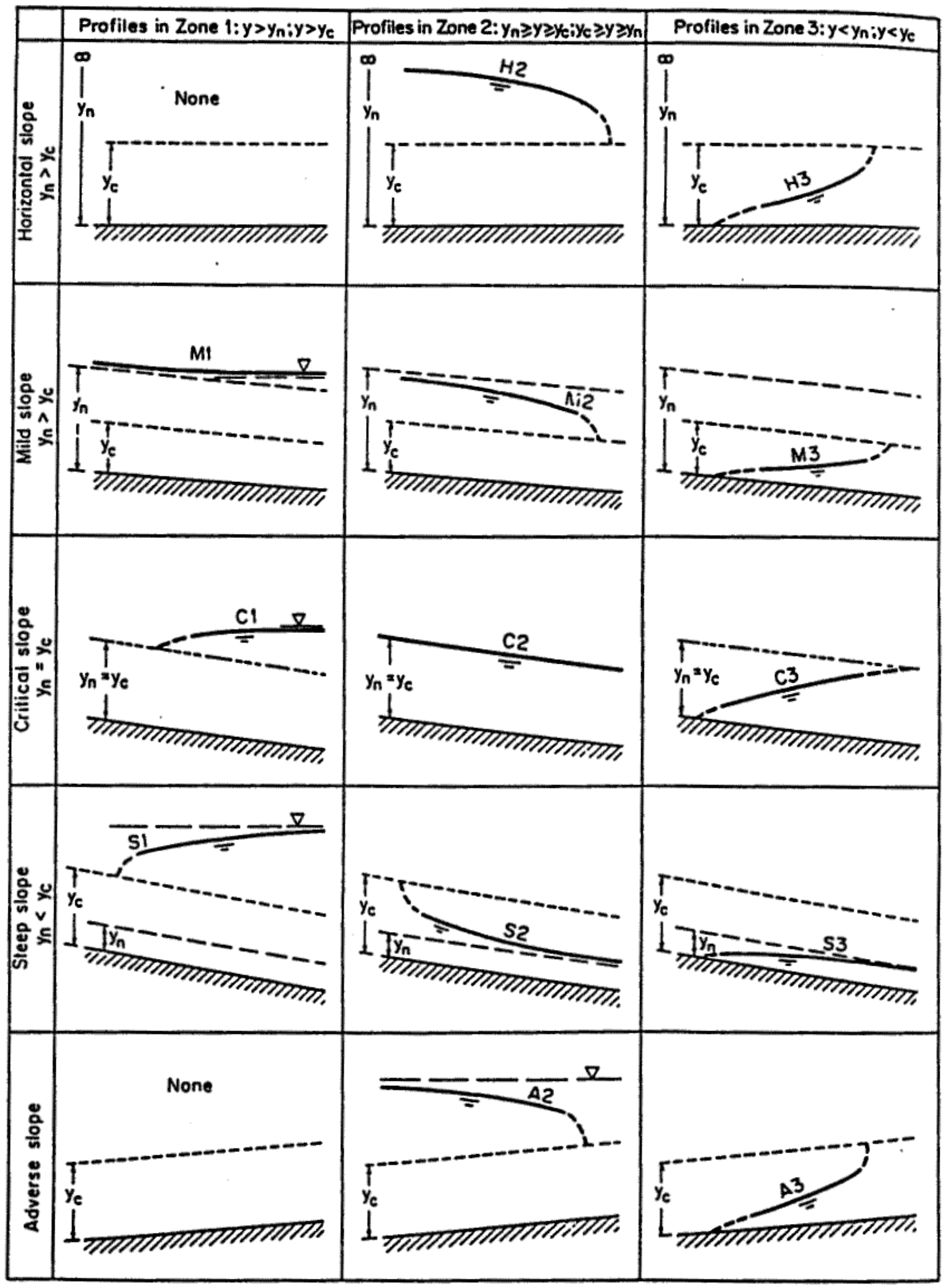
GRADUALLY-VARIED-FLOW (GVF)

In contrast to Rapidly-varied-flow, depth changes gradually over a long distance resulting in water surface profile Curves. The change in depth (and water surface profile) would resemble one of 12 curves for the five available types of slopes.

The following is a summary of the curves and their properties:

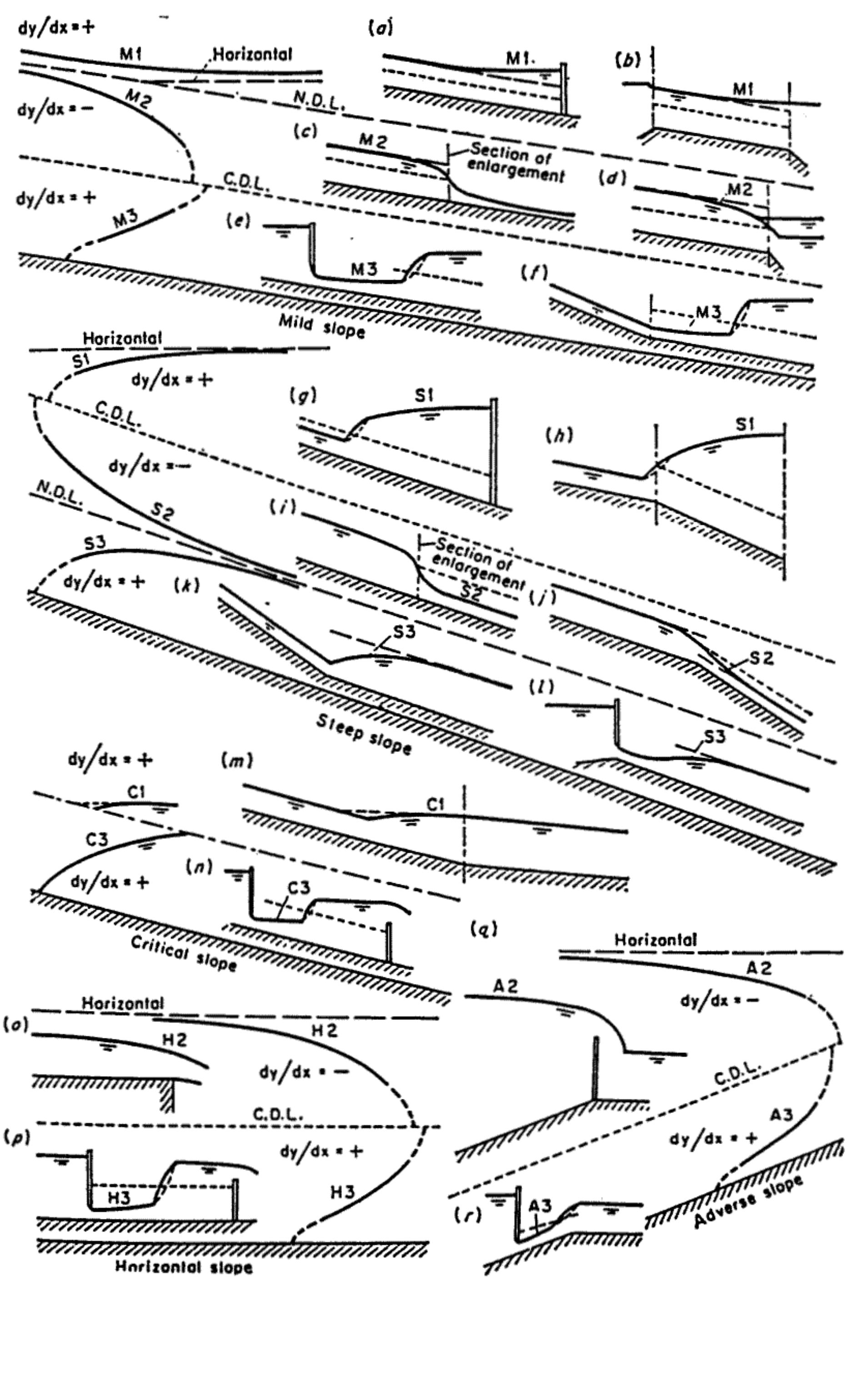
Slope	Curve Designation	Relation of d to dn & dc	Curve	Type of Flow
Horizontal S=0	H1	dn = Infinity	None	None
	H2	dn>d>dc	Drawdown	Subcritical
	H3	dn>dc>d	Backwater	Supercritical
Mild Sc>S0>0	M1	d>dn>dc	Backwater	Subcritical
	M2	dn>d>dc	Drawdown	Subcritical
	M3	dn>dc>d	Backwater	Supercritical
Critical (Sc=S0)>0	C1	d>(dc=dn)	Backwater	Subcritical
	C2	dn=d=dc	None (straight-line)	Uniform-Critical
	C3	(dc=dn)>d	Backwater	Supercritical
Steep S0>Sc>0	S1	d>dc>dn	Backwater	Subcritical
	S2	dc>d>dn	Drawdown	Supercritical
	S3	dc>dn>d	Backwater	Supercritical
Adverse S0<0	A1	dn undefined	None	None
	A2	dn>d>dc	Drawdown	Subcritical
	A3	dn>dc>d	Backwater	Supercritical

GRADUALLY-VARIED-FLOW CURVES



Note: $Y = d$, $Y_n = d_n$ and $Y_c = d_c$

GRADUALLY-VARIED-FLOW CURVES - EXAMPLES



POINT OF CONTROL

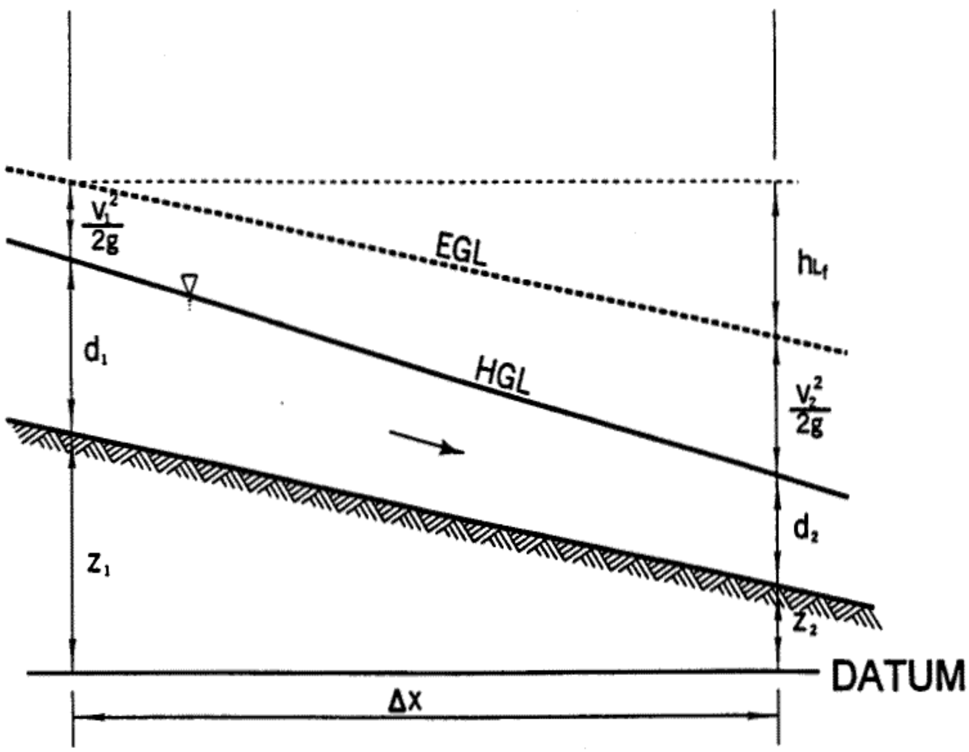
The Point of Control (POC) is a location in the hydraulic system where either:

a) there is a known water surface (reservoirs, ocean etc.), or

b) there is a unique relationship between the depth and the discharge

The POC is where gradually-varied-flow calculations initiate from. In a subcritical flow regime, the calculations start at the POC and move to the upstream direction, while in the supercritical flow regime, the calculations start at the POC and move to the downstream direction.

GRADUALLY-VARIED-FLOW COMPUTATION



Several methods are available for calculating the gradually-varied-flow profiles. The primary methods used are the Direct Step (prismatic channels) and the Standard Step methods. Both of these methods solve the basic energy equation

$$d_1 + Z_1 + \frac{V_1^2}{2g} = d_2 + Z_2 + \frac{V_2^2}{2g} + h_{fl}$$

from earlier

E = d + $\frac{V^2}{2g}$

re-arranging,

$$E_1 - E_2 = Z_2 - Z_1 + h_{lf}$$

where

$$h_{lf} = \bar{S}_f \Delta X \;, \qquad \bar{S}_f = \frac{S_{f1} + S_{f2}}{2}, \qquad S_f = \frac{n^2 V^2}{2.22 R^{4/3}}$$

and

$$Z_1 - Z_2 = S_o \Delta X$$

then

$$E_1 - E_2 = -S_0 \Delta X + \bar{S}_f \Delta X$$

$$\Delta X = \frac{E_1 - E_2}{-S_0 + \bar{S}_f}$$

Direct Step Method

Using the re-arranged energy equation with a known POC (either E_1 or E_2), assume the other E and solve for ΔX . This method is the simplest and may be easily applied to a prismatic channel.

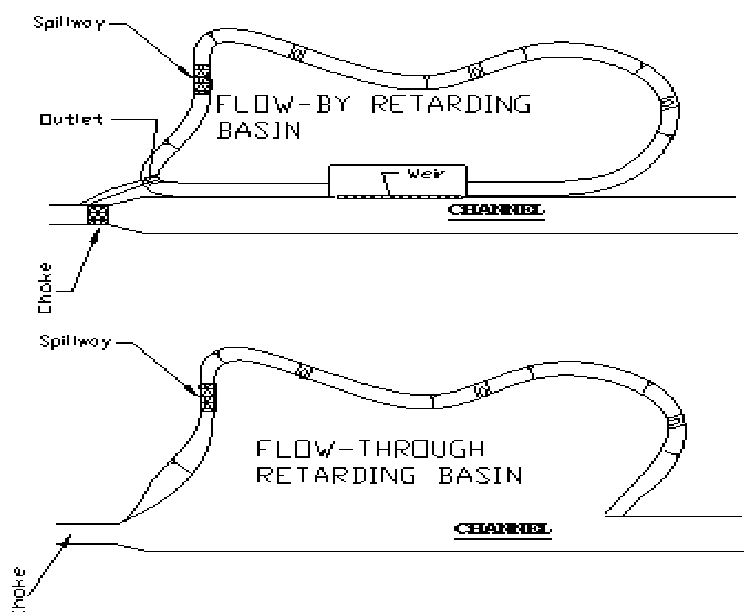
Standard Step Method

Using the re-arranged energy equation with a known POC (either E_1 or E_2), set a fixed ΔX and solve for the other E. This method may be applied to any type of channel; however, it involves several iterations for each step of the calculations. Almost all of the water surface calculation programs use this method, such as: LA County's WSPG, COE's HEC2 and HECRAS, etc....

APPENDIX B - THE OFFLINE RETARDING SYSTEM

Over time, development continued in floodplains despite the impact on the floodplain and the accompanied flood risks. In most cases, measures were taken to provide some flood protection. In others, adequate understanding of the development's impacts to the floodplain didn't exist. Where measures were taken, they usually consisted of concrete-lining channels. With time, some of these channels became inadequate to convey the intended flood discharges. The options that became available included retarding systems which reduce the peak of the storm to a level that can be conveyed in the existing channel. Such retarding systems consisted of two types of retarding basins; the classic online and the offline. The terms "offline" and "online" are often exchangeable with "offline" and "online" respectively. Although the mechanics of the offline were somewhat understood, yet difficult to analyze and design, online basins flourished. They were much simpler to analyze and design than their counterpart offlines. Engineers and public entities knew exactly what to look for since such basins are the grandchildren of the classic dams. Neither system is intended to permanently store water but only delay its discharge into the downstream drainage system. These two types are shown in the figure below:

TYPES OF RETARDING SYSTEMS



DESIGN OBJECTIVES – Questions that must be asked

The following are critical questions that should be considered in order to properly define the objectives of a retarding basin (ASCE 1985):

Hydraulic Function

- What are the hydrologic characteristics of the tributary basin?
- What is the future development potential of the basin and resulting runoff peak flows, volumes, and hydrograph characteristics?
- What are the design inflows for various frequencies of occurrence?
- What are the design outflow objectives?
- How does and how should the detention facility interact with the overall basin drainage and flood control system?
- Should the detention pond outlet control only one design flow, or a full range of frequencies?
- How precise outflow controls need to be in terms of matching the desired hydrograph?

Water Quality

- Is it important to detain a large portion of the volume of runoff, or is it satisfactory to merely reduce the peak of the runoff hydrograph?
- To what extent should water quality be improved and for which pollutants?

Safety

- Is the proposed detention facility located in a residential, commercial, industrial, or agricultural setting and what are the implications of the particular setting?
- Is the basin readily accessible to the public? Is it safe?
- Can the outlet works be simplified in any way to increase reliability and safety?
- What is the risk if an event larger than the design flood overtops the embankment?

Maintenance

- How will trash, debris, and sediment be controlled?
- Who will maintain the detention facility? Are resources available for maintenance?

- What will be the rate of sediment accumulation in the basins? How will sediments be removed? Where will they be disposed off?
- How will the basin's outlet(s) be accessed?
- Does the basin need to be devoid of vegetation? If not, what vegetation is desirable?
- How will the slopes be protected from erosion?

Aesthetics and Multiple Use

- Should the detention facility have multi-purposes such as recreation and water quality improvement? Should the basins be wet or dry?
- Should the basin be used as a wetlands mitigation bank?
- Is appearance a factor? Is the basin readily visible to the public?
- Can it serve as an amenity to the neighborhood?

General

- Will benefits from the pond exceed the total costs of land, construction, maintenance, added infrastructure extensions, investment amortization, and reduced tax revenues?

CAUTIONS ON RETARDING

It is paramount to realize that retarding may not be a solution to each and every flood control challenge. There are several cautions that should be realized when considering retarding within a flood control system. The following is a summary of those cautions:

- Effectiveness decreases as the watershed size increases. The watershed downstream of the retarding basin has a shorter time of concentration (when analyzed separately) and higher intensity rainfall.
- If a system is designed for a specific storm, i.e. 100-year, it may not be effective in controlling runoff from more frequently occurring storms of 2 or 10 years.
- Random on-site detention effects are uncertain and it is unwise to assume that flows along major drainage ways will remain at existing levels in the future.
- A retarding basin is designed for an estimate of both a peak flow and peak volume while an all-channel system is designed for an estimate of peak flows only. Therefore, uncertainties may increase in the retarding concept. A retarding basin loses effectiveness after it becomes full;

whereas, an all-channel system provides significant benefits for floods greater than its design capacity.

- Random on-site detention can hold back the runoff until the peak flow arrives from upstream, thereby increasing the peak flows downstream and actually causing more harm than good.
- Many detention basins have been located in remote areas of developments where access for maintenance is difficult.
- Detention basins are often designed with steep banks where vegetation is difficult to establish and maintain.
- Often outlet structures are very small to limit runoff, but they are also prone to clogging which results in stagnant pools of water remaining for long periods.
- Detention basins become the depository for sediment and urban litter.
- The deposit of sediment (in online basins) deprives the downstream system from sediment, which causes degradation, erosion and local scour.
- Online retarding systems prevent upstream migration of fish.
- Weed control is a major problem, especially in facilities which do not drain dry after each storm event.
- Fences and landscaping used to keep children out of the detention area are often damaged and/or destroyed.
- Sediment tends to deposit in the channel (along the side-weir), requiring maintenance after storms.

INFLOW INTO THE BASIN

Inflow types

Weir inflow

- Fully aerated nappe
- Submerged nappe

Inflow hydrographs from other tributaries

- Channels
- Creeks
- Ditches
- Storm Drains

- Rainfall (over basin) hydrograph - Significant only for large basin surface areas

Inflow through the outlet

Where outlets are not gated, flows may reverse through the outlet when the water surface in the main channel (at the outlet) is higher than that of the basin. Flapgates, which prevent such reverse-flows, may significantly reduce volume requirements. Flapgates are most beneficial when flows in the channel (at the outlet) are very subcritical.

Inflow through an Environmental Inlet

Such openings are not currently common. However, there may be cases where low-flows need to be directed into the basin through an opening in the channel wall (between the basin and channel). Such an opening would typically be needed for water quality treatment of the low-flows. Depending upon the size of the opening, the discharge through the opening (and its associated volume) may have a significant impact upon the available storage volume where the basin's volume could be depleted earlier than desired. The sizing of an optimum inlet includes performing sensitivity analyses. On the other hand, the opening provides an incidental benefit as it acts as an outlet during the receding portion of the hydrograph and helps drain the basin faster.

OUTFLOW OUT OF THE BASIN

Outflow types

Outlet conduits

- Pipes
- RCB's
- Standpipes

The most recommended outlet conduit is a pipe outlet due to its efficiency and economy. When allowed, flapgates may be used. However, the system should not be designed based upon the presence of flapgates.

Reverse-flow over the weir

The retarding basin typically receives storm flows through the outlet and over the weir. However, as the basin fills, its water surface exceeds the weir crest and the water surface in the channel recedes, then reverse-flows occur over the weir.

Spillway Flow

Similar to reverse-flows over the weir, the spillway also acts as an outlet device when the water surface in the basin exceeds the spillway's crest elevation.

Environmental Inlet

The environmental inlet is an opening in the channel wall that allows flows into or out of the basin depending on the direction of the head differential.

Pumped Flow

In cases where the basin's invert elevation needs to be set below the channel's invert at the location of the outlet, the initial impoundment needs to be pumped.

STORAGE

The flood storage in a basin is the net volume of all inflows, outflows and initial storage. The following is the simplified equation:

Net Storage = Initial Storage + Time (Gross Inflow – Gross Outflow)

Where: Initial Storage is the assumed storage at the beginning of the storm.

Time is the hydrograph time-increment

Gross Inflow is the total (for each time-increment) inflow into the basin including weir inflow, rainfall, tributaries, reverse-flow from outlet, environmental inlet, etc.

Gross Outflow is the total (for each time-increment) outflow from the basin including outlet, spillway, reverse weir, environmental inlet, etc.

THE OFFLINE ROUTING CYCLE – AN OVERVIEW

THE STAGES OF A STORM

Below is a progression of the stages of a storm along and through a classical offline retarding basin. There are of course cases that do not follow this behavior. For example, the basin may be large enough so as not to result in a water surface which submerges the side-weir or causes reverse-flows over the weir.

STAGE 1 - PRIOR TO WEIR FLOW

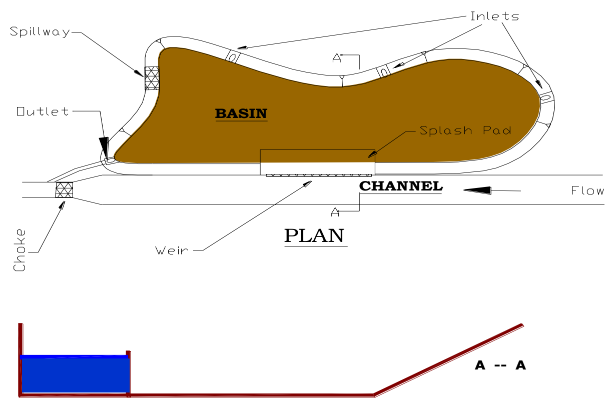
This is the beginning of the design storm where the basin is empty and the water depth in the channel is below the weir crest elevation. It is unlikely that uniform flow could exist in the main channel since a properly designed by-pass channel would include a choke in order to lift the water surface above the weir crest and into the basin. At this stage, it is likely that an M1 curve forms since the channel is intended to be a mild sloped channel.

Although no weir flows occur during this phase, flows may enter the basin from one or more of the following sources:

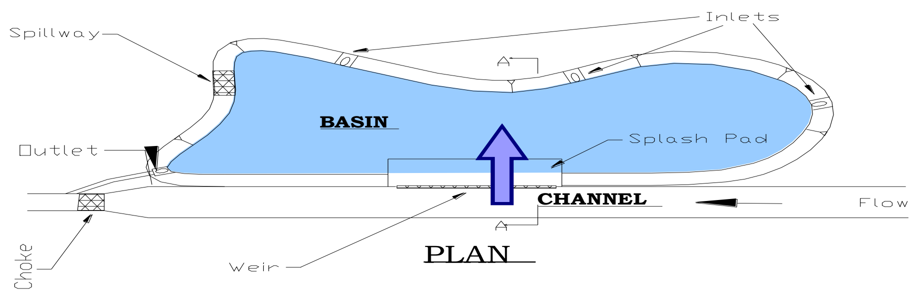
- Other inlets into the basin
- Reverse-flow through the outlet
- Rainfall over the basin
- Environmental Inlet

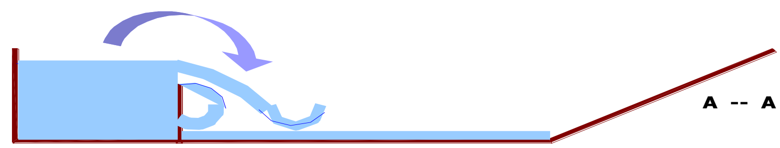
If such flows exist, then there may be some flows exiting the basin through its outlet if the water surface in the basin exceeds that of the channel at the location of the outlet.

STAGE 2 - UNSUBMERGED WEIR FLOW INTO THE BASIN

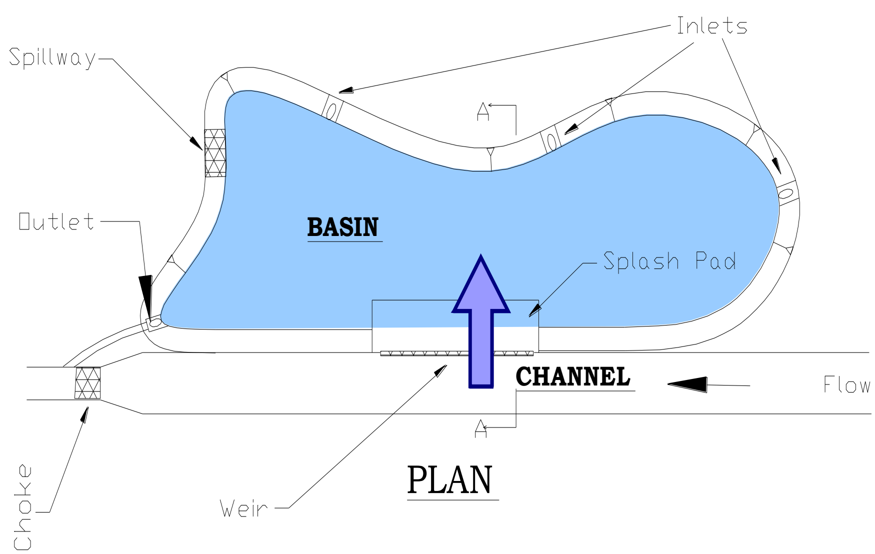


As the storm progresses and the discharge in the channel increases, the effect of the choke (downstream of the weir) becomes more pronounced and the depth increases rapidly. Once the depth exceeds the weir crest, weir flows into the basin commence. By this time, it is likely that the basin is receiving flows from other sources (as mentioned in the previous step). Also, it is likely that the outlet will be releasing stored volume in this stage.





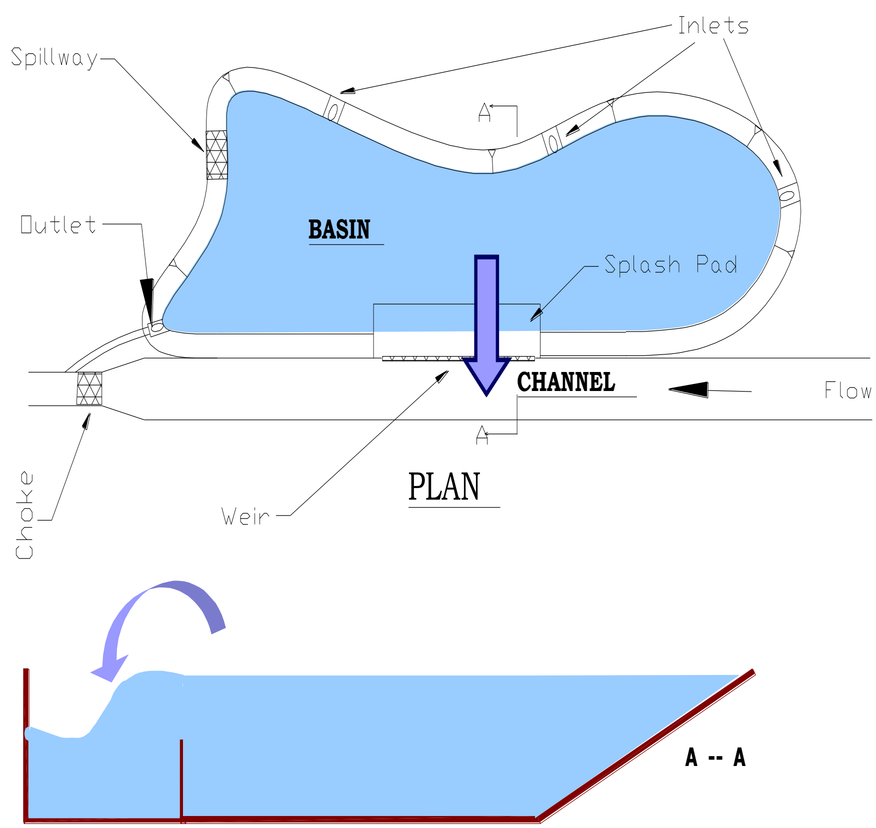
STAGE 3 - SUBMERGED WEIR FLOW INTO THE BASIN



Depending upon the available volume within the basin, the water surface may exceed the weir crest during incoming flows over the weir. This is called submerged-flow. The effect of this submergence is a decrease in the weir's coefficient of discharge and therefore the weir discharge. Since less flows are able to enter the basin, more flows are proceeding downstream in the by-pass channel. The outlet is likely to be releasing flows during this stage.

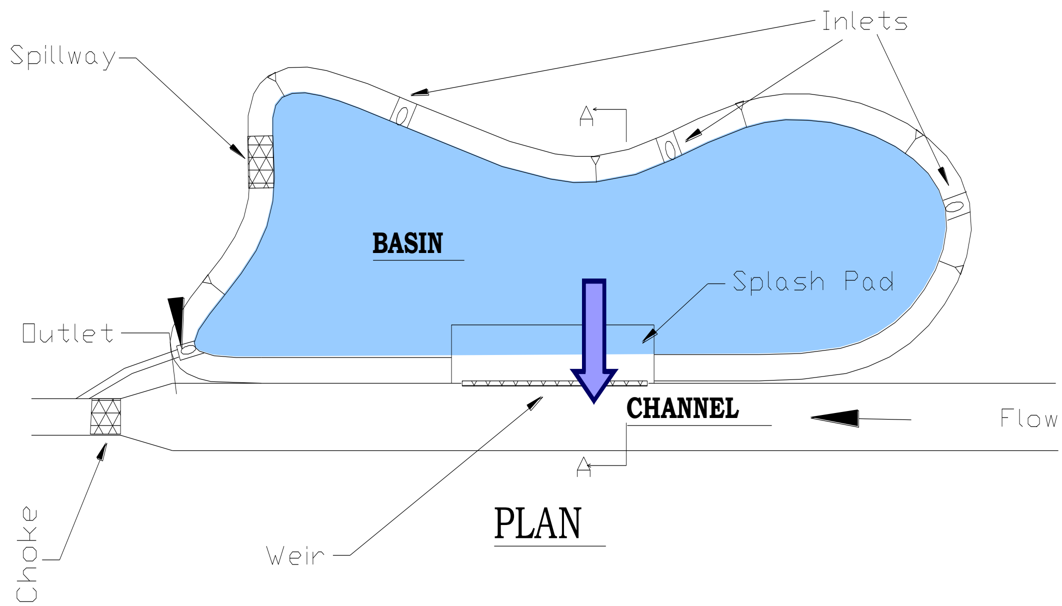
STAGE 4 - SUBMERGED REVERSE-FLOW OUT OF THE BASIN

After operating under the submerged weir stage for some time, and the channel's hydrograph is on the receding leg, the basin's water surface will momentarily equal the water surface of the channel. No flows occur over the weir during this short period. As the main hydrograph's discharge decreases further, the water surface in the channel will drop below that of the basin. This will result in reverse-flows over the submerged weir. The outlet and the weir would release stored water during this stage.

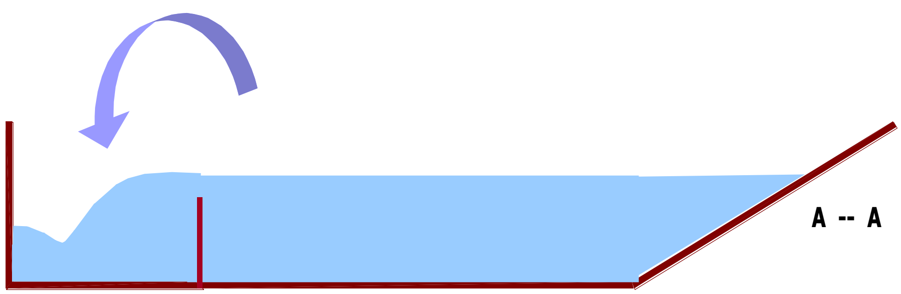


STAGE 5 - UNSUBMERGED REVERSE FLOW OUT OF THE BASIN

Similar to the previous stage, the basin releases water over the weir and through the outlet. However, during this stage, the water depth in the channel has decreased sufficiently so as not to submerge the weir.



Spillway
Inlets
BASIN
Splash Pad
Outlet
CHANNEL
Flow
A
A
Choke
Weir
PLAN



A -- A

COMPONENTS OF THE ANALYSIS

THE INFLOW HYDROGRAPH

This is the main component of the analysis and represents the “load” on the system. The main hydrograph must meet the criteria of the jurisdictional authority. Many tools exist for this analysis such as HEC-1, HECHMS and various others.

INFLOW HYDROGRAPHS FROM OTHER SOURCES

Similar to the main hydrograph these are also loads on the system and must meet the criteria of the jurisdictional authority. Many tools exist for this analysis such as HEC-1, HECHMS and various others.

WEIR RATING

Each weir is capable of conveying flows which depend upon a number of factors as discussed in Appendix C. If the offline system is being analyzed manually, it is convenient to construct a weir rating table in order to simplify the analysis of a system. This table would include the flows upstream, over and downstream of the weir. Then, when performing the routing operation for each upstream flow, a weir flow and by-pass flow is found from the table. However, such a table would typically not include the effects of weir submergence.

DEPTH-DISCHARGE RELATIONSHIP (CHANNEL AT WEIR)

The Depth – Discharge relationship at the downstream end of the weir requires an analysis of the channel system starting at a proper hydraulic control section and proceeding upstream (through the choke) to the downstream side of the weir. This analysis would be completed for a series of flows; starting at very low values and ending at flows slightly higher than the intended by-pass flows. It is recommended that several sets of stage-discharge values be provided at a maximum water surface elevation interval of 0.5-feet. Or, the rating table may start at a discharge of 5 percent of the proposed by-pass discharge at the downstream end of the weir and move up by five percent increments up to 105 percent. To arrive at such a relationship, many tools are available such as HEC-2, HECRAS, WSPG or others.

DEPTH-DISCHARGE RELATIONSHIP (CHANNEL AT OUTLET)

This relationship provides the downstream control for the outlet hydraulics and is part of the same analysis of the system downstream of the weir. The same hydraulic runs used for the Depth-Discharge (Channel at Weir) are used here except at a different location.

THE BY-PASS CHANNEL CHOKE

As mentioned earlier, a properly designed system would include a choke section immediately downstream of the weir. The purpose of this choke is to create a bottleneck for the flow in order to raise it sufficiently over the weir. An efficiently designed system would ensure hydraulic separation between the upstream and downstream of the choke section. This could be accomplished (when possible) by creating a supercritical segment of channel directly downstream of the choke. In essence, this ensures that outlet flows do not create a higher downstream control for the weir section.

DEPTH-VOLUME RELATIONSHIP (BASIN)

This relationship is needed to maintain accounting of the basin volume. Initially, the basin may be assumed as rectangular-based with side-slopes of 3:1. Once a satisfactory system has been preliminarily designed, grading plans may be completed that achieve the same, or similar, stage-volume relationship.

A SIMPLE SPREADSHEET

In order to track the system's inflows and outflows and their respective volumes, a spreadsheet could be constructed. Below would be the simplest form of such a spreadsheet. Naturally, for full detail and tracking, more columns are needed.

HYDROGRAPH		INFLOW				OUTFLOW		Q channel d/s	Vol. Chg.	Net Vol.
Time	Q u/s	Q weir	Q inlet 1	Q inlet 2	Q inlet 3	Q outlet	Q spillway			

The following describes each of the columns above:

- Time = inflow hydrograph time-column
- Q u/s = inflow hydrograph discharge-column
- Q weir = inflow into basin over the weir
- Q inlet 1 through inlet 3 = other tributaries to the basin
- Q outlet = outflow through basin outlet
- Q spillway = outflow over the basin's spillway
- Q ch d/s (outlet) = Q u/s – Q weir + Q outlet

- Vol. Chg. = volume change since the previous time-step and is equal to (INFLOW – OUTFLOW) x Time step (of hydrograph)
- Net Vol. = previous Net Volume + Vol. Chg.

A DETAILED SPREADSHEET

Below is a sample spreadsheet which includes all the pertinent parameters for the proper tracking of the basin volume and system flows.

Time (hrs)	Q ch. @ us Weir (cfs)	Q Weir (cfs)	Discharge Coeff.	Q ch. @ ds Weir (cfs)	Q Spillway (cfs)	Q Outlet (cfs)	Outlet control/type	Q inlet#1 (cfs)	Q inlet#2 (cfs)	Q inlet#3 (cfs)	Low-Flow Inlet (cfs)	Qch. @ Outlet (cfs)	WS @ ds Weir (ft)	WS @ us Weir (ft)	WS @ Outlet (ft)	WS @ Basin (ft)	Vol. Change (Ac-ft)	Net Vol. (AF)	Weir Flow Type
17.139	3859.3	203.6	0.057	3649.7	448.3	-201.6	Prs/Outlet	179.2	3.5	22.1	6.0	3851.2	270.00	269.96	256.35	269.87	-0.545	195.022	Submerged
17.167	3784.0	146.3	0.043	3632.9	423.8	-201.4	Prs/Outlet	175.0	3.5	21.4	4.8	3834.3	269.93	269.91	256.34	269.83	-0.635	194.387	Submerged
17.188	3715.3	94.1	0.029	3621.1	395.9	-201.2	Prs/Outlet	171.8	3.5	20.9	0.0	3822.3	269.86	269.85	256.33	269.80	-0.533	193.855	Submerged
17.208	3646.5			3646.5	373.0	-200.8	Prs/Outlet	168.8	3.5	20.4	0.0	3847.3	269.80	269.53	256.34	269.76	-0.662	193.193	No Flow
17.229	3577.8			3577.8	345.3	-200.8	Prs/Outlet	165.6	3.5	19.9	0.0	3778.6	269.76	269.76	256.31	269.72	-0.620	192.574	No Flow
17.250	3509.0			3515.9	319.9	-200.8	Prs/Outlet	162.5	3.5	19.4	-6.9	3716.6	269.60	269.60	256.28	269.69	-0.594	191.980	No Flow
17.271	3440.3			3449.1	296.2	-200.6	Prs/Outlet	159.4	3.5	18.9	-8.8	3649.7	269.49	269.49	256.26	269.65	-0.562	191.417	No Flow
17.291	3371.5	-183.8	0.376	3564.3	274.4	-200.5	Prs/Outlet	156.3	3.5	18.4	-9.0	3764.7	269.45	269.45	256.25	269.60	-0.849	190.568	Subm. Rev.
17.312	3302.8	-192.1	0.411	3503.9	242.5	-200.3	Prs/Outlet	153.2	3.5	17.9	-9.0	3704.2	269.40	269.40	256.22	269.55	-0.815	189.753	Subm. Rev.
17.333	3234.0	-278.7	0.622	3521.9	213.3	-200.0	Prs/Outlet	150.0	3.5	17.3	-9.2	3721.9	269.34	269.34	256.21	269.50	-0.920	188.833	Subm. Rev.
17.361	3156.7	-341.0	0.800	3507.2	181.8	-199.7	Prs/Outlet	145.8	3.5	16.7	-9.5	3706.8	269.27	269.27	256.20	269.42	-1.310	187.523	Subm. Rev.
17.389	3079.3	-403.1	1.017	3492.4	140.0	-199.2	Prs/Outlet	141.7	3.4	16.0	-10.0	3691.6	269.17	269.17	256.18	269.34	-1.368	186.155	Subm. Rev.
17.417	3002.0	-448.2	1.226	3460.7	100.5	-198.7	Prs/Outlet	137.5	3.4	15.3	-10.5	3659.4	269.06	269.06	256.17	269.25	-1.392	184.763	Subm. Rev.
17.458	2915.5	-436.0	1.300	3362.1	65.1	-197.6	Prs/Outlet	131.3	3.4	14.3	-10.6	3559.7	268.97	268.97	256.23	269.14	-1.899	182.864	Subm. Rev.
17.500	2829.0	-476.8	1.615	3317.4	26.1	-197.0	Prs/Outlet	125.0	3.4	13.3	-11.6	3514.5	268.80	268.80	256.19	269.02	-1.978	180.886	Subm. Rev.
17.541	2749.5	-508.3	1.990	3270.8	1.2	-197.0	Prs/Outlet	118.8	3.4	12.2	-13.0	3467.7	268.59	268.59	256.08	268.90	-1.982	178.904	Subm. Rev.
17.583	2670.0	-530.6	2.445	3215.5		-196.3	Prs/Outlet	112.5	3.4	11.2	-14.9	3411.9	268.33	268.33	256.04	268.76	-2.134	176.771	Subm. Rev.

The following provides a brief description of each of the columns:

Time (hrs): Time values shown in this column represent the hydrograph time values.

Qch @ us Weir (cfs): Discharge values shown in this column represent the discharge values at the very upstream end of the weir (inflow hydrograph).

Q Weir (cfs): Positive discharge values in this column represent flows into the basin while negative values represent flows out (reverse-flow) of the basin.

Discharge Coeff: Discharge coefficient values shown in this column are the side-weir coefficients used in the analysis.

Qch @ ds Weir (cfs): Discharge values shown in this column represent the channel's discharge values at the downstream end of the weir.

Q Spillway (cfs): Discharge values shown in this column represent the discharge values over the basin's spillway.

Q Outlet (cfs): Discharge values shown in this column represent the discharge values through the basin's outlet. Positive values represent flows into the basin while negative values represent flows out of the basin.

Outlet control/type: This column provides information regarding the flow-hydraulics through the basin's outlet. The following are the various types of flows through the outlet:

- No Flow: No flow occurs through the outlet.
- M/Open/Out: The slope of the outlet is mild. The flow through the outlet is open-channel and the control is downstream at the channel.
- M/Open/Out/Dc: The slope of the outlet is mild. The flow through the outlet is open-channel and the control is downstream at critical depth.
- M/Prs/Out: The slope of the outlet is mild. The flow through the outlet is under pressure and the control is downstream at the channel.
- M/Orf/In: The slope of the outlet is mild. The hydraulic control is at the inlet and is orifice flow. This may occur in a very short conduit.
- S/Open/In/Dc: The slope of the outlet is steep. The flow through the outlet is open-channel and the control is at the inlet at critical depth.
- S/Prs/Out: The slope of the outlet is steep. The flow through the outlet is under pressure and the control is at the outlet (at the channel).

- S/Prs/In: The slope of the outlet is steep. The flow through the outlet is pressurized and the control is at the inlet (at the basin).
- S/Orf/In: The slope of the outlet is steep. The hydraulic control is at the inlet and is orifice flow.

Q inlets #1 - #3 (cfs): Discharge values shown in this column represent the discharge values entering the basin from sources other than the weir. Such sources may be storm drains, weirs from adjacent basins or rainfall over the basin site.

Low-Flow Inlet (cfs): Discharge values shown in this column represent the discharge values entering or exiting the basin through the Environmental Inlet. Positive discharge values in this column represent flows into the basin while negative values represent flows out of the basin.

Qch @ Outlet (cfs): Discharge values shown in this column represent the discharge values in the main channel downstream of the confluence with the basin's outlet.

WS @ ds Weir (ft): Values shown in this column represent the channel's water surface elevation values at the downstream end of the weir.

WS @ us Weir (ft): Values shown in this column represent the channel's water surface elevation values at the upstream end of the weir.

WS @ Outlet (ft): Values shown in this column represent the channel's water surface elevation values at the basin's outlet.

WS @ Basin (ft): Values shown in this column represent the basin's water surface elevation values.

Vol. Change (Ac-ft): Values shown in this column represent the volume change within the basin for the current time interval (from the previous time-increment to the current one).

Net Vol. (AF): Values shown in this column represent the net volume within the basin at the current time step.

Weir Flow Type: This column includes information regarding the flow hydraulics at the weir. The following are the various types of flows at the weir:

No Flow: No flow occurs at the weir.

Submerged: The flow at the weir is into the basin and is submerged (the water surface elevation in the basin exceeds the weir crest elevation).

Unsubmerged: The flow at the weir is into the basin and is not submerged (the water surface elevation in the basin is below the weir crest elevation).

WS Balanced: The water surface elevation in the channel equals the water surface elevation in the basin and no flow occurs at the weir.

Subm. Rev.: The flow at the weir is out of the basin (into channel) and is submerged (the water surface elevation in the channel exceeds the weir crest elevation).

Unsubm. Rev.: The flow at the weir is out of the basin (into channel) and is not submerged (the water surface elevation in the channel is below the weir crest elevation).

APPENDIX C - SIDE-WEIR FLOW

Side-weirs are almost always preferred over automatic gates or siphons because they don't require periodic inspection and maintenance. Typically, weirs have a horizontal water surface profile and are relatively simple to design and analyze. They are commonly referred to as "normal" or "plane" weirs. However, when channel flows overtop a side-weir, they usually contain momentum in the direction of the channel's flow. This results in weir flows that are less than those of a normal weir, at the same head over the weir. That, along with the fact that the channel's discharge is being reduced in the downstream direction, results in a varying water surface profile. This varying discharge and water surface within the channel (and along the side-weir) are referred to as spatially-varied-flow.

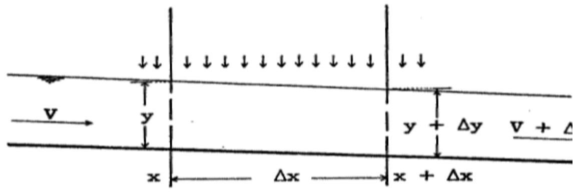
For supercritical flows in the main channel, the water surface curves down to the downstream end of the weir. Conversely, the water surface rises for subcritical flows. In this appendix, an overview of the spatially-varied-flow (SPV) phenomenon is presented along with the history of developing its equation and the side-weir's coefficient of discharge. Finally, recommendations are presented for the determination of the side-weir's flow.

SPATIALLY VARIED FLOW

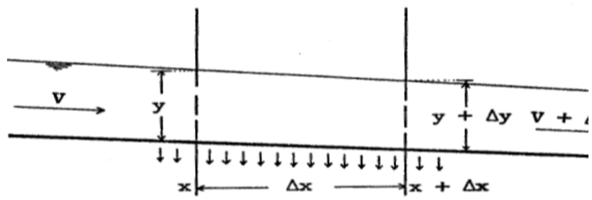
Spatially-varied-flow has a non-uniform discharge, velocity and water surface that results from the adding or decreasing of flow in a channel or water path. In contrast to continuous flow, the discharge varies with distance along the weir, i.e. $\frac{dQ}{dx} \neq 0$. This adding or decreasing, results in disturbances to the energy and momentum of the main channel's flow. Two types of (gradually) spatially-varied-flow exist.

1. Spatially-varied-flow with increasing discharge
2. Spatially-varied-flow with decreasing discharge

Weisz (1973) provided the figure below for increasing and decreasing discharges during spatially-varied-flow



a. Increasing Discharge



b. Decreasing Discharge

The following provides a brief description of each type:

INCREASING DISCHARGE

This type of flow usually occurs in situations such as side-channel spillways or when sheet flows enter a channel over its channel walls (sides). In this type, a significant amount of energy loss is caused by the mixing of the incoming flow with the main channel's flow. Since there doesn't exist any analytical mechanisms to determine the energy losses, the momentum equation is commonly used for such analyses.

Utilizing the momentum equation, the following equation results:

$$\frac{dh}{dx} = \frac{S_0 - S_f - \frac{2\alpha}{g}\frac{Q}{A^2}\frac{dQ}{dx}}{1 - \frac{\alpha}{g}\frac{Q^2}{A^2 d_h}}$$

DECREASING DISCHARGE

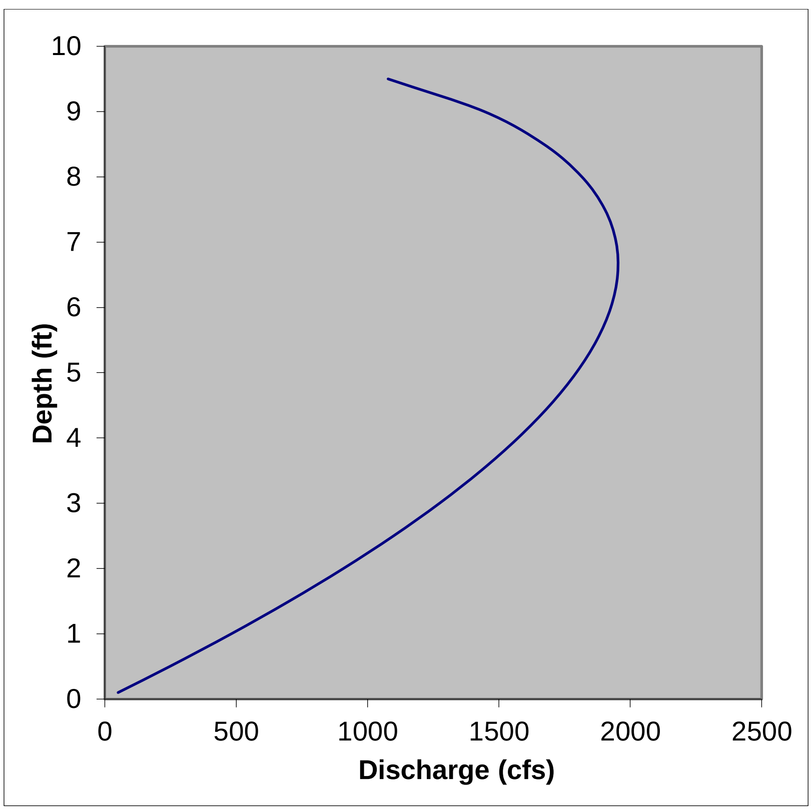
Commonly this type of spatially-varied-flow occurs at diversions where the main channel (or watercourse) releases flows into a side basin or conduit. In contrast to the case of increasing discharge, the diverted flows do not cause any changes to the main channel's energy head. Therefore, the energy equation is commonly used for such analyses. However, the momentum equation may also be used.

Utilizing either the energy or the momentum equation, the following equation results:

$$\frac{dh}{dx} = \frac{S_0 - S_f - \frac{\alpha}{g}\frac{Q}{A^2}\frac{dQ}{dx}}{1 - \frac{\alpha}{g}\frac{Q^2}{A^2 d_h}}$$

PROFILE TYPES IN SPATIALLY VARIED FLOW

Spatially-varied-flow water surfaces behave differently depending upon whether the flow is in the supercritical or subcritical regime. As will be discussed in the following sections, the specific energy along a weir remains relatively constant for the case of decreasing flows. Considering this, when the depth versus discharge is plotted for a specific channel geometry, it can be seen that the depth decreases (in the direction of flow) for the supercritical flow case. For the case of subcritical flow, it can be seen that the depth increases as the discharge decreases (flowing out of the channel and over the weir). The figure below exhibits these trends.



The figure below represents the three main types of water surface profiles that may occur at a side-weir as identified by De Marchi.

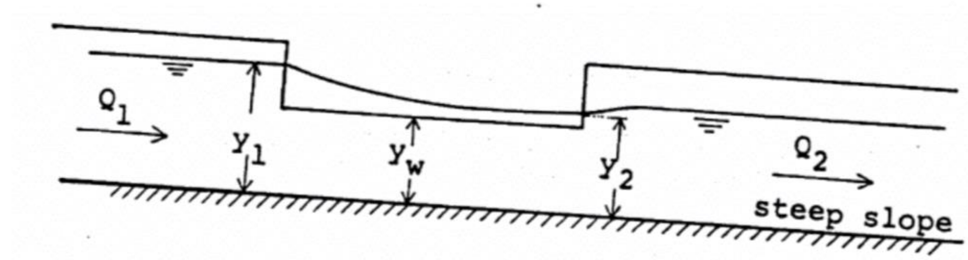


Figure 11

Case -a- of de Marchi (14)

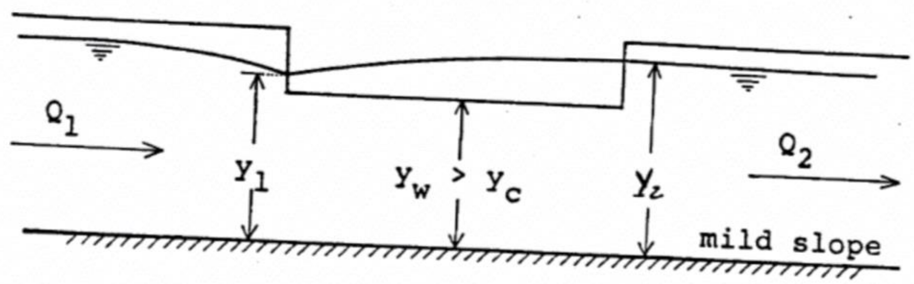


Figure 12

Case -b- of de Marchi (14)

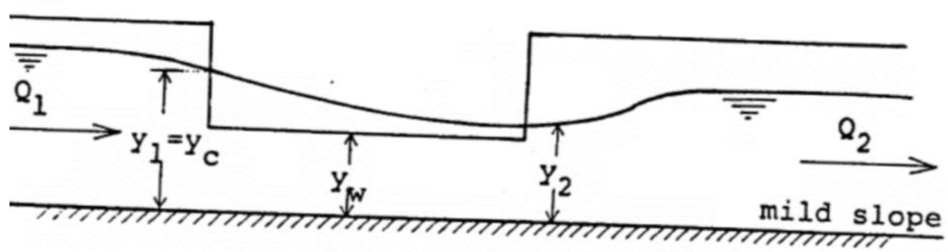


Figure 13

Case -c- of de Marchi (14)

The first case is that of a steep channel with supercritical flow occurring upstream, along and downstream of the side-weir. As can be seen, the water surface decreases as the flow in the channel flow decreases. The second case is that of a mild channel with subcritical flow upstream, downstream and along the side-weir. In contrast to case one, the water surface increases as the channel flow decreases. The third case is that of a mild channel with subcritical flow upstream and downstream of the side-weir. However, supercritical flow occurs within the weir region. What distinguishes this case from case two is the height of the weir. In this case, the height of the weir is well below the channel's critical depth, causing the flow depths to be below critical depths, i.e. supercritical

DERIVATION OF THE SPATIALLY-VARIED-FLOW EQUATION (Decreasing Discharge)

In the derivation and analyses of spatially-varied-flow, the following assumptions are usually (but not in all cases) made:

- The flow is one-dimensional. Although there are strong cross currents in the form of spiral flow, the effect of these currents is not easily determined but may be included if the momentum approach is followed.
- The velocity distribution in the main channel is constant and uniform.
- The slope of the main channel is small.
- The Manning formula is adequate for evaluating the main channel's frictional losses.
- The effect of air entrainment is negligible.
- Pressures in the channel are hydrostatic (approximately) despite some curvature in the water surface profile
- Conventional weir equation is assumed $q = \frac{dQ}{dx} = -\frac{2}{3} C_d \sqrt{2g}\,(h-w)^{1.5}$

Energy Approach

In general, this approach assumes that the longitudinal component of spill velocity is equal to the average channel velocity. However, some researchers such as Khashab (1976) observed that this was not true and that the longitudinal component of the spill velocity is higher than the average channel velocity. This approach also assumes that the total energy of the flow in the main channel should remain constant. By assuming that H = the total energy at a channel section along a weir and differentiating with respect to x, the following results:

$$H = z + h + \alpha \frac{V^2}{2g}$$

$$H = z + h + \alpha \frac{Q^2}{2A^2 g}$$

$$\frac{dH}{dx} = \frac{dz}{dx} + \frac{dh}{dx} + \frac{\alpha}{2g}\left(\frac{2Q}{A^2}\frac{dQ}{dx} - 2\frac{Q^2}{A^3}\frac{dA}{dx}\right)$$

since $S_f = -\frac{dH}{dx}$ and $S_o = -\frac{dz}{dx}$ and $\frac{dA}{dx} = T\frac{dh}{dx}$ for constant width of T

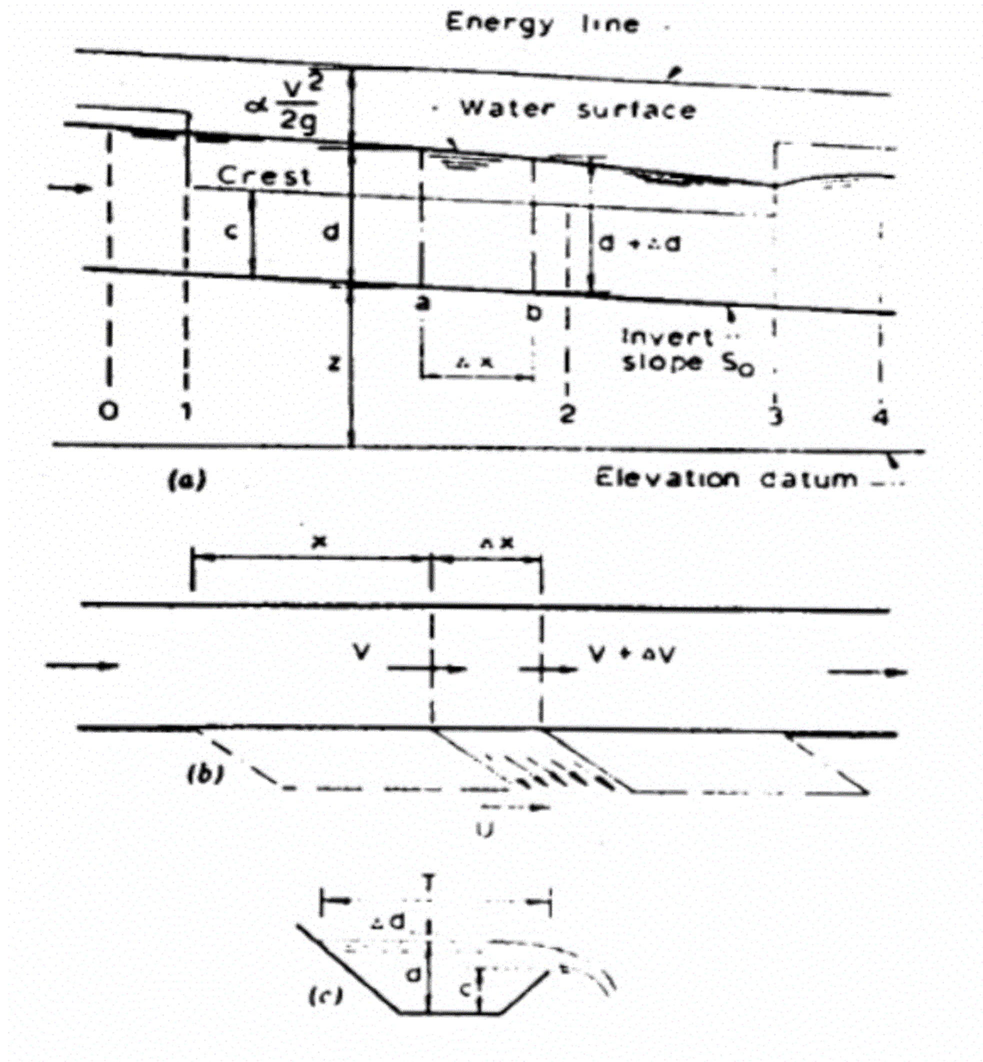
$$-S_f = -S_0 + \frac{dh}{dx} + \frac{\alpha}{g}\frac{Q}{A^2}\frac{dQ}{dx} - \frac{\alpha}{g}\frac{Q^2 T}{A^3}\frac{dh}{dx}$$

$$S_0 - S_f - \frac{\alpha}{g}\frac{Q}{A^2}\frac{dQ}{dx} = \frac{dh}{dx}\left(1 - \frac{\alpha}{g}\frac{Q^2 T}{A^3}\right)$$

$$\frac{dh}{dx} = \frac{S_0 - S_f - \frac{\alpha}{g}\frac{Q}{A^2}\frac{dQ}{dx}}{1 - \frac{\alpha}{g}\frac{Q^2}{A^3}T} \qquad \text{where} \qquad \frac{dQ}{dx} = -\frac{2}{3}C_d\sqrt{2g}\,(h-w)^{1.5}$$

Momentum Approach

For the derivation of the spatially-varied-flow equation using the momentum approach, Khashab's (1976) presented the following figure:



Khashab (1976) presented the momentum equation as

$M_a - M_b - M_w = P_a - P_b + W_f \sin\theta - F_f$ where M_w is the momentum departing over the weir.

$$\beta\rho(Q + \Delta Q)(V + \Delta V) - \rho\frac{dQ}{dx}\Delta x U - \beta\rho QV = P_a - P_b + W\sin\theta - F_f$$

for small values of Δx, $Pa - Pb = -\rho g A \Delta h$

and $W\sin\theta = \rho g A S_0 \Delta x$

and $F_f = \rho g A S_f \Delta x$

$$\beta\rho(QV+\Delta QV+Q\Delta V+\Delta Q\Delta V-QV)-\rho\frac{dQ}{dx}\Delta xU=-\rho gA\Delta h+\rho gAS_0\Delta x-\rho gAS_f\Delta x$$

dividing by ρ, deleting $\Delta Q\Delta V$ as insignificant

$$\beta(\Delta QV+Q\Delta V)-\frac{dQ}{dx}\Delta xU=-gA\Delta h+gAS_0\Delta x-gAS_f\Delta x$$

dividing by gA

$$\frac{\beta}{gA}(\Delta QV+Q\Delta V)-\frac{dQ}{dx}\frac{\Delta xU}{gA}=-\Delta h+S_0\Delta x-S_f\Delta x$$

since Q=VA

$$\frac{1}{g}\left[\left(\frac{\beta\Delta QV}{A}+\frac{\beta VA\Delta V}{A}\right)-\frac{dQ}{dx}\frac{\Delta xU}{A}\right]=-\Delta h+S_0\Delta x-S_f\Delta x$$

$$\frac{\beta}{g}V\Delta V+\frac{\beta}{g}\frac{V}{A}\Delta Q-\frac{dQ}{dx}\frac{\Delta xU}{gA}=-\Delta h+S_0\Delta x-S_f\Delta x$$

dividing by Δx

$$\frac{\beta}{g}\frac{V\Delta V}{\Delta x}+\frac{\beta}{g}\frac{V}{A}\frac{\Delta Q}{\Delta x}-\frac{dQ}{dx}\frac{U}{gA}=-\frac{\Delta h}{\Delta x}+S_0-S_f$$

letting Δx approach zero

$$\frac{dh}{dx}+\frac{\beta}{g}\frac{VdV}{dx}=S_0-S_f+\frac{U}{gA}\frac{dQ}{dx}-\frac{\beta}{g}\frac{V}{A}\frac{dQ}{dx}$$

developing the second term on the left-hand side

$$\frac{dQ}{dx}=\frac{d(VA)}{dx}=A\frac{dV}{dx}+V\frac{dA}{dx}$$

$$\frac{dV}{dx}=\frac{1}{A}\left[\frac{dQ}{dx}-V\frac{dA}{dx}\right]$$

$$\frac{V}{g}\frac{dV}{dx}=\frac{V}{gA}\left[\frac{dQ}{dx}-V\frac{dA}{dx}\right]$$

since $\frac{dA}{dx}=\frac{T}{}\frac{dd}{dx}$ for a prismatic channel and $V=\frac{Q}{A}$

$$\frac{V}{g}\frac{dV}{dx}=\frac{V}{gA}\left[\frac{dQ}{dx}-\frac{QT}{A}\frac{dh}{dx}\right]$$

or $\frac{\beta V}{g}\frac{dV}{dx}=\frac{\beta V}{gA}\frac{dQ}{dx}-\frac{\beta VQT}{gA^2}\frac{dh}{dx}$

$$\frac{\beta V}{g}\frac{dV}{dx} = \frac{\beta V}{gA}\frac{dQ}{dx} - \frac{\beta Q^2 T}{gA^3}\frac{dh}{dx}$$

placing this term in the equation above (that contains the S_0-S_f terms)

$$\frac{dh}{dx} + \frac{\beta V}{gA}\frac{dQ}{dx} - \frac{\beta Q^2 T}{gA^3}\frac{dh}{dx} = S_0 - S_f - \frac{\beta V}{gA}\frac{dQ}{dx} + \frac{U}{gA}\frac{dQ}{dx}$$

collecting terms

$$\frac{dh}{dx}\left(1 - \frac{\beta Q^2 T}{gA^3}\right) = S_0 - S_f - \frac{1}{gA}(2\beta V - U)\frac{dQ}{dx}$$

Finally

$$\frac{dh}{dx} = \frac{S_0 - S_f - \frac{1}{gA}(2\beta V - U)\frac{dQ}{dx}}{1 - \frac{\beta Q^2 T}{gA^3}}$$

This would be identical to the Energy Equation if U is taken equal to V and $\alpha = \beta = 1$

Note that the denominator may be written in terms of the Froude number since

$$Fr = \frac{V}{\sqrt{gd_h}} = \frac{V}{\sqrt{g\frac{A}{T}}}$$

$$F_r^{\,2} = \frac{V^2}{g\frac{A}{T}} = \frac{V^2 T}{gA} = \frac{Q^2 T}{gA^3}$$

$$\frac{dh}{dx} = \frac{S_0 - S_f - \frac{1}{gA}(2\beta V - U)\frac{dQ}{dx}}{1 - F_r^{\,2}}$$

Numerical Integration

Summing the forces acting on the channel/side-weir system

$$M_1 - M_2 - M_w = P_1 - P_2 + W_f \sin\theta - F_f$$

where

M_1 = momentum at the upstream bounding cross-section = $\frac{\gamma}{g}QV$

M_2 = momentum at the downstream bounding cross-section

$M_2 = \frac{\gamma}{g}(V - \Delta V)(Q - \Delta Q)$

M_3 = momentum associated with weir flow = $\frac{\gamma}{g}\Delta Q\left(V - \frac{\Delta V}{2}\right)$

P_1 = hydrostatic pressure force at upstream bounding cross-section

$P_1 = \gamma \bar{z} A$ where $\bar{z}$ is the depth to the centroid of area A below the water surface

P_2 = hydrostatic pressure force at downstream bounding cross-section

$P_2 = \gamma(\bar{z} + \Delta h)A + \frac{\gamma}{2}\Delta A \Delta h$ where Δh is the difference between the depths in section 1 and 2. Since the higher order differential $\Delta A \Delta h$ is negligible, it can be ignored

$P_2 = \gamma(\bar{z} + \Delta h)A$

$P_1 - P_2 = -\gamma A \Delta h$

$W =$ is the weight of the body of water between section 1 and 2

$W \sin\theta =$ the force in the direction of flow of the body of water between section 1 and 2

θ = the angle formed by the invert of the channel with the horizontal

$W \sin\theta = \gamma S_0 \left(A + \frac{1}{2}\Delta A\right)\Delta x = \gamma S_0 \bar{A} \Delta x$

similarly

$F_f = \gamma S_f \left(A + \frac{1}{2}\Delta A\right)\Delta x = \gamma S_f \bar{A} \Delta x$

where

$$S_f = \frac{V^2 n^2}{2.22 R^{4/3}} = \frac{Q^2 n^2}{2.22 A^2 R^{4/3}}$$

and S_f is the average of S_{f1} and S_{f2} (average friction slope)

$$\frac{\gamma}{g}QV - \frac{\gamma}{g}(V - \Delta V)(Q - \Delta Q) - \frac{\gamma \Delta Q}{g}\left(V - \frac{\Delta V}{2}\right) = -\gamma \bar{A} \Delta h + \gamma S_0 \bar{A} \Delta x - \gamma S_f \bar{A} \Delta x$$

$$\frac{\gamma}{g}\left[QV - (V - \Delta V)(Q - \Delta Q) - \Delta Q\left(V - \frac{\Delta V}{2}\right)\right] = -\gamma \bar{A} \Delta h + \gamma S_0 \bar{A} \Delta x - \gamma S_f \bar{A} \Delta x$$

$$\frac{\gamma}{g}\left[QV-\left(VQ-V\Delta Q-Q\Delta V+\Delta V\Delta Q\right)-V\Delta Q+\frac{\Delta V\Delta Q}{2}\right]=-\gamma\overline{A}\Delta h+\gamma S_0\overline{A}\Delta x-\gamma S_f\overline{A}\Delta x$$

$$\frac{\gamma}{g}\left[QV-QV+V\Delta Q+Q\Delta V-\Delta V\Delta Q-V\Delta Q+\frac{\Delta V\Delta Q}{2}\right]=-\gamma\overline{A}\Delta h+\gamma S_0\overline{A}\Delta x-\gamma S_f\overline{A}\Delta x$$

$$\frac{\gamma}{g}\left[Q\Delta V-\frac{\Delta V\Delta Q}{2}\right]=-\gamma\overline{A}\Delta h+\gamma S_0\overline{A}\Delta x-\gamma S_f\overline{A}\Delta x$$

$$\frac{\gamma\Delta V}{g}\left[Q-\frac{\Delta Q}{2}\right]=-\gamma\overline{A}\Delta h+\gamma S_0\overline{A}\Delta x-\gamma S_f\overline{A}\Delta x$$

since $Q=Q_1$ and by multiplying the left side by $\frac{Q_1}{Q_1}$

$$\frac{\gamma Q_1\Delta V}{g}\left[1-\frac{\Delta Q}{2Q_1}\right]=-\gamma\overline{A}\Delta h+\gamma S_0\overline{A}\Delta x-\gamma S_f\overline{A}\Delta x$$

by dividing both sides by $\overline{A}$ and γ

$$\frac{Q_1\Delta V}{g\overline{A}}\left[1-\frac{\Delta Q}{2Q_1}\right]=-\Delta h+S_0\Delta x-S_f\Delta x$$

since the discharge varies with the finite increment of channel length, the average area may be taken as $\overline{A}=\frac{(Q_1+Q_2)}{(V_1+V_2)}$

$$\Delta h=-\frac{Q_1(V_1+V_2)\Delta V}{g(Q_1+Q_2)}\left[1-\frac{\Delta Q}{2Q_1}\right]+S_0\Delta x-S_f\Delta x$$

and by introducing an energy coefficient due to non-uniform velocity distribution

$$\Delta h=-\frac{\alpha Q_1(V_1+V_2)\Delta V}{g(Q_1+Q_2)}\left[1-\frac{\Delta Q}{2Q_1}\right]+S_0\Delta x-S_f\Delta x$$

COEFFICIENT OF DISCHARGE

Unsubmerged-flow

The side-weir's coefficient of discharge has been the subject of much research in the past century. Despite all the efforts, exact determination of the discharge and general applicability remains the subject of continuous research. However, there has been some very impressive work accomplished at different periods with some being utilized for the design of numerous systems. Such is the work of Gamal Mostafa. Mostafa (1972, 1974 and 1987) conducted research and experimentation on side-weirs and arrived at relationships that were utilized for the design of numerous basins in Southern California. Another is Willi Hager (1987). The thoroughness of his research, experimentation and publications points, in this writer's opinion, to the state-of-the-art methodology for the analyses of side-weirs.

Submerged-flow

Once the retarding basin fills to a level greater than the side-weir's crest elevation, the weir flow nappe becomes submerged. The result is a decrease in the weir flow which may be accounted for by decreasing the weir's coefficient of discharge. Mostafa (1974) tested the effects of submergence on the weir's coefficient and arrived at the following equation:

$$\frac{C_{ds}}{C_d} = \left[1 - \left(\frac{H_b}{H}\right)^{3/2}\right]^{C4}$$

where C_{ds} is the coefficient of discharge for the submerged weir, C_d is the corresponding coefficient when the side-weir is not submerged, H is the water depth above the weir crest in the channel and H_b is the water depth above the weir crest in the basin, and C_4 is a constant ranging from 0.3 to 0.4. He states, "while H_b may be constant for a certain case, H is usually varying along the weir and therefore C_{ds}/C_d is expected to be also varying along the weir. However, an average value of H and C_4 may be used in applying the above mentioned equation without too much error." Chow (1959) references work by Villamonte in 1947 which resulted in a C_4 coefficient of 0.385 for a plane weir.

DEVELOPMENT OF THE SIDE-WEIR ANALYSIS

Research into available information on side-weirs revealed an abundance of theories and formulae dating back to the beginning of the 20th century. In general, the historical progression of this knowledge can be divided in several eras:

- Experimentation prior to utilization of the momentum and energy principles
- Experimentation and concepts utilizing the momentum and energy principles
- Development of relationships for the determination of a varying side-weir coefficient of discharge
- Automation of the design and analyses of side-weirs

1905 Parmley

Parmley determined the length of a weir based upon the head at the upstream end as well as the channel width and average velocity. He proposed the following equation:

$L = \left(\frac{bV}{1.67}\right)\left(4 - \sqrt{\frac{1}{h_1}}\right)$ where: h_1 is the height of water above the weir crest at its upper end and b is the channel width.

1917 Engels

Engels was the first to carry out experimental studies on side-weirs. Several size channels were utilized. However the velocities were on the order of 1.7 to 1.9 fps. Since the water surface profile tended to rise in the downstream direction, it was thought at that time that this phenomenon is characteristic to all flow regimes. His work resulted in the following equation for a prismatic channel:

$$Q_w = Q_1 - Q_2 = 3.32L^{0.82}\left(h_2 - w\right)^{1.67}$$

For a contracting channel the following equation was proposed:

$$Q_w = Q_1 - Q_2 = 3.32L^{0.90}\left(h_2 - w\right)^{1.60}$$

1923 Coleman and Smith

These researchers' work consisted of experiments on side-weirs in small size pipes. The following is known as the Coleman and Smith formula:

$L = 0.548 b V h_1^{0.13} \left(\sqrt{\frac{1}{h_2}} - \sqrt{\frac{1}{h_1}} \right)$ where: h_1 and h_2 are the upstream and downstream depths, respectively. b is the channel width and L is the side-weir length.

The water surface profiles resulting from this research appeared to contradict that of Engels. While Engels showed a rising (in the direction of flow) profile, Coleman and Smith showed a gradually dropping profile. This disagreement was later explained since Engels was modeling subcritical flows while Coleman and Smith modeled supercritical flows.

1926 Forchheimer

According to Tults (1956), Forchheimer considered the energy gradeline to be parallel to the spillway crest and the canal bottom. Although such an assumption is utilized occasionally in current theories, he proposed that the water surface along the weir is a straight line.

1926 Hinds

Hinds is credited with being the first to analyze spatially-varied-flow with the momentum principle for the case of increasing discharge (i.e. side-channel spillway). He paved the way for applying the momentum principle to the case of decreasing flows over a side-weir.

The spatially-varied-flow equation that Hinds derived is

$$\frac{dh}{dx} = \frac{S_0 - S_f - \frac{2\alpha Q}{gA^2}\frac{dQ}{dx}}{1 - \frac{\alpha Q^2}{gA^2 d_h}}$$

1928 Nimmo

Two years after Hinds applied the momentum principle to the case of increasing discharge, Nimmo utilized the same approach for the case of decreasing discharge. His approach summed the forces on a body of water in a channel along a side-weir. In that, he proposed that the change in momentum between two sections (in the main channel) along the weir minus the momentum lost due to the exiting flows over the weir should equal all the forces acting on this system. This very approach is the same utilized in current methodologies for analyzing a side-weir using the momentum approach. However, Nimmo assumed that the energy grade-line slope is parallel to the channel invert, i.e. $S_f = S_0$. The resulting equation is:

$$\frac{dh}{dx} = \frac{AQ}{TQ^2 - gA^3}\frac{dQ}{dx}$$

The other shortcoming of this approach was the assumption that the coefficient of discharge remains constant.

1934 De Marchi

He is credited with being the first to systematically analyze spatially-varied-flow over side-weirs based upon the energy conservation principle. The following assumptions were made in the course of developing de Marchi's equation:

- Conditions of steady flow exist,
- The weir is situated in an infinitely long channel of uniform flow cross-section,
- The weir is sharp-crested,
- The weir crest is parallel to the channel bed,
- At certain distances upstream and downstream of the side-weir, flow in the channel is uniform,
- The discharge over an increment length of the weir (dx) can be calculated from the classic weir formula $\Delta Q = -\frac{2}{3}C_d\sqrt{2g}(h-w)^{1.5}(\Delta x)$
- The total energy-line remains parallel to the bed of the channel.

De Marchi ends up in $$\frac{dh}{dx} = \frac{AQ}{TQ^2 - gA^3}\frac{dQ}{dx}$$

which is identical to the equation that Nimmo came up with using the momentum approach.

De Marchi deduced the following three possible profiles along a side-weir:

1. <u>Channel slope is steep</u>, producing supercritical flow upstream of the weir. Along the weir there is a gradual reduction in depth. Beyond the weir, the depth increases as the flow is retarded tending asymptotically to the normal depth of flow in the downstream channel
2. <u>Channel slope is mild (producing subcritical flow upstream of weir) and the depth of the weir is more than the critical depth</u>, thus maintaining a rising profile. The effect of the weir in this case will be felt in the upstream direction only, while the depth at the downstream end of the weir will be the normal depth for the discharge at that location. At the other end of the weir and beyond, the depth will tend asymptotically to the normal depth of that discharge.

3. Channel slope is mild but the weir depth is less than critical depth (at upstream) and the weir is long enough that the reduced water surface will lead to a depth less than critical depth within the weir region. This will cause the flow to become supercritical and thus lead to a falling profile.

The figure below presents the de Marchi's three possible profiles as shown in Mostafa (1972)

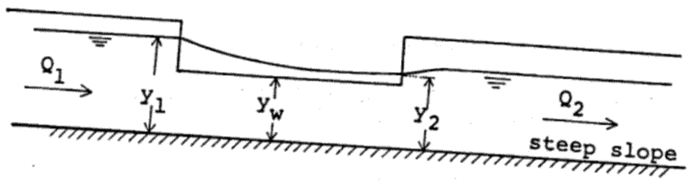


Figure 11

Case -a- of de Marchi (14)

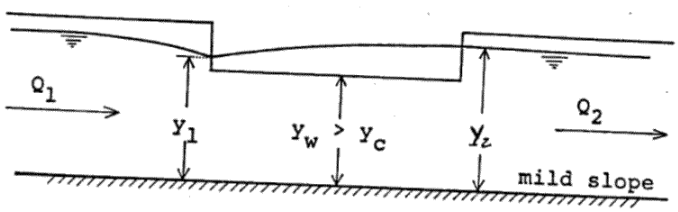


Figure 12

Case -b- of de Marchi (14)

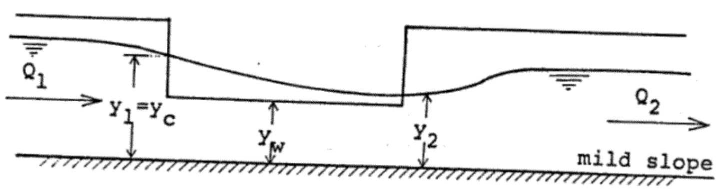


Figure 13

Case -c- of de Marchi

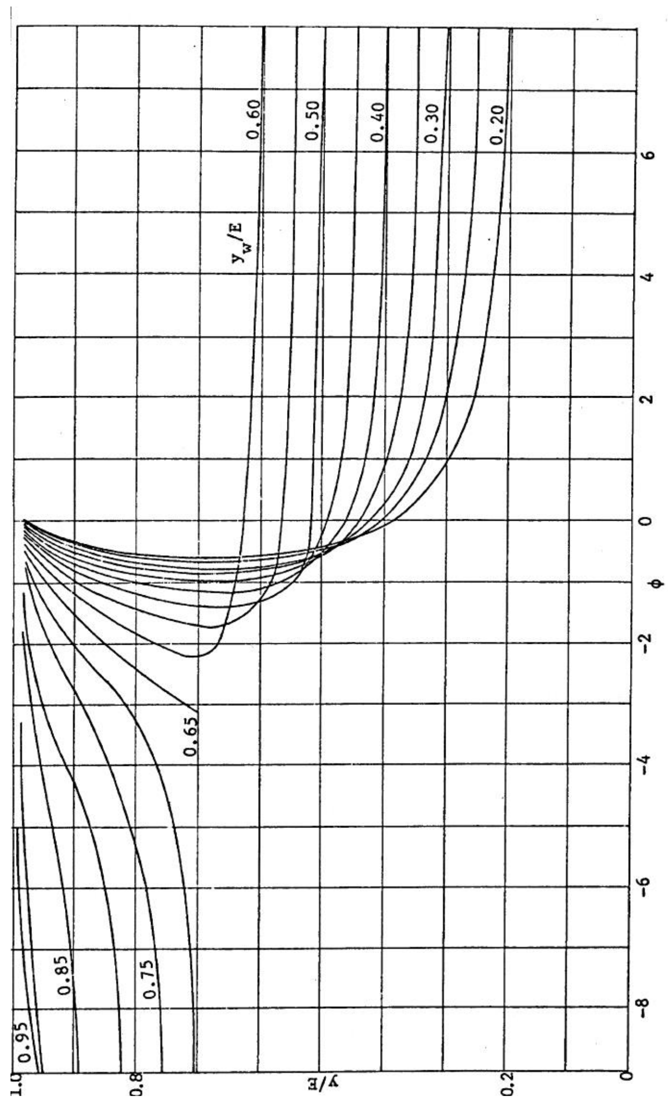
Finally de Marchi arrived at the following equation for a prismatic rectangular channel

$$L = x_2 - x_1 = \frac{3b}{2C_d}(\phi_2 - \phi_1)$$

$$\phi_1 = \frac{2E_1 - 3w}{E_1 - w}\sqrt{\frac{E_1 - h_1}{h_1 - w}} - 3\sin^{-1}\sqrt{\frac{E_1 - h_1}{E_1 - w}}$$

$$\phi_2 = \frac{2E_2 - 3w}{E_2 - w}\sqrt{\frac{E_2 - h_2}{h_2 - w}} - 3\sin^{-1}\sqrt{\frac{E_2 - h_2}{E_2 - w}}$$

However, according to Subramanya (1972) sufficient information on the variation of the weir coefficient used in his equation was not available. Therefore, they felt that there has been a lack of reliability in predicting the discharge with confidence. Below is a graph of De Marchi's function as presented by Weisz (1973).



1938 Gentilini

Mostafa (1972) writes, “Laboratory experiments by Gentilini confirmed de Marchi’s theory for the case of subcritical flow, but showed discrepancies between theory and physical model results for the supercritical flow conditions.”

1953 Babbit

According to Mostafa (1972), Babbit experimented on supercritical flows in circular pipes that were larger than those used in 1923 by Coleman and Smith. His equation has been popular in sanitary engineering and consists of:

$$L = 2.3V_1 D \log_{10}\left(\frac{h_1 - w}{h_2 - w}\right)$$ where: D is the pipe’s diameter.

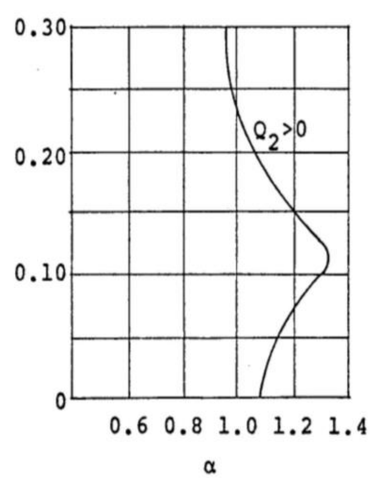
1956 Tults

Tults was one of the first researchers who derived the step procedure for analyzing spatially-varied-flow along a side-weir. Similar to Hinds (for increasing flows), Tults applied the momentum principle to the case of decreasing flows. By assuming that $S_f = S_0$ and $\alpha = 1$, he derived the following step equation:

$$\Delta h = \frac{\alpha Q_1}{g}\left(\frac{V_1 - V_2}{Q_1 - Q_2}\right)\Delta V\left(\frac{C(h-w)^{1.5}\Delta x}{2Q_1}\right)$$ where: $C = \frac{2}{3}C_d\sqrt{2g}$

As for the C coefficient, Tults concluded from the earlier work of Gentilini, Forchheimer and others, that the weir coefficient could approximate that of a normal weir reduced by 7 percent.

Finally, Tults proposed the figure below for the energy correction coefficient as a function of the relative depth over the side-weir.



1957 Collinge

Collinge confirmed De Marchi's solution for the case of subcritical flow. Mandatory to this conclusion is that the flow depth must be above critical depth at the upstream end of the weir. Otherwise, if the flow depth is below critical depth, supercritical flows occur and would result in a falling profile. Both, Mostafa (1972) and Subramanya (1972), conclude from the Collinge study that C_d varies with mean velocity of flow in the upstream channel. For the case of a free nappe, the reduction ratio (in percent) in the weir coefficient (from that of normal weir) is $3.7\overline{V}$ for subcritical flow while $0.3\overline{V}$ for supercritical flow. In this case, $\overline{V}$ = the average channel velocity.

He developed the following equation:

$$Q_w = LC_d\sqrt{2g}(h-w)^{1.5}$$

1957 Ackers

According to Subramanya (1972), Ackers suggested that C_d=0.625 if h is measured at a remote distance from the plane of the weir and C_d =0.725 if measured in the plane of the weir. He apparently assumed that C_d is constant for various flow situations. Mostafa (1972) opined that Acker's theory for the length of a side-weir is the same as that proposed by De Marchi twenty three years earlier, except for the inclusion of pressure and energy correction factors. For example, Ackers suggested the use of $\alpha = 1.4$. The resulting Acker's equation is identical to De Marchi's

$L = \dfrac{3b}{2\alpha C_d}(\phi_2 - \phi_1)$ where ϕ is defined earlier in the De Marchi discussion.

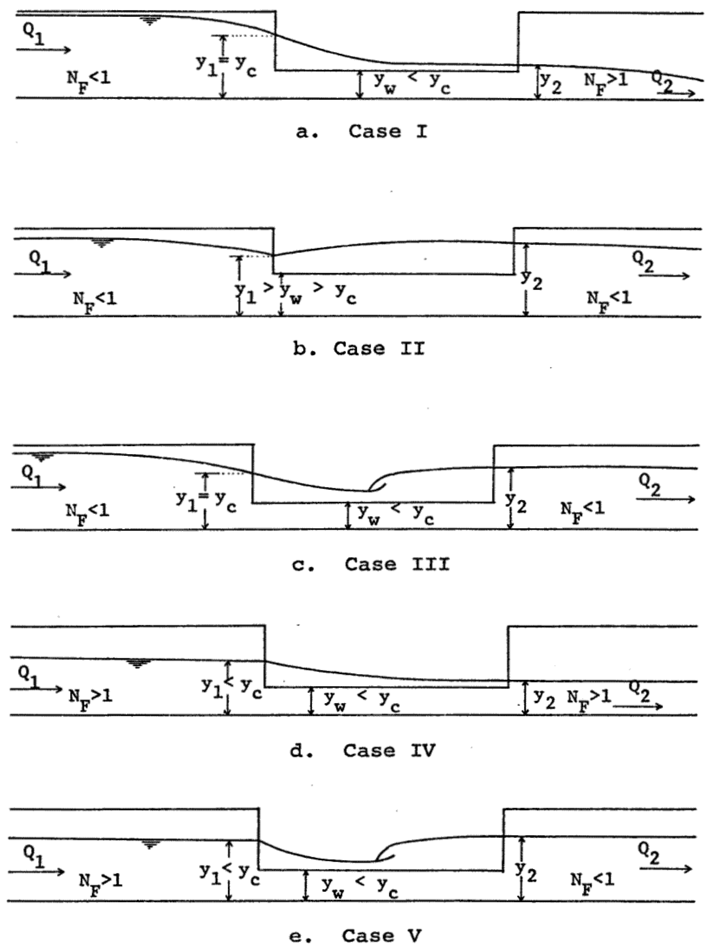
1957 Frazer

Although Frazer experimented on the three main possible profiles (subcritical, supercritical and combined with a jump), he presented five cases of weir flow.

- **Case I** - Critical conditions at or near the entry with supercritical flow in the weir section, the depth of flow decreasing along the weir section. This case is similar to De Marchi's Case 3.
- **Case II** - Depth of flow greater than critical at the entry with subcritical flow in the weir section, the depth of flow increasing along the weir section. This case is the same as de Marchi's Case 2
- **Case III** - Subcritical flow in the upstream channel going to critical flow at the beginning of the weir section with a hydraulic jump occurring in the weir section and Case II type of flow after the jump at a lower specific energy level owing to jump losses.

- **Case IV** – Depth of flow less than critical at the entry with supercritical flow in the weir section, the depth of flow decreasing along the weir section. This case is similar to de Marchi's Case 1.
- **Case V** – Supercritical flow in the upstream channel with normal depth at the beginning of the weir section with a hydraulic jump occurring in the weir section and Case II flow after the jump at a lower specific energy level owing to jump losses.

The following figure shows the five cases as presented by Weisz (1973)



a. Case I

b. Case II

c. Case III

d. Case IV

e. Case V

He concluded that the coefficient C in $Q_w = CL(h-w)^{1.5}$ is

$C = 4.15 - 1.81\left(\dfrac{h_c}{h_1}\right) - 0.812\dfrac{h_c}{L}$ where: y in the figure is the same as h in the equation.

1959 Chow

Chow analyzed spatially-varied-flow for both, increasing as well as decreasing discharges. He found that the energy principle is adequate for analyzing the case of decreasing discharge since there isn't an appreciable loss of energy in the main channel due to the weir flow. However, for the case of increasing flows, an appreciable loss of internal energy occurs which requires the use of the momentum principle. Chow noted that one can arrive at the same formula for decreasing discharge by using either principle. Similarly, the same equation can be arrived at for increasing flow by using either principle.

Of note is that Chow utilized the α (energy correction factor) for the momentum equation. His reasoning is based upon the use of the Manning's equation for the evaluation of frictional energy losses.

A numerical integration procedure was presented in Chow's text for the case of increasing flow. The same principles arrived at for this procedure may be modified to be used for the case of decreasing discharges. He acknowledged that the weir flow term in the equations derived has been thoroughly researched by others and many equations have been proposed. However, he states "For practical purposes, the formula for the regular weir of similar crest shape may be used if the corresponding discharge coefficient is reduced by 5%."

1967 Coulomb

Coulomb is credited for being the first to perform systematic studies for the lateral outflow pattern. He determined a correction for the lateral velocity over the side-weir by using the *sin* correction factor. Therefore, the longitudinal component of the overflow velocity, U, would need to by multiplied by the sin of the angle that the side-weir flow makes with the plane of the weir. For a normal weir, where the angle is 90 degrees, the correction is 1. The following equation includes the correction to the side-weir flow from that of a normal weir. ***h*** in the equation is the depth while ***w*** is the height of the weir.

$\sin\phi = \left(\frac{1-W}{y-W}\right)\left[\frac{3y-2-W}{y-W}\right]^{0.5}$ with *w* only defined if *y* > = (2+*W*)/3

Where: $W = \frac{w}{H}$ and $y = \frac{h}{H}$

Hager in 1987 arrived at the same equation but by a different means. As can be seen, the effects of contraction (θ) and invert slope are ignored. The following equation defines the lateral outflow intensity as presented by Coulomb:

$$Q' = -\left(\frac{2}{3}\right)^{1.5}\sqrt{gH^3}\,(1-W)\sqrt{3y-2-W}$$

1972 Subramanya and Awasthy

These researchers are credited for being the first to perform systematic investigations of the internal flow characteristics of a weir with zero height. They applied the Bernoulli equation on a streamline leaving the channel laterally and determined the outflow direction and discharge correction coefficient in terms of the approaching Froude No. Their emphasis was on defining the variation of the De Marchi coefficient. They proposed that the De Marchi equation can be used to predict the discharge over a side-weir by knowing the variation of the coefficient C_d. The following equations represent the variation of C_d with F_1:

$$C_d = 0.611\sqrt{1-\left(\frac{3F_1^2}{F_1^2+2}\right)} \qquad \text{Subcritical}$$

$$C_d = 0.36 - 0.08F_1 \qquad \text{Supercritical}$$

They claimed that the discharge coefficient C_d of a side-weir of finite height is essentially the same as that of a side-weir of zero height. In all, the De Marchi equation for spatially-varied-flow was modified for rectangular, prismatic, frictionless and horizontal channel. As can be seen, they ignore the effect of the weir height/depth of flow.

1972, 1974 and 1987 MOSTAFA

Mostafa in two investigations for the Orange County Flood Control District in 1972 and 1974 summarized his research and physical modeling results then arrived at relationships for defining the side-weir's coefficient of discharge. Similar to the derivation of the spatially-varied-flow equation earlier in this appendix, Mostafa carried the derivation further to include non-prismatic trapezoidal sections.

$$\beta\rho(Q+\Delta Q)(V+\Delta V) - \rho\frac{dQ}{dx}\Delta xU - \beta\rho QV = P_a - P_b + W\sin\theta - F_f$$

Assumes β corresponding to U is equal to 1.0

For small values of Δx, $Pa - Pb = -\rho gA\Delta h$

And $W\sin\theta = \rho gAS_0\Delta x$

And $F_f = \rho gAS_f\Delta x$

$$\beta\rho(QV+\Delta QV+Q\Delta V+\Delta Q\Delta V-QV) - \rho\frac{dQ}{dx}\Delta xU = -\rho gA\Delta h + \rho gAS_0\Delta x - \rho gAS_f\Delta x$$

dividing by ρ, deleting $\Delta Q\Delta V$ as insignificant

$$\beta(\Delta QV + Q\Delta V) - \frac{dQ}{dx}\Delta x U = -gA\Delta h + gAS_0\Delta x - gAS_f\Delta x$$

dividing by gA

$$\frac{\beta}{gA}(\Delta QV + Q\Delta V) - \frac{dQ}{dx}\frac{\Delta x U}{gA} = -\Delta h + S_0\Delta x - S_f\Delta x$$

since Q=VA

$$\frac{1}{g}\left[\left(\frac{\beta\Delta QV}{A} + \frac{\beta VA\Delta V}{A}\right) - \frac{dQ}{dx}\frac{\Delta x U}{A}\right] = -\Delta h + S_0\Delta x - S_f\Delta x$$

$$\frac{\beta}{g}V\Delta V + \frac{\beta}{g}\frac{V}{A}\Delta Q - \frac{dQ}{dx}\frac{\Delta x U}{gA} = -\Delta h + S_0\Delta x - S_f\Delta x$$

dividing by Δx

$$\frac{\beta}{g}\frac{V\Delta V}{\Delta x} + \frac{\beta}{g}\frac{V}{A}\frac{\Delta Q}{\Delta x} - \frac{dQ}{dx}\frac{U}{gA} = -\frac{\Delta h}{\Delta x} + S_0 - S_f$$

letting Δx approach zero

$$\frac{V}{g}\frac{dV}{dx} = \frac{V}{gA}\left[\frac{dQ}{dx} - V\frac{dA}{dx}\right] \qquad V = \frac{Q}{A}$$

$$\frac{V}{g}\frac{dV}{dx} = \frac{Q}{gA^2}\left[\frac{dQ}{dx} - \frac{Q}{A}\frac{dA}{dx}\right] \qquad A = bh + zh^2$$

$$\frac{dA}{dx} = b\frac{dh}{dx} + h\frac{db}{dx} + 2zh\frac{dh}{dx} + h^2\frac{dz}{dx}$$

$$\frac{dA}{dx} = \frac{dh}{dx}\left(b + 2zh)\right) + h\frac{db}{dx} + h^2\frac{dz}{dx} \qquad V = \frac{Q}{A}$$

$$\frac{dV}{dx} = -QA^{-2}\frac{dA}{dx} + \frac{1}{A}\frac{dQ}{dx} = \frac{1}{A}\frac{dQ}{dx} - \frac{Q}{A^2}\frac{dA}{dx}$$

$$\frac{dV}{dx} = \frac{1}{A}\frac{dQ}{dx} - \frac{Q}{A^2}\left[\frac{dh}{dx}\left(b + 2zh) + h\frac{db}{dx} + h^2\frac{dz}{dx}\right)\right]$$

$$\frac{dh}{dx} + \frac{\beta V}{g}\frac{dV}{dx} = S_0 - S_f + \frac{U}{gA}\frac{dQ}{dx} - \frac{\beta}{g}\frac{V}{A}\frac{dQ}{dx}$$

$$\frac{dh}{dx}+\frac{\beta V}{g}\left[\frac{1}{A}\frac{dQ}{dx}-\frac{Q}{A^2}\frac{(b+2zh)}{}\frac{dh}{dx}-\frac{Qh}{A^2}\frac{db}{dx}-\frac{Qh^2}{A^2}\frac{dz}{dx}\right]=S_o-S_f+\frac{U}{gA}\frac{dQ}{dx}-\frac{\beta}{g}\frac{V}{A}\frac{dQ}{dx}$$

$$\frac{dh}{dx}+\frac{\beta Q}{gA}\left[\frac{1}{A}\frac{dQ}{dx}-\frac{Q}{A^2}\frac{(b+2zh)}{}\frac{dh}{dx}-\frac{Qh}{A^2}\frac{db}{dx}-\frac{Qh^2}{A^2}\frac{dz}{dx}\right]=S_o-S_f+\frac{U}{gA}\frac{dQ}{dx}-\frac{\beta}{g}\frac{Q}{A^2}\frac{dQ}{dx}$$

$$\frac{dh}{dx}+\frac{\beta Q}{gA^2}\frac{dQ}{dx}-\frac{\beta}{g}\frac{Q^2}{A^3}\frac{(b+2zh)}{}\frac{dh}{dx}-\frac{\beta}{g}\frac{Q^2h}{A^3}\frac{db}{dx}-\frac{\beta}{g}\frac{Q^2h^2}{A^3}\frac{dz}{dx}=S_0-S_f+\frac{U}{gA}\frac{dQ}{dx}-\frac{\beta}{g}\frac{Q}{A^2}\frac{dQ}{dx}$$

$$\frac{dh}{dx}\left[1-\frac{\beta(b+2zh)}{g}\frac{Q^2}{A^3}\right]=S_0-S_f+\left[\frac{U}{gA}-\frac{\beta Q}{gA^2}-\frac{\beta}{g}\frac{Q}{A^2}\right]\frac{dQ}{dx}+\frac{\beta}{g}\frac{Q^2h}{A^3}\frac{db}{dx}+\frac{\beta}{g}\frac{Q^2h^2}{A^3}\frac{dz}{dx}$$

$$\frac{dh}{dx}\left[1-\frac{\beta(b+2zh)}{g}\frac{Q^2}{A^3}\right]=S_0-S_f-\left[\frac{2\beta Q}{gA^2}-\frac{U}{gA}\right]\frac{dQ}{dx}+\frac{\beta}{g}\frac{Q^2h}{A^3}\frac{db}{dx}+\frac{\beta}{g}\frac{Q^2h^2}{A^3}\frac{dz}{dx}$$

$$\frac{dh}{dx}\left[1-\frac{\beta(b+2zh)}{g}\frac{Q^2}{A^3}\right]=S_0-S_f-\left[\frac{2\beta V-U}{gA}\right]\frac{dQ}{dx}+\frac{\beta}{g}\frac{Q^2h}{A^3}\frac{db}{dx}+\frac{\beta}{g}\frac{Q^2h^2}{A^3}\frac{dz}{dx}$$

$$\frac{dh}{dx}\left[1-\frac{\beta(b+2zh)}{g}\frac{Q^2}{A^3}\right]=S_0-S_f-\left[\frac{2\beta V-U}{gA}\right]\frac{dQ}{dx}+\frac{\beta}{g}\frac{Q^2}{A^3}\left(\frac{h}{}\frac{db}{dx}+\frac{h^2}{}\frac{dz}{dx}\right)$$

$$\frac{dh}{dx}=\frac{\left(S_0-S_f-\left[\frac{2\beta V-U}{gA}\right]\frac{dQ}{dx}+\frac{\beta}{g}\frac{Q^2}{A^3}\left(\frac{h}{}\frac{db}{dx}+\frac{h^2}{}\frac{dz}{dx}\right)\right)}{\left[1-\frac{\beta}{g}\frac{Q^2}{A^3}\left(b+2zh\right)\right]}$$

Finally, Mostafa (1972) arrived at the following expression for the arrival at a coefficient of discharge

$C_d=0.64\left(\frac{Q_w}{Q_1}\right)^{0.206}$ where Q_w is the weir discharge and Q_1 is the discharge at the upstream end of the weir. Although Mostafa experimented with the variation of the coefficient as a function of the depth above the weir to the total water depth $\frac{(h-w)}{h}$, that work was temporarily abandoned. Several years later, Mostafa (1987) published an additional correction to the coefficient equation which includes this term.

$$C_d=C'+0.259C'\frac{(h-w)^2}{h^2}$$

where $C'=0.60\left(\frac{Q_w}{Q_1}\right)^{0.167}$

1972 Nadesamoorthy and Thompson

According to Hager (1987), these researchers arrive at the following expression for the coefficient of discharge

$$C_d = 0.407\left\{\frac{(2+F_1^{\,2})}{2(1+2F_1^{\,2})}\right\}^{0.5}$$

where F_1 is the approaching Froude number. Hager opined that their approach overlooks local flow patterns and relates only to a particular weir configuration.

1976 El-khashab and Smith

According to their paper, a complete analytical solution of the equations governing the flow in the side-weir and channel is not possible. Until that time, approximate methods were used based upon experiments conducted over a limited range of the many variables involved. Alternatively, restricted analytical solutions have been available such as that of De Marchi (1934), but direct application to practical cases was not generally possible and their use was considered tedious. Smith in 1973, developed a step procedure for side-weir analyses based upon the traditional assumption of constant energy in the channel (except for energy loss due to friction).

Khashab, for his doctorate dissertation, carried out an investigation on a reasonably large-scale model (Q=7.5 cfs, Weir length=7.5') and then detailed velocity plots along the main channel. After allowing for friction losses, it was apparent that there was a substantial fall in the total energy line. This was observed for both super and subcritical flows. At first, it was thought that this was due to neglect of energy contained in the secondary currents. Subsequent measurements of the magnitude of secondary currents showed that these only correspond to a small amount of energy, not comparable with the drop in the total energy line. The constant energy assumption, which forms the basis of conventional side-weir analyses, implies that at any section, the channel's average velocity is equal to the average weir velocity at that section.

Their experiments showed otherwise. The longitudinal component of the spill flow velocity was higher than the channel's average velocity. Because of such a difficulty with the energy approach, the momentum approach was adopted.

It was seen that the thread of maximum velocity, which is more or less central upstream of the weir, moves across to the top of the weir, in other words the high velocity flow is skimmed off over the weir.

During the course of the work, a fairly clear break emerged for subcritical flows, depending on whether the value of the spill discharge ratio $\frac{Q_w}{Q_1}$ is less than or greater than 0.5

Where $\frac{Q_w}{Q_1} < 0.5$ a considerable part of the approach flow remains in the channel and there is a strong forward velocity, which has a dominant effect on flow conditions. For this category, U could be related to the mean velocity at the same cross section and a satisfactory relationship was found between (see Khashab's figure below) $\frac{U}{V}$ and $\frac{h}{w}$, with ***h*** being the depth of flow at any point and ***w*** the weir height.

When $\frac{h}{w} = 1$, $U = V$

When $\frac{h}{w} = 1.5$, $\frac{U}{V} = 1.4$

Where $\frac{Q_w}{Q_1} > 0.5$ there is a large amount of spill as the flow over the weir could be visualized as a jet originating in the upstream channel and rolling over the side-weir. Accordingly, the longitudinal component of the spill flow at any section, U, was much more closely related to V_1 at the beginning of the weir. Starting with $U = V_1$. and progressing along the weir, the ratio $\frac{U}{V_1}$ decreased as the height of water above the weir crest increased. Nevertheless, U always remained substantially larger than V at the same section.

When $\frac{(h-w)}{(h_1-w)} = 1$ then $\frac{V_1}{U} = 1$

When $\frac{(h-w)}{(h_1-w)} = 1.5$ then $\frac{V_1}{U} = 1.45$

Finally, for supercritical flow Khashab postulated that the ratio of U to V increased with the intensity of flow at the beginning of the weir. He presented his figure no. 7 (shown below) where he correlates U/V in terms of F_1.

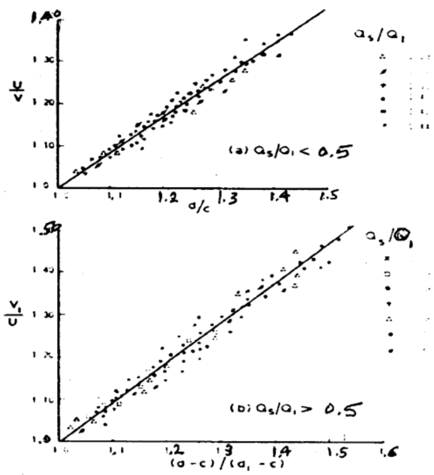


FIG. 6.—Relationships for Spill Flow Forward Velocity, Subcritical Flows

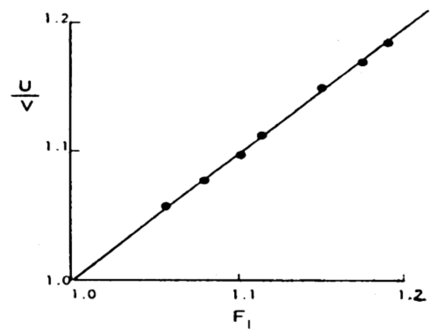


FIG. 7.—Relationship for Spill Flow Forward Velocity, Supercritical Flows

In this wirters opinion, these researchers advanced the understanding of weir flow. However, missing was an absence of discussion on the variation of the side-weir's coefficient of discharge.

1979 Ranga Raju, Prasad, and Gupta

These researchers concentrated on defining the weir coefficient for a weir connected to a side channel at 90 degrees to the main channel flow. They base their computational scheme on the de Marchi free surface profile, and adjust the discharge coefficient C_d in terms of the inflow Froude No.

From De Marchi

$$L = \frac{3}{2}\frac{B}{C_d}(\phi_2 - \phi_1)$$

in which B is the width of the main channel and

$$\phi_1 = \frac{2E_1 - 3w}{E_1 - w}\sqrt{\frac{E_1 - h_1}{h_1 - w}} - 3\sin^{-1}\sqrt{\frac{E_1 - h_1}{E_1 - w}}$$

$$\phi_2 = \frac{2E_2 - 3w}{E_2 - w}\sqrt{\frac{E_2 - h_2}{h_2 - w}} - 3\sin^{-1}\sqrt{\frac{E_2 - h_2}{E_2 - w}}$$

Their studies showed the effect of the separation of flow at the upstream end of the weir. They concluded that the full width of the weir may not be effective because of the pronounced separation. In contrast to earlier work by Khashab (1976), they agreed with De Marchi's assumption of constant specific energy since their experiments showed only a drop of 2 percent in specific energy along the weir.

The De Marchi equation was slightly modified to account for the reduction in effective width

$C_d = \frac{3}{2}\frac{B}{B_e}(\phi_2 - \phi_1)$ where B_e is the effective width of the side-weir

For their experiments, they observed that the $B_e = B - 0.05$m. However, they also noted that if a well rounded corner (at the upstream end of the weir) is included, B may well equal B_e. Their work concentrated on securing a weir coefficient in terms of the upstream Froude number for a subcritical weir situation. They arrived at the following equation

$$C_d = 0.81 - 0.6F_1$$

As for broad crested weirs, the writers concluded that at large values of head to crest length ratio, the weir eventually behaves as a sharp crested weir.

The adjustment for the broad crested weirs is

$$C_d = (0.81 - 0.6F_1)K$$

K = Empirical coefficient as presented in Raju's graph below

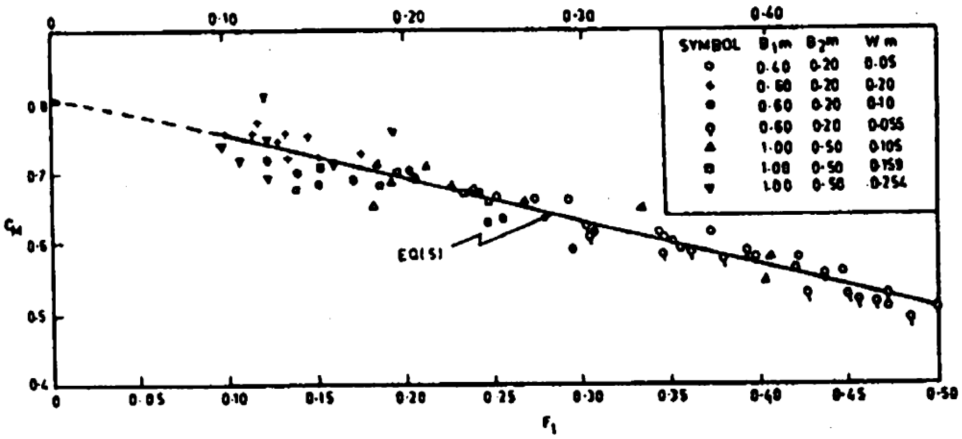


FIG. 5.—Variation of C_M with F_1 for Sharp-Crested Weirs

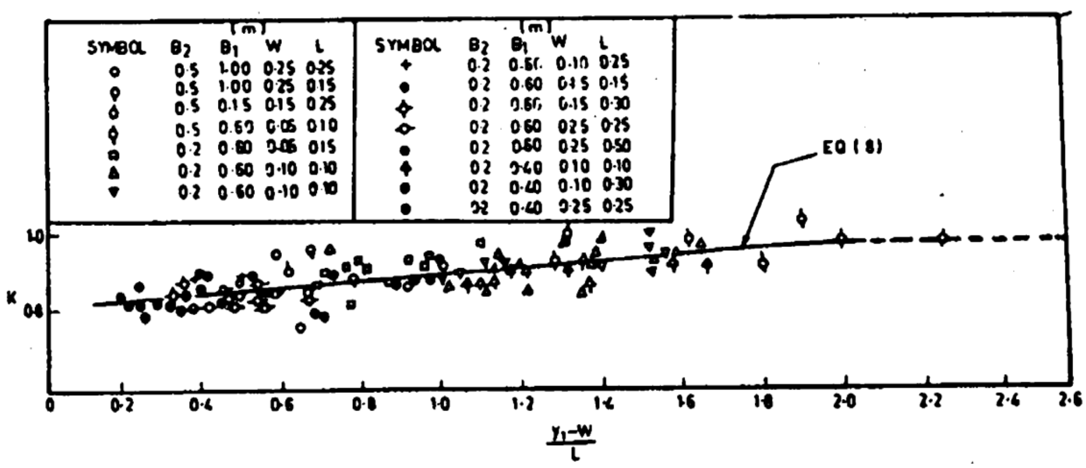


FIG. 6.—Variation of K with $(y_1 - W)/L$ for Broad-Crested Weirs

It was seen that K tends towards unity at $\frac{(h_1 - w)}{L} \geq 2.0$ where L is the width of the crest.

Finally, for K < 2.0 $\quad C_d = (0.81 - 0.6F_1)\left(0.8 + 0.1\frac{h_1 - w}{L}\right)$

As can be seen, they ignore the effect of the weir height/depth of flow.

1987 Hager

Using the energy approach, Hager investigated a one-dimensional approach considering the effects of flow depth, approaching velocity, lateral outflow direction and channel slope. His work resulted in one-dimensional expressions for the lateral outflow angle and lateral discharge intensity. He showed that the plane lateral outflow over a weir is a particular case of the side-weir flow. Compared to plane-weir flow, streamlines over a side-weir deviate from the channel axis by the angle ϕ. Below is a summary discussion of Hager's findings.

Hager's approach

Hager's $Q' = \frac{dQ}{dx} < 0$ is the lateral outflow per unit length in which x is the longitudinal coordinate. He then expressed Q'=***jq*** where **q** corresponds to the lateral outflow for plane-weir conditions and ***j*** corresponds to a correction due to the lateral outflow.

With U as the local lateral outflow velocity, the lateral outflow intensity is $dQ = -U \sin \phi h dx$ Note that Hager's U differs from that used by Khashab (1976) in that the latter is the longitudinal velocity component of the side-weir flow.

The lateral flow coefficient, ***j***, includes the effects of lateral velocity (U), lateral angle (ϕ) and lateral outflow depth (h). For a plane-weir (ϕ = 90), ***j*** = 1.

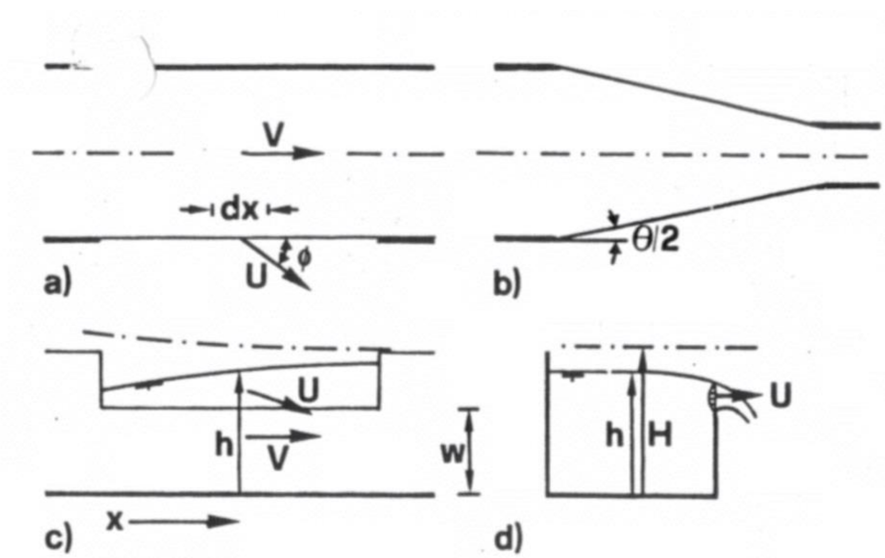


FIG. 2.—Lateral Outflow Geometry, Plan View; (*a*) Prismatic; (*b*) Non-Prismatic Side Weir; (*c*) Longitudinal; and (*d*) Transversal Sections

Effect of flow depth

For the plane-weir flow, Hager wrote the weir flow equation as $q = C_d \sqrt{2g}(h-w)^{1.5}$ where he considered the depth h at the side-weir equal to the energy head of a plane-weir (h = H). Therefore he changed the equation to be in terms of h rather than H.

Effect of Velocity of approach

The outflow velocity U for a plane-weir is $U_1 = \sqrt{2g(H_1 - w)}$ and for a side-weir $U_2 = \sqrt{2g(H_2 - w)}$ with H_1 approaching h and H_2 approaching H

Then the effect of approaching velocity is $j_u = \dfrac{U_2}{U_1} = \left[\dfrac{(H-w)}{(h-w)}\right]^{1.5}$

As can be seen, for plane flow conditions (H=h), $j_u = 1.0$

However, $j_u > 1$ for side-weir flow since H>h . As a consequence, the velocity of approach actually increases the lateral flow intensity Q'.

Effect of Lateral Outflow Angle

Since $dQ = -U \sin\phi \bar{h} dx$ therefore $j_\phi = \sin\phi$. The maximum of j_ϕ occurs at an angle of 90 degrees (plane-weir flow).

Assuming uniform velocity distribution, the average channel velocity would equal the axial component of the lateral outflow velocity $V = U\cos\phi$. The latter assumption is contrary to El-Khashab's (1976) conclusions that the longitudinal component of the side-weir velocity exceeds that of the main channel.

Since $\cos\phi = (1-\sin^2\phi)^{.5}$ then

$$V = U(1-\sin^2\phi)^{.5}$$

$$\left(\frac{V}{U}\right)^2 = 1-\sin^2\phi$$

$$\sin^2\phi = 1-\left(\frac{V}{U}\right)^2$$

$$\sin\phi = \left[1-\left(\frac{V}{U}\right)^2\right]^{0.5}$$

$$\sin\phi = j_\phi$$

$$j_\phi = \left[1 - \left(\frac{V}{U}\right)^2\right]^{0.5}$$

by assuming hydrostatic pressure and uniform velocity distribution in his writing of the energy equation, from the channel to the weir section, the following expression results

$$H = h + \frac{V^2}{2g} = w + \rho(h - w) + \frac{U^2}{2g}$$

and $y = \frac{h}{H}, W = \frac{w}{H} and, \rho.$= residual pressure coefficient

writing V as a function of h and H results in

$$V^2 = (H - h)2g = \frac{(H - h)}{H}2gH = (1 - y)2gH$$

writing U as a function of h and H results in

$$U^2 = [H - w - \rho(h - w)]2g = \left[\frac{H - w - \rho(h - w)}{H}\right]2gH$$

$$U^2 = [1 - W - \rho(y - W)]2gH$$

Now we have expressions for $U^2 and V^2$

since $\sin\phi = \left[1 - \left(\frac{V}{U}\right)^2\right]^{0.5}$

then $\sin\phi = \left[1 - \left(\frac{(1 - y)2gH}{[1 - W - \rho(y - W)]2gH}\right)\right]^{0.5}$

eliminating *2gH* results in $\sin\phi = \left[\left(\frac{1 - W - \rho(y - W) - 1 + y}{[1 - W - \rho(y - W)]}\right)\right]^{0.5}$

further consolidating results in

$\sin\phi = \left[\left(\frac{(y - W)(1 - \rho)}{[1 - \rho y - W(1 - \rho)]}\right)\right]^{0.5}$ and since $\rho = 2/3$ (where ρ is the flow depth at the weir, i.e. $\frac{2}{3}(h - w)$

$$\sin\phi = \left[\left(\frac{(y-W)(1/3}{\left[1-2/3\,y-W(1/3\right]}\right)\right]^{0.5}$$

$$\sin\phi = \left[\left(\frac{(y-W)}{\left[3-2y-W\right]}\right)\right]^{0.5}$$

Now we can include the j_u as adjustment to this term

$$j_u = \frac{U_2}{U_1} = \left[\frac{(H-w)}{(h-w)}\right]^{1.5} = \left[\frac{\frac{(H-w}{H}}{\frac{(h-w)}{H}}\right]^{0.5} = \left[\frac{1-W}{y-W}\right]^{0.5}$$

$\sin\phi = \left[\left(\frac{(y-W)}{\left[3-2y-W\right]}\right)\left(\frac{1-W}{y-W}\right)\right]^{0.5} = \left[\frac{1-W}{3-2y-W}\right]^{0.5}$ which is comprised of j_u, j_h and j_ϕ

Effect of Channel Shape

According to earlier work by Hager, a longitudinal width contraction yields a more uniform lateral outflow distribution and nearly a horizontal free surface profile. On the other hand, a longitudinal expansion leads to extremely non-uniform flow characteristics. When a contraction is present, the following total flow angle forms

$\phi_{tot} = \phi - 1/2\,\theta \geq \phi$ where ϕ_{tot} is the angle formed between the plane of the weir and the lateral flow direction. θ (negative number) is the contraction angle of the channel.

A corresponding effect on the lateral outflow angle is obtained for bottom slopes So<1 when compared to nearly horizontal channels. Then the earlier *sin* equation modifies to

$$\sin\left\{\phi + \frac{1}{2}(\theta + S_0)\right\} = \left[\left(\frac{(y-W)}{\left[3-2y-W\right]}\right)\right]^{0.5}$$

Letting $\lambda = \frac{1}{2}\left(\theta + S_0\right) << 1$

$$\sin(\theta+\lambda)/2 = \sin\phi\cos\gamma + \cos\phi\sin\lambda = \sin\phi(1 - 1/2\,\lambda 2) + \lambda\cos\phi = \Phi$$

after rearranging

$$\sin\phi = \left[\left(\frac{(y-W)}{[3-2y-W]}\right)\right]^{0.5} . \left\{1-\lambda\left[\frac{3(1-y)}{y-W}\right]^{0.5}\right\}$$

Since λ <<1 then the second term is only corrective to the first term.

Note: Coulomb (1967) arrived at a similar equation. However, that equation neglected the effects of channel slope and shape.

Lateral Outflow Intensity

Combining the above equations thereby accounts for *j*. The lateral Intensity becomes:

$$Q' = \frac{3}{5} n * c\sqrt{gH^3}(y-W)^{3/2}\left[\frac{1-W}{3-2y-W}\right]^{0.5} . \left\{1-(\theta+S_0)\left[\frac{3(1-y)}{y-W}\right]^{0.5}\right\}$$

where $n*$ = the number of outflow sides (one or two weirs)

c= weir crest influence. c=1 for sharp crested, c=8/7 for zero height and for

broad crested weirs $c = 1-\frac{2}{9(1+\zeta_b{}^4)}; \zeta_b = \frac{H-W}{Lc}$, *Lc* in this case is the width of the broad crest.

For round crested weirs $c = \frac{\sqrt{3}}{2}\left(1+\frac{\frac{22}{81}\zeta_r{}^2}{1+\frac{1}{2}\zeta_r{}^2}\right); \zeta_r = \frac{H-W}{r}$ r= crest radius

Hager compared his equation (by modifying it for zero height weir) to Nadesamoorthy

$H = h + \frac{Q^2}{2gA^2} = h(1+\frac{F^2}{2})$ so $\frac{1}{y} = 1+\frac{1}{2}F^2$

for zero height weirs (w=0) Hager's equation becomes:

$$Q' = -\frac{3}{5}\frac{8}{7}\left[\frac{2}{2+F^2}\left(\frac{2}{2+3F^2}\right)^{0.5}\right] n * \sqrt{gH^3}$$

then $C_d = \frac{-Q'}{n*\sqrt{2gh^3}} = 0.485\left[\frac{(2+F^2)}{(2+3F^2)}\right]^{0.5}$

The comparison concludes that there is a 20% higher C_d for F approaching F_1. The differences increase asymptotically as F_1 approaches infinity. There, the $C_d = \frac{0.407}{2}$ (Nadesamoorthy) while $C_d = \frac{0.485}{\sqrt{3}}$ (Hager)

The range for Hager's experiments was

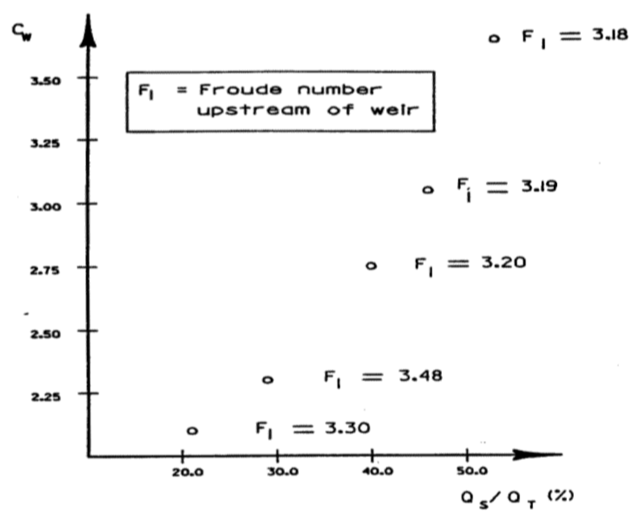
$0.3 \le F_1 \le 2$ and $-0.5 \le S_0 \le 2\%$

1987 Natsuhara

Natsuhara, as part of his graduate research on supercritical weir flow, identified the weir coefficient and the channel friction coefficient as "two key factors" in determining the weir flow. He then proceeded to compare weir coefficients in subcritical with those in supercritical flows. He concluded that the weir coefficient is not sensitive to the variation of the energy correction factor. In that, his experiments resulted in 0.91 percent difference in α when C varied from 2.0 to 4.0. On the other hand, a 16 percent change in by-pass discharge was realized by varying the C by that same amount. When the Manning's friction factor was varied from 0.01 to 0.02, the weir flow varied by 22 percent.

As his research focused on one specific case-study with a limited range of Froude numbers, he remarked that a side-weir may only be designed efficiently and reliably by use of both physical and mathematical models.

He presented the following graph for the determination of the weir coefficient versus the discharge ratio $\frac{Q_w}{Q_{ds}}$ for a Manning's factor of 0.014 and for a small range of Froude numbers between 3.18 and 3.3.



Weir Coefficients Versus Discharge

1991 Cheong

According to Swamee (1994), Cheong arrived at a discharge coefficient relationship with the upstream Froude number for a zero height weir.

$C_d = 0.45 - 0.22F_1^2$ for w=0

1994 Singh and Satyanarayana

They used the De Marchi approach of constant specific energy. They disagreed with Subramanya and Ranga Raju in that the weir height/flow depth is material in the determination of Cd. They arrived at

$Cd = 0.33 - 0.18F_1 + 0.49\frac{w}{h_1}$ which was derived in an earlier paper by Singh in 1992.

1994 Swamee, Pathak, Mohan, Agrawal and Ali

They presented several equations such as Subramanya's, Nadesmoorthy's, Raju's, Cheong's and Hager's and claimed that the absence of the term $\frac{(h-w)}{w}$ made all the equations erroneous and then proposed their own equation $C_d = 0.447\left[\left(\frac{44.7w}{49w+h}\right)^{6.67} + \left(\frac{h-w}{h}\right)^{6.67}\right]^{-0.15}$ with an average percentage error of 6.6%

The range of application is for $F_1 = 0.1 - 0.93$

1999 Borghei, Jallili and Ghodsian

These researchers claimed that a universally acceptable discharge coefficient did not exist. Through several experiments, they also confirmed that in subcritical flow, the De Marchi assumption of constant energy is acceptable. Their experiments concluded that the change in average energy is about 3.7% from the upstream of the weir to the downstream.

They also "discovered" that the De Marchi coefficient of discharge is a function of the upstream Froude number F_1 and the ratios of the weir height to upstream depth (w/h) and weir length to channel width (L/B).

They discussed several earlier studies such as those of Subramanya, Raju, Hager and Singh and pointed that all accept the upstream Froude number as the main variable in the formula and most have introduced the C_d as a function only of F_1. They disagreed and referred to the Ramamurthy and Carballada studies that showed that the L/B is another parameter to be considered. They also referred to Singh's belief that the influence of w/h_1 is more than L/B. Since as w/h_1 increases, the discharge decreases, w/h_1 should appear in the formula with a negative sign. As for the channel slope, they concluded that the influence is very small and should be ignored.

Finally, they arrived at an equation that includes all three parameters.

$$C_d = 0.7 - 0.48F_1 - 0.3\frac{w}{h_1} + 0.06\frac{L}{B}$$

CONCLUSIONS ON THE ANALYSIS OF SIDE-WEIRS

The discussion presented in this appendix is a brief summary of research conducted by the author. For a complete understanding of the theory and practical application, it is encouraged to review the source papers. In all, this summary yields insight to the complexities of analyzing spatially-varied-flow for the case of decreasing discharge. It is realized that significant work has been accomplished by competent researchers. The disparity of results however, casts confusion on the direction that the engineer must follow. In that light, a responsible engineer must be satisfied with the methods that he/she implements. The author of this document is not an exception to this.

The author chose two methods as most applicable for the design and analyses of side-weirs in subcritical flow. The methods are those of Mostafa (1972, 1974 and 1987) and Hager (1987). In either method, the author utilizes the energy or the momentum principles. However, the coefficient of discharge (C_d) varies from one method to the other. Mostafa arrived at a relationship for the determination of C_d. However, Hager wrote the whole lateral intensity (dQ/dx) term in terms of the $\sin\phi$ term. It was observed that for a limited range of weir flows, the correction term using both methods was very similar. For others, it varied. Since the Mostafa term has been extensively used in the Southern California region and verified by constructing numerous retarding basins which are based upon this term, the author, for the present, recommends a continuation of this approach. The Hager method appears to be thoroughly researched (and modeled) and his conclusions are very reasonable. In the opinion of this author, this method has significant promise and should be considered in future studies.

APPENDIX D - OUTLET CONDUITS

Offline retarding basin outlets are included in the retarding system to evacuate the stored flood volume at a retarded rate. Outlets are essential components of a retarding basin and play a major role in the efficiency of flow retardation. Improperly designed outlets could result in either; excess flows to the downstream channel or not sufficiently releasing the flood-storage, which could result in overwhelming the basin as inflows exceed outflows. Properly designed outlets may provide substantial hydraulic efficiency to a system, which in turn yields high economy.

Offline basins generally provide best function for single recurrence frequency storms. For such basins, the design of the outlet size and configuration is usually for the peak head within the basin. However, increasingly retarding basins are being used to retard multiple recurrence frequency storms. Further, some basins are required to retain early flows for water quality purposes. Although information in this appendix is geared towards a single-recurrence frequency storm, the same hydraulic principles may be applied for other frequencies.

Outlet Features

Outlet Size

The following are general guidelines for sizing the outlet conduit of an offline retarding basin:

- Evacuate a basin within a specified time period while considering inflows into the basin.
- Consider limiting the flows to acceptable amounts in the main channel, downstream of the outlet.
- Permit human inspection of the conduit
- Sized to evacuate incoming base flows at a small available head.
- Include sufficient outlet slope to result in a minimum flushing velocity of approximately 3-4 fps at low basin head.

Outlet Location and Type

Basin outlets may be tunnel or conduit outlets. Unlike dam outlets, offline retarding system outlets are conduits that are usually constructed in a cut and cover rather than by a tunneling operation. Therefore, issues of settlement, piping, leakage and seepage are more seriously considered since failure of a portion of the outlet could become catastrophic for the basin.

Typically, the outlet's flow-line is set at either the lowest point in the basin's invert or even lower. At the main channel, the outlet's flow line should be set at the lowest elevation possible. Naturally, this elevation should be above the maximum anticipated aggraded (sedimented) channel grade at that location.

When flows are conveyed through the outlet and into a natural river or channel, proper energy dissipation should be included at the downstream end of the conduit. For proper design of the outlet, the following should be addressed properly:

- Place outlet on the most competent portion of the basin's foundation. When possible, place the conduit on bedrock. Else, excavation and fill with competent material must be accomplished until it is determined that settlement will be minimized.
- Design details should allow for expected settlement. The conduit profile must be adjusted to provide for the drop in grade near the center of an embankment above the conduit.
- Ensure that non-permeable material surround the conduit. Cutoff collars should be used intermittently to ensure that the chance for piping is minimized.
- Conduit structure must be able to withstand the full vertical and lateral external loads imposed upon the conduit.
- Ensure that the conduit can withstand the full pressure head. A conservative value for this head is the water surface within the basin at an elevation slightly greater than the elevation of the emergency spillway crest.
- Ensure water tightness of the conduit by placing water seals at the joints.

Gated Outlets

Gated outlets allow for better control on releases. However, they are more costly to construct and maintain. An improperly maintained outlet may null the retarding benefits and eventually result in the system's failure. In addition, care should be taken at small gate openings as cavitation may result.

When gates are positioned at the lower end of the outlet, full internal pressure needs to be included in the structural design. On the other hand, when gates are placed at the upstream end in a steep conduit, partial flow will occur throughout the conduit and internal pressure need not be considered. In this case, only external loads need to be considered.

Un-Gated Outlets

Offline basin outlets are typically un-gated and are designed for relatively low capacities. While at a glance, the outlet's design considerations may appear simplistic, the opposite may be the case. Properly designed outlets need to consider a multitude of issues; including debris clogging, cavitation, energy dissipation, transitions, etc..

Flap-gated Outlets

Flap-gated outlets prevent back flows from the main channel into the basin. In essence, they are one-way valves. Such gates are usually included in systems with subcritical, or high, depths in the main channel at its junction with the basin's outlet. In such systems, significant benefits may be realized by use of flap-gates.

A disadvantage to utilizing such a gate is the possibility of the gate becoming lodged open or closed. If the gate is lodged open, some depletion of available basin storage will occur and the basin will not function as intended. However, if the gate is lodged closed, the inability of the basin to evacuate its storage volume will quickly fill the basin and null the basin's retarding benefits. This may have catastrophic results on the system's function as a whole.

Trash Racks

Trash racks should be included at the entrance of the outlet to prevent debris from blocking it. The hydraulic capacity of trash racks should be between two and ten-times the capacity of the outlet at the same head.

Outlet Flow Types

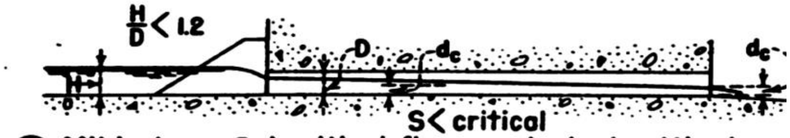
Flow through an outlet conduit may be directed into or out of a basin depending upon the relative water surface elevations of the basin and the main channel (at the location of the outlet). Discussions herein will revolve around flows out of the retarding basin. Further, since outlets are normally designed with a uniform cross-section and placed at a uniform grade, the following discussion will assume these conditions. There are two main hydraulic conditions that the outlet may experience, regardless of the flow direction. As the level of water in the basin is low, open-channel may occur through the outlet. Once the water surface rises, pressure flow may ensue. Except for the beginning and end of the storm hydrograph, pressure flow usually occurs. Significant flow over-estimation errors occur if flow is assumed as pressure throughout the storm hydrograph.

The combined effect of head differential (between the basin and channel), slope, size, shape, length, roughness and inlet geometry determines the nature of the flow in the outlet conduit. The Design of Small Dams (1987) text presents the figures (USBR fig. 9-67) below, which highlight the main flow types, cases and controls. Also presented below is an enhanced tabular summary of the flow-types and their hydraulic controls.

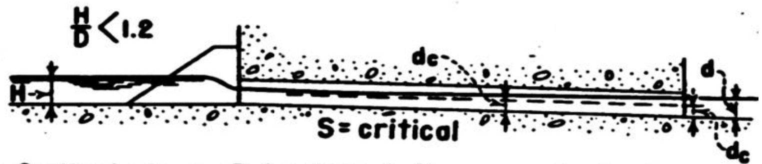
The following table summarizes those flow types, conditions and controls:

Slope	Case	H/D	Flow inside	Hyd. Control	Equation	Comment
Mild	1	<1.2	Open/Sub.	Outlet/dc	Bernoulli	
	6	>1.2	Pressure	Outlet/Ch Tail water or Dia.	Bernoulli	Outlet is supported
	6	>1.2	Pressure	Outlet/Ch at ½ Dia.	Bernoulli	Outlet is free discharging
	4	>1.2	Open/Sub.	Inlet	Orifice	Short conduit
Critical	2	<1.2	Open/Sub	Outlet/ Critical or Tail water	Bernoulli	Similar to case 1
Steep	3	<1.2	Open/Super	Inlet/dc	Bernoulli	
	5	>1.2	Open/Super.	Inlet/ Orifice	Orifice	
	7	1.2-1.5	Open and Pressure (slug)	Inlet and Outlet	Bernoulli and Orifice	Vortex action at Inlet
	8	>1.5	Pressure	Outlet/Ch Greater of Tail water or Soffit	Bernoulli	Outlet is supported
	8	>1.5	Pressure	Outlet/Ch Greater of Tail water or ½ Dia	Bernoulli	Outlet is free discharging

PART FULL FLOW - INLET NOT SUBMERGED



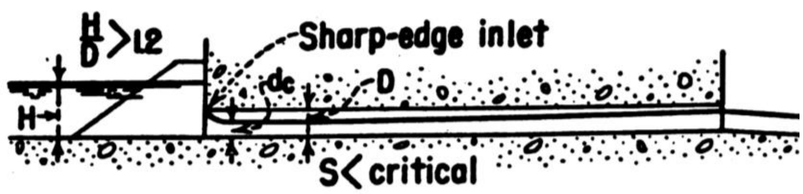
① Mild slope. Subcritical flow, control at critical depth at outlet.



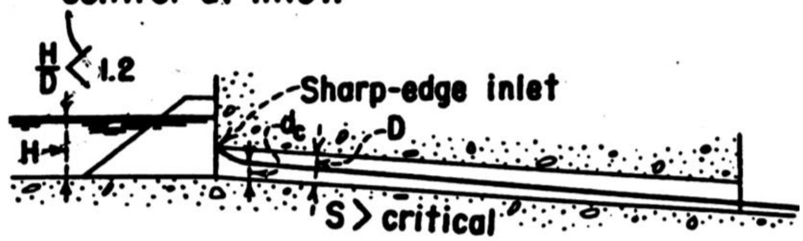
② Critical slope. Subcritical flow, control at critical depth at outlet or some greater depth if backwater exists.

③ Steep slope. Supercritical flow, control at critical depth at inlet.

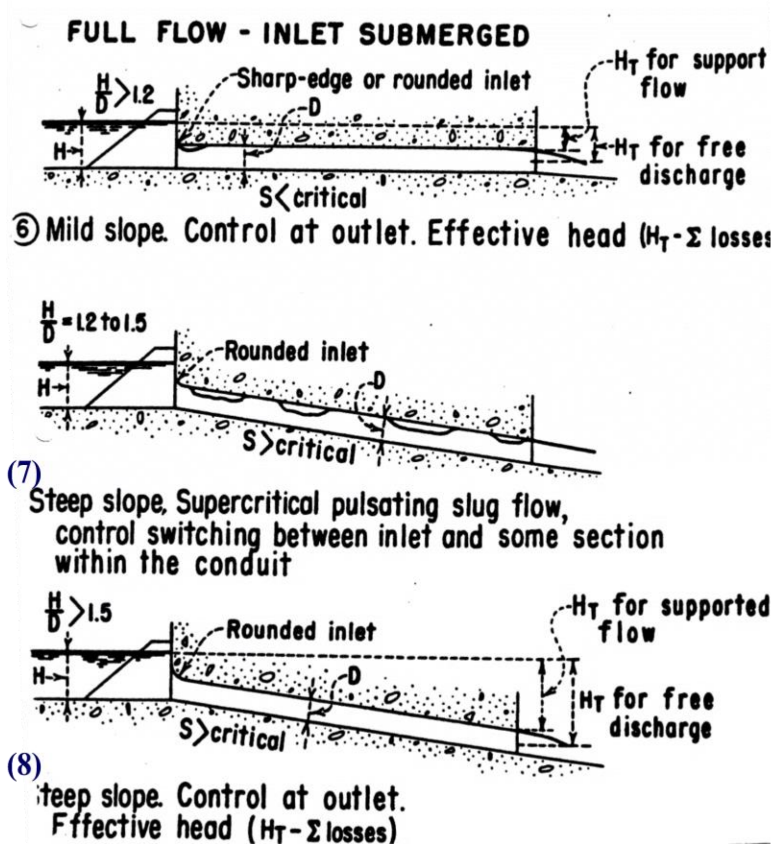
PART FULL FLOW - INLET SUBMERGED



④ Mild slope. Supercritical flow, orifice flow control at inlet.



⑤ Steep slope. Supercritical flow, orifice flow control at inlet.



Partially full flow

Partially full flow is governed by the same principles that apply to open-channel flow. For information regarding open-channel flow, see Appendix A.

Full flow

Full flow is governed by the same principles that apply to pressure flow. For information regarding pressure flow, see Appendix A.

Outlet Flow Analysis

Steps for the analysis

- Determine if the outlet slope is mild or steep. This is needed for open-channel flow calculations.

- Based upon previous hydrograph time-step, use the water surfaces of the channel (at the outlet) and the basin for determining the type of flow

- Determine the hydraulic control.

- Calculate the outlets discharge.

- Combine the outlet discharge with the channel discharge downstream of the weir and determine the water surface at the outlet. This water surface could become the downstream control for the next step.

Determining outlet slope

This is done by assuming that the conduit is flowing half-full at critical flow and calculating the critical slope (Sc). Then, this slope is compared with the outlet's physical slope (S_0). If the (S_0) is greater than (Sc), then the slope is Steep. Otherwise the slope is Mild. The following is the derivation of the critical slope relationship at half full circular conduit:

$$Fr = \frac{V}{\sqrt{gd_h}} = 1 \text{ where } dh = \frac{A}{T} = \frac{\pi D^2}{2(4)D} = \frac{\pi D}{8} \text{ (at half full conduit)}$$

$$V = Fr\sqrt{gd_h} = (1)\sqrt{g\frac{\pi D}{8}}$$

and from Manning $V = \frac{1.486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$

then $\sqrt{g\frac{\pi D}{8}} = \frac{1.486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$ where $R = \frac{A}{P} = \frac{\pi D^2}{2(4)\frac{\pi D}{2}} = \frac{D}{4}$

simplifying

$$3.56\sqrt{D} = \frac{0.59}{n} D^{\frac{2}{3}} S_c^{\frac{1}{2}}$$

$$Sc = \left[\frac{3.56n\sqrt{D}}{0.59D^{\frac{2}{3}}}\right]^2 = \frac{36.41n^2}{D^{\frac{1}{3}}}$$

Where D is the diameter of the pipe and T is the width at surface flow. Since the pipe is half full, T = D.

Determining the hydraulic controls

Inlet Control

The control section is located near the entrance of the pipe. The discharge through the conduit is dependent upon the inlet geometry and the head. This type of control is indicative that the conduit's capacity is greater than the inlet's. In steep conduits, almost always the flow will be open-channel and supercritical past the inlet. However, in mild conduits, there will be a short distance of open-channel while the remaining length of the conduit may flow under pressure.

Outlet Control

The control section is located near the outlet of the pipe. The discharge through the conduit is dependent upon the inlet geometry, the head differential (between the basin and channel), shape, size, slope, length and friction factor. Conduits that are controlled at their outlets, may have either pressure or open-channel flows within.

The Bernoulli Equation for full-flow pipes

The following summarizes the Bernoulli equation for various tail water conditions:

$$\Delta H = .0252\left[1 + K_e + \frac{29.1n^2L}{R^{4/3}} + additional\right]\left(\frac{Q}{D^{5/2}}\right)^2$$

where

$\Delta H = H + L\sin\theta - D/2$ for jet conditions at outlet

$\Delta H = H + L\sin\theta - D$ for supported jet conditions at outlet

$\Delta H = WS_{ba\sin} - WS_{channel}$ for tail water above pipe soffit

and

H = depth within the basin at the inlet

Losses for full flow pipes

Since outlet energy loss factors are largely based upon empirical data, the designer should consider the range for each coefficient on the system. For example, for basin capacity calculation, high loss factors should be used. On the other hand, low-loss factors should be used when considering the main-channel's capacity downstream of the outlet. Similarly, low-loss factors should be used if energy dissipation is being considered at the discharge point of the outlet. The main loss-factors are presented in Appendix A. Below are additional factors to be considered:

- **Trash racks (K_t)** – Depends upon the rack's bar spacing and net flow area. The following equation may be used

$$K_t = 1.45 - 0.45\frac{a_n}{a_g} - \left(\frac{a_n}{a_g}\right)^2$$

where a_n = net area through the rack bars and a_g = gross area of racks and supports

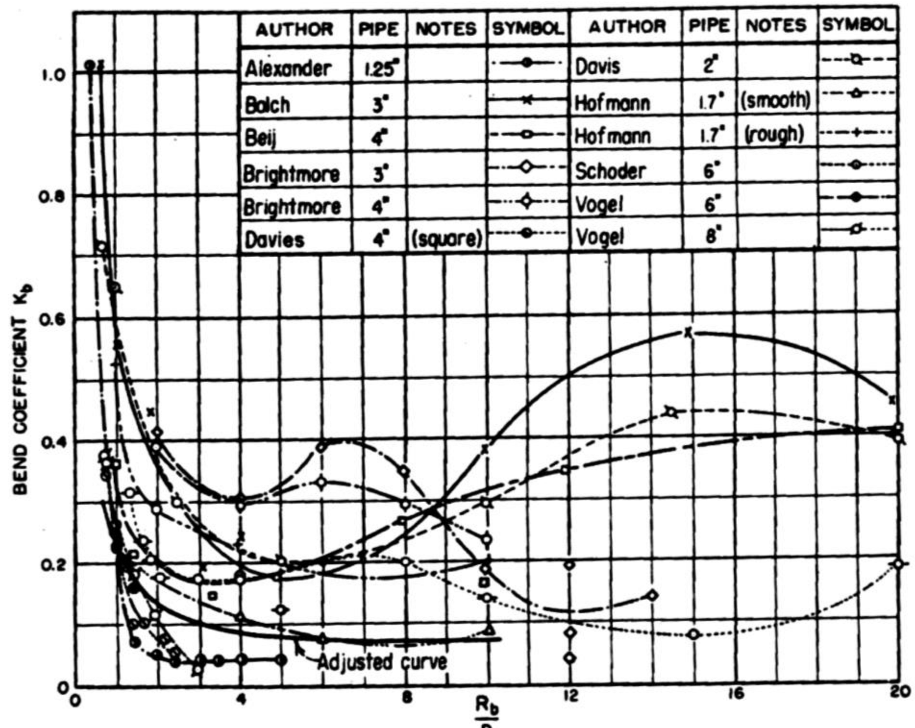
- **Entrance (K_e)** – Inlets may be rounded or square edged. The table below (USBR 1987) presents the range for the K_e coefficient

Shape	Max	Min	Average
Square cornered	0.7	0.4	0.5
Slightly rounded	0.6	0.18	0.23
Fully rounded (r/D)=>0.15	0.27	0.08	0.10

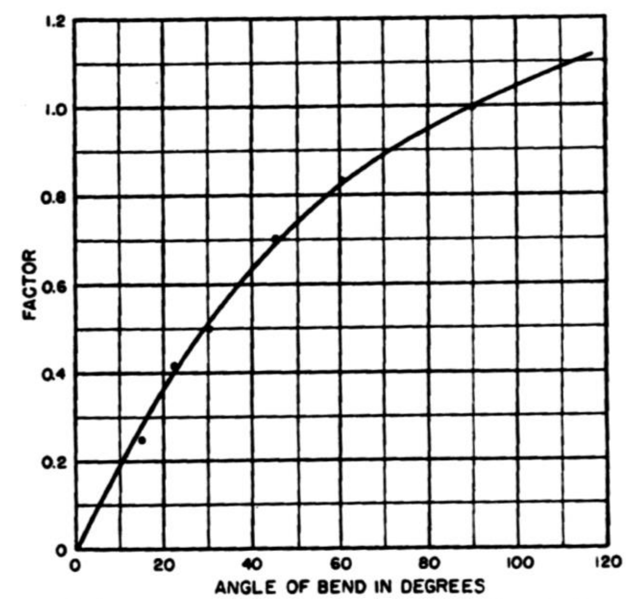
- **Friction (K_f)** – friction factors may be obtained from local criteria for conduits. In general, the maximum and minimum Manning's "n" factors for RCP pipes are 0.014 and 0.008 respectively
- **Contraction (K_c)** – the losses in contraction transitions are equal to the K_c multiplied by the absolute value of the difference in velocity head from the upstream and the downstream of the transition. The factor varies from 0.1 for gradual contraction to 0.5 for abrupt contraction
- **Expansion (K_{exp})** – the losses in expansion transitions are equal to the K_{exp} multiplied by the absolute value of the difference in velocity head from the upstream and the downstream of the transition

Flare Angle	2^o	5^o	10^o	12^o	15^o	20^o	25^o	30^o	40^o	50^o	60^o
K_{exp}	0.03	0.04	0.08	0.10	0.16	0.31	0.40	0.49	0.60	0.67	0.72

- **Bend (K_b)** – the figure below (USBR 1987) presents a guide for the bend loss coefficient
- **Gates (K_g)** – no loss will occur if the gate is mounted at the entrance to a



(A) VARIATION OF BEND COEFFICIENT WITH RELATIVE RADIUS FOR 90° BENDS OF CIRCULAR CROSS SECTION, AS MEASURED BY VARIOUS INVESTIGATORS



(B) FACTORS FOR OTHER THAN 90° BENDS

conduit where the gate does not interfere with the flow. However, where the gate does interfere and creates a jet at the entrance, the following values for the coefficient should be considered:

Max	Min	Average
1.8	1.0	1.5

- **Exit (K_{ex})** – the exit loss coefficient for the flow cases presented earlier is 1.0

The Orifice Equation

$$Q = CA\sqrt{2gH}$$

where *A* = area of opening, *H* = difference between upstream and downstream water levels and *C* = discharge coefficient

Shape	Max	Min	Average
Square cornered	0.85	0.77	0.82
Slightly rounded	0.92	0.79	0.9
Fully rounded (r/D)=>0.15	0.96	0.88	0.95

Other Considerations

Cavitation

Cavitation is defined as the successive formation and collapse of vapor pockets in low-pressure areas associated with high velocity flow. Such phenomenon causes severe damage (pitting) to the concrete (or steel) structure of the outlet. Cavitation in an outlet structure may be caused by abrupt changes in the flow boundary, which causes separation of flow. In general, maximum values for velocities have been concluded from empirical experimentation. For example, it is recommended that the maximum velocity on a concrete surface not exceed 35 fps.

The presence of pits along the flow boundary will accelerate cavitation. Therefore, it is essential that repair be completed with durable material (epoxy, mortar, etc.) as early as practicable.

Air Demand

In cases where pressures in a conduit may fall below atmospheric levels (approaching vapor pressures of water), fluctuations in pressures may be accompanied by excessive vibrations, which may significantly damage the conduit. Typical locations of such negative pressures may be just upstream of the location of a hydraulic jump within a conduit. This would occur in a conduit that is steep, yet controlled both at the upstream and downstream (inlet and outlet controls). One potential solution to this problem is to include an air vent at the location of reduced pressures.

APPENDIX E - SPILLWAYS

Offline basin spillways are included in the retarding system to evacuate excess storage safely away from the basin's embankment. Hydraulic and structural adequacy of the spillway cannot be over emphasized in order for the spillway's flows to not undermine or erode the basin's embankment.

The following discussion on Spillways is gleaned from the US Army Corps of Engineers' (COE 1992), US Bureau of Reclamation's manual (1987) and miscellaneous other resources. Therefore, it is recommended to review such manuals during the design of spillways for a complete understanding and application of the issues and options involved.

General Considerations

Offline basin spillways are generally used as "emergency" spillways and are included as a main system design component. In that, the design of the system should include freeboard between the crest of the spillway and the design water surface elevation within the basin. Storms producing excess volumes beyond the design volume may then exceed the maximum design water surface in the basin and even cause overtopping of the spillway's crest (spillway flow). Federal, State and Local criteria may require the design of a retarding system for a 100-year flood event while requiring the analysis of the spillway under a 1000-year or higher flood event. Further, the designer should consider the behavior of the retarding system under recurring floods where the storage within the basin is unable to be completely evacuated before the arrival of a subsequent storm.

Since emergency spillways are provided to primarily avoid an overtopping of the basin's embankment, the structure must be designed for erosion resistance higher than the embankment itself. To that effect, the spillway crest is set lower than the basin's embankment and is usually constructed with durable material such as concrete with proper cutoffs and riprap protection along its boundary.

At the downstream end of the spillway, the dissipation of the high-energy flows must be accomplished prior to depositing the flows into a natural drainage course. Design of energy dissipation devices is beyond the scope of this manual.

Spillway Categories

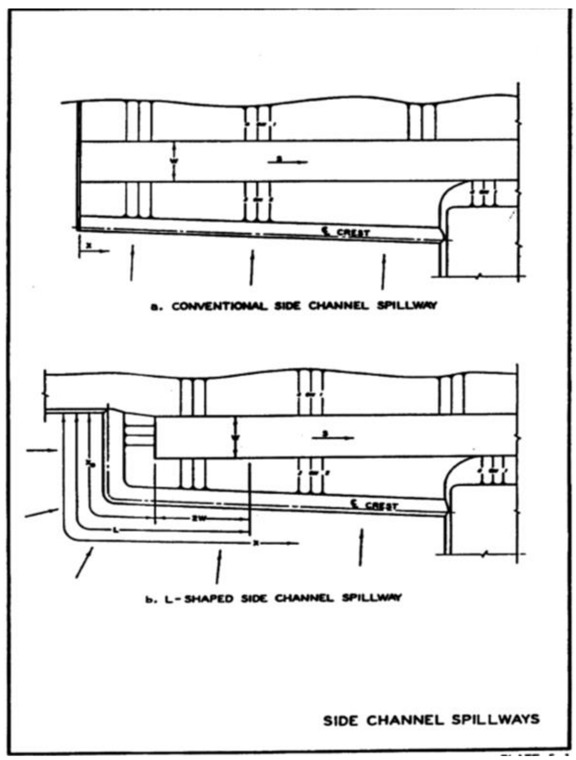
While there are several types of spillways, there are three main categories that are usually considered. These include

- Side channel Spillway

- Overflow Spillway
- Chute Spillway

Such devices may be controlled or uncontrolled. Controlled spillways are regulated with gates and valves to control the discharge amount. For an offline retarding system, the most common type of a spillway is either an ogee or sharp-crested overflow spillway. Obvious advantages of such spillways are their less complicated construction, automatic and trouble-free operations and their ability to function without an attendant. Therefore, with the exception of the following figures for each category, discussions in this manual are limited to the sharp-crested weir and overflow (ogee) spillway types.

Side Channel Spillway (COE 1990)



Chute Spillway (COE 1990)

EM 1110-2-1603
16 Jan 90

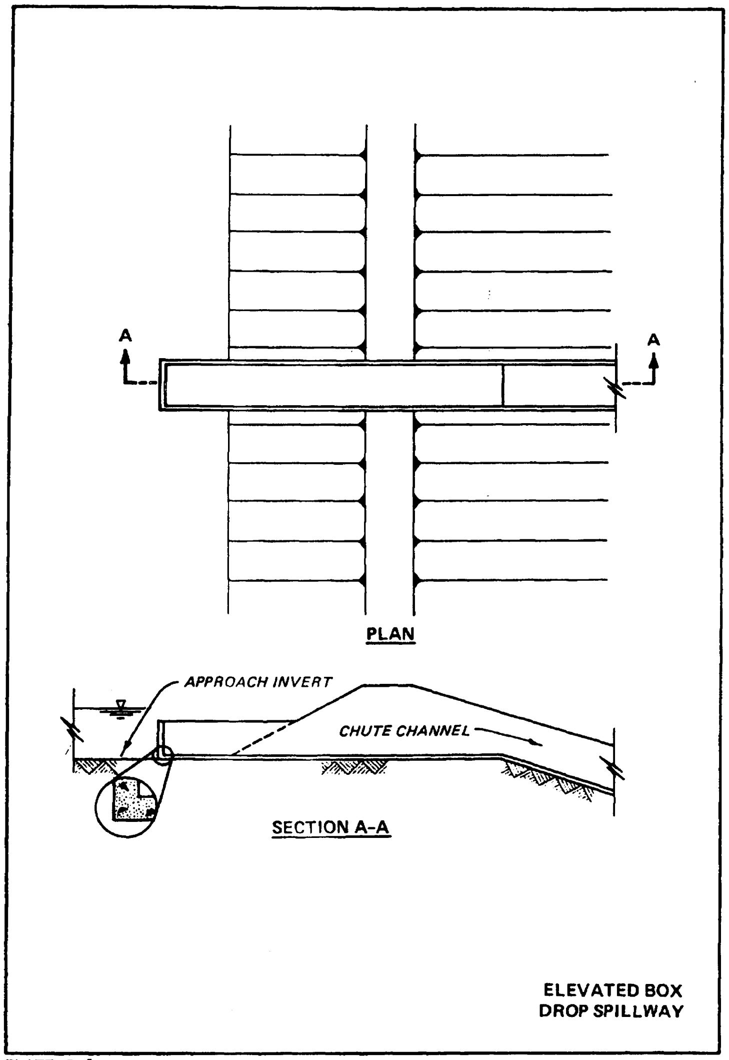


PLATE 5-6

Sharp-crested weir analysis

Flow Shape

The sharp-crested weir is the simplest form of a spillway. The characteristics of flow over this type of spillway form the basis for more complicated shapes such as the ogee spillway. The shape of the underside of the nappe (for sharp crests) is the ideal surface shape of an ogee spillway. That shape resembles a projectile's trajectory in both the x and y directions. Chow (1959) presents the figure below showing the shape of the underside of the nappe:

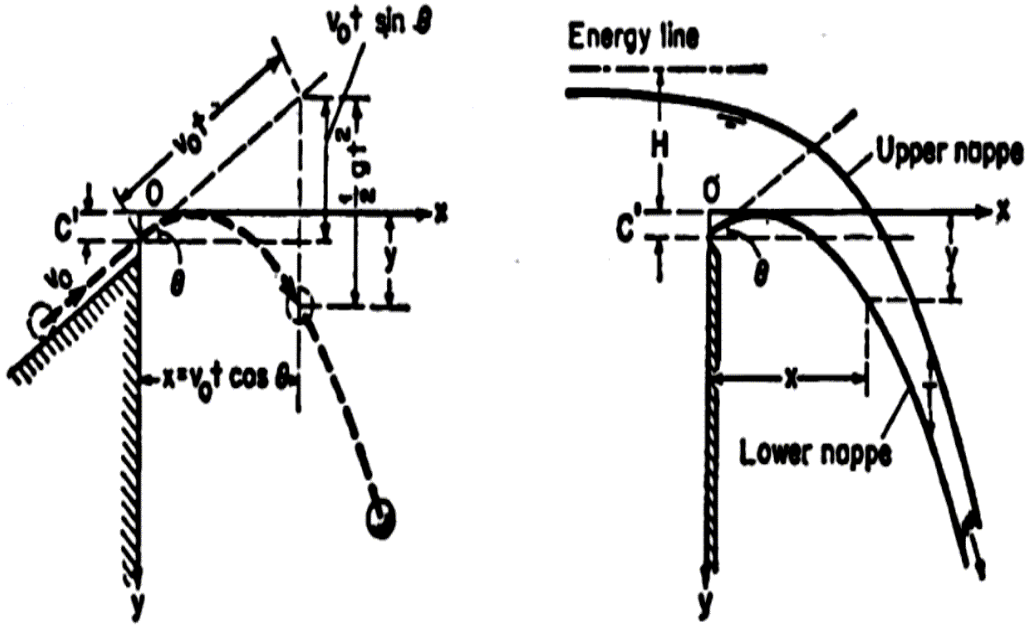


FIG. 14-1. Derivation of nappe profiles over sharp-crested weir by the principle of projectile.

A projectile travels a distance x in a time t. The relationship between the two terms with respect to the velocity at the brink is

$$x = v_0 t \cos\theta$$

In the y direction, the particle travels

$$y = -v_0 t \sin\theta + \frac{1}{2} g t^2 + C'$$

where C' is the value of y at x=0

by eliminating t from both equations and dividing each term by the total head (H) above the crest, the following results:

$$\frac{y}{H} = A\left(\frac{x}{H}\right)^2 + B\left(\frac{x}{H}\right) + C$$

where $A = \frac{gH}{2{v_0}^2 \cos^2\theta}$ and $B = -\tan\theta$ and $C = \frac{C'}{H}$

since the horizontal velocity is constant, the vertical thickness of the nappe T may be assumed constant. By adding the term $D = \frac{T}{H}$, the general term becomes $\frac{y}{H} = A\left(\frac{x}{H}\right)^2 + B\left(\frac{x}{H}\right) + C + D$

The constants for the above equation, based upon the USBR (1987), are

$$A = -0.425 + 0.25\frac{h_v}{H}$$

$$B = 0.411 + 1.603\frac{h_v}{H} - \sqrt{1.568\left(\frac{h_v}{H}\right)^2 - 0.892\frac{h_v}{H} + 0.127}$$

$$C = -0.150 + 0.45\frac{h_v}{H}$$

$D = 0.57 - 0.02(10m^2)\exp(10m)$ where $m = \frac{h_v}{H} - 0.208$ and h_v is the velocity head of the approach flow.

For high weirs, the velocity of approach is relatively small and can be ignored which results in A=-0.425, B=0.055, C=0.15 and D=0.559. Experiments have shown that these equations are not valid for x/H < 0.5 and when x/H > 0.2 additional data for verification are needed. It should be noted that the above only applies when the flow is subcritical which resembles the flow exiting an offline retarding basin.

Sharp-crested weir hydraulics

The discharge equation for the sharp-crested weir spillway is:

$$Q = CL_e H_e^{1.5}$$

where:

Q = discharge,

C = variable discharge coefficient. According to Rehbok (Chow 1959), the formula for C is $C = 3.27 + 0.40\frac{H}{w}$ where w is the height of the weir.

H_e = measured head above the crest, excluding velocity of approach, h_v.

L_e = effective length of crest = $L - 0.1nH_e$

where

- L=net length of crest
- n = number of contractions (=2 for 2 end contractions)

Chow (1959) states that experiments by Hunter Rouse showed that this equation holds up to H/w = 5 but can be extended to H/w=10 with fair approximation. For H/w greater than 15, the weir becomes a sill and the discharge is dependent upon the critical section immediately upstream of the sill. Then, the coefficient becomes

$$C = 5.68\left(1 + \frac{w}{H}\right)^{1.5}$$

The transition between the weir and the sill (H/w=10 and 15) has not yet been clearly defined.

Ogee Spillway Analysis

Spillway Shape

The shape of an ogee spillway is that of an inverted S. The shape of the upper curve follows the trajectory of flows over a sharp crested weir. Therefore, flow over the crest adheres to the concrete face and prevents access of air to the underside of the flow. A reverse curve at the bottom gradually directs flows onto an apron or downstream channel. If the upper curve of the spillway is broader than the nappe of a sharp crested weir flow, then the flow would be supported and positive hydrostatic pressures would occur along the surface. However, if the upper curve of the spillway crest is sharper, the flow tends to pull away from the crest and create negative pressures along the contact surface. Although this negative pressure may temporarily increase the discharge (and create pulsating flows), it can result in serious damage to the spillway surface should cavitation result. In conclusion, when a spillway is designed with the crest shape the same as that of the under-nappe of a jet flowing over a sharp-crested weir, maximum discharges would result for that particular flow area.

The following two figures (USBR 1987) define the ogee crest shape. The spillway crest is divided at the apex into an upstream and downstream sections. The first figure provides the shape for the downstream section while the second figure provides the shape for the upstream section.

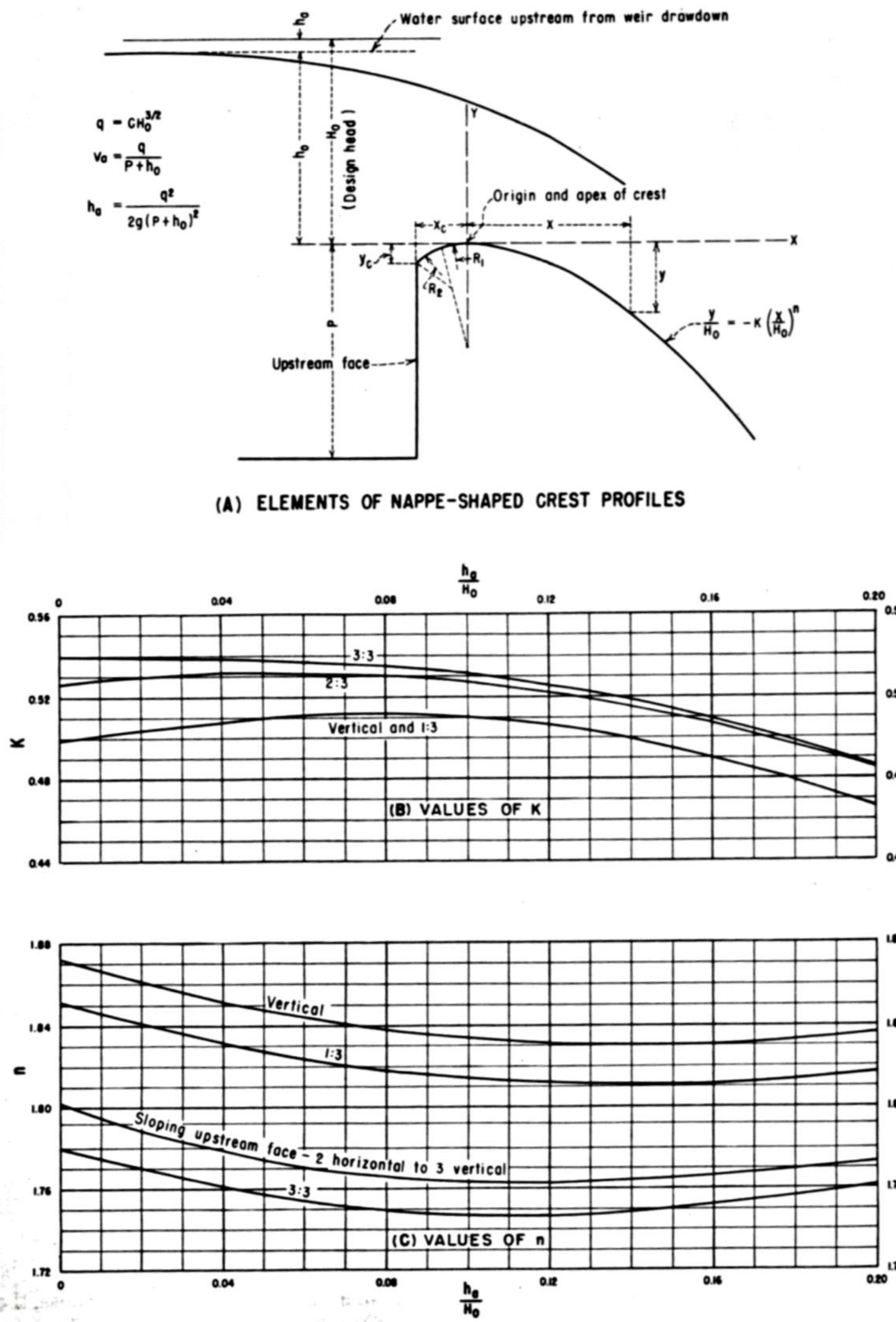
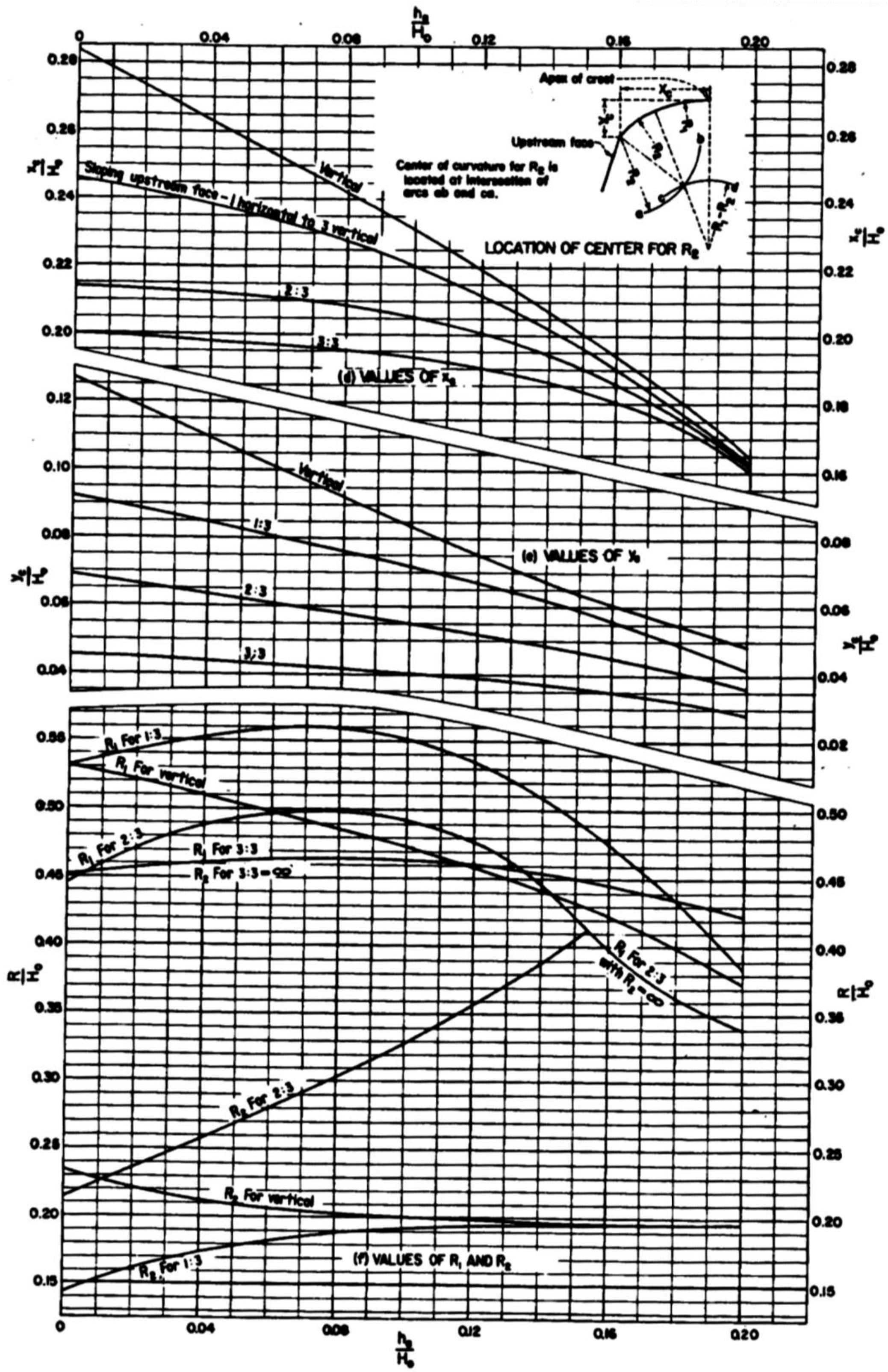


Figure 9-21.—Factors for definition of nappe-shaped crest profiles. 288–D–2406. (Sheet 1 of 2).



LOCATION OF CENTER FOR R_2
Apex of crest
Upstream face
Center of curvature for R_2 is located at intersection of arcs ab and ce.
Sloping upstream face - 1 horizontal to 3 vertical
Vertical
2:3
3:3
(d) VALUES OF X_c
Vertical
1:3
2:3
3:3
(e) VALUES OF Y_c
R_1 For 1:3
R_1 For vertical
R_1 For 2:3
R_1 For 3:3
R_2 For 3:3 = ∞
R_1 For 2:3 with R_2 = ∞
R_2 For 2:3
R_2 For vertical
R_2 For 1:3
(f) VALUES OF R_1 AND R_2
$\frac{h_a}{H_o}$

Ogee Spillway Hydraulics

The discharge equation for the ogee-shaped spillway is:

$$Q = C_{og} L_e H_e^{1.5}$$

where:

Q = discharge,

C_{og} = variable discharge coefficient

The C_{og} is influenced by the following factors:

- The depth of approach
- Relation of the actual crest shape to the ideal nappe shape
- Upstream face slope
- Downstream apron interference
- Downstream submergence

H_e = actual head being considered on the crest, including velocity of approach, h_a.

L_e = effective length of crest,

$L_e = L - 2(nK_p + K_a)H_e$ where

- L=net length of crest
- n = number of piers
- K_p = pier contraction coefficient

  K_p = 0.02 for square nosed piers

  K_p = 0.01 for round nosed piers

  K_p = 0.0 for pointed nosed piers
- K_a = abutment contraction coefficient

  K_a = 0.2 for square abutment with headwall at 90 deg to flow

  K_a = 0.01 for round abutment with headwall at 90 deg to flow

  See reference documents for more options and conditions.

Effect of Depth of Approach

USBR (1987) presents figure 9-23 below for the coefficient. As can be seen from the figure, a zero height weir results in a coefficient equal to that of a broad crested weir, 3.087. For weir heights between zero and the head value, the coefficient increases dramatically from 3.087 to 3.89. As the weir height exceeds the head over the weir, the coefficient varies much slower with a maximum coefficient of 3.95 at a weir height equaling three times the head.

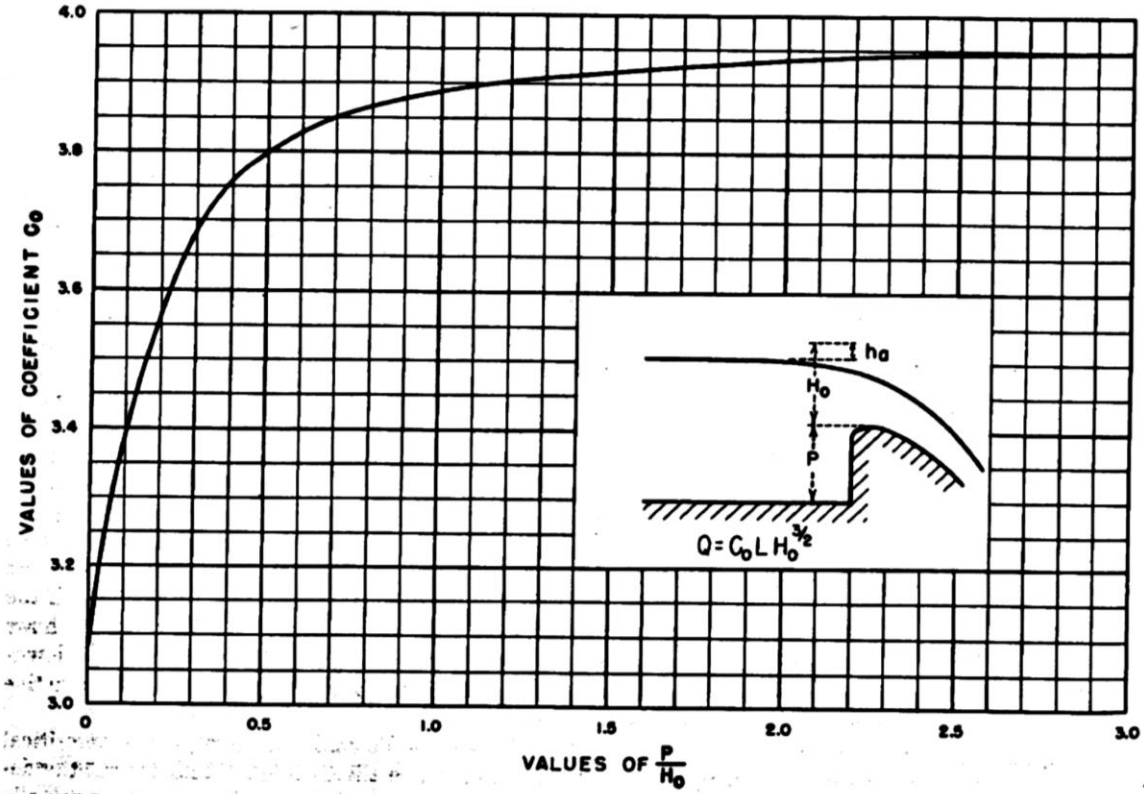


Figure 9-23.—Discharge coefficients for vertical-faced ogee crest. 288–D–2409.

Effect of Heads Different from Design Head

USBR (1987) presents figure 9-24 below for the coefficient's adjustment.

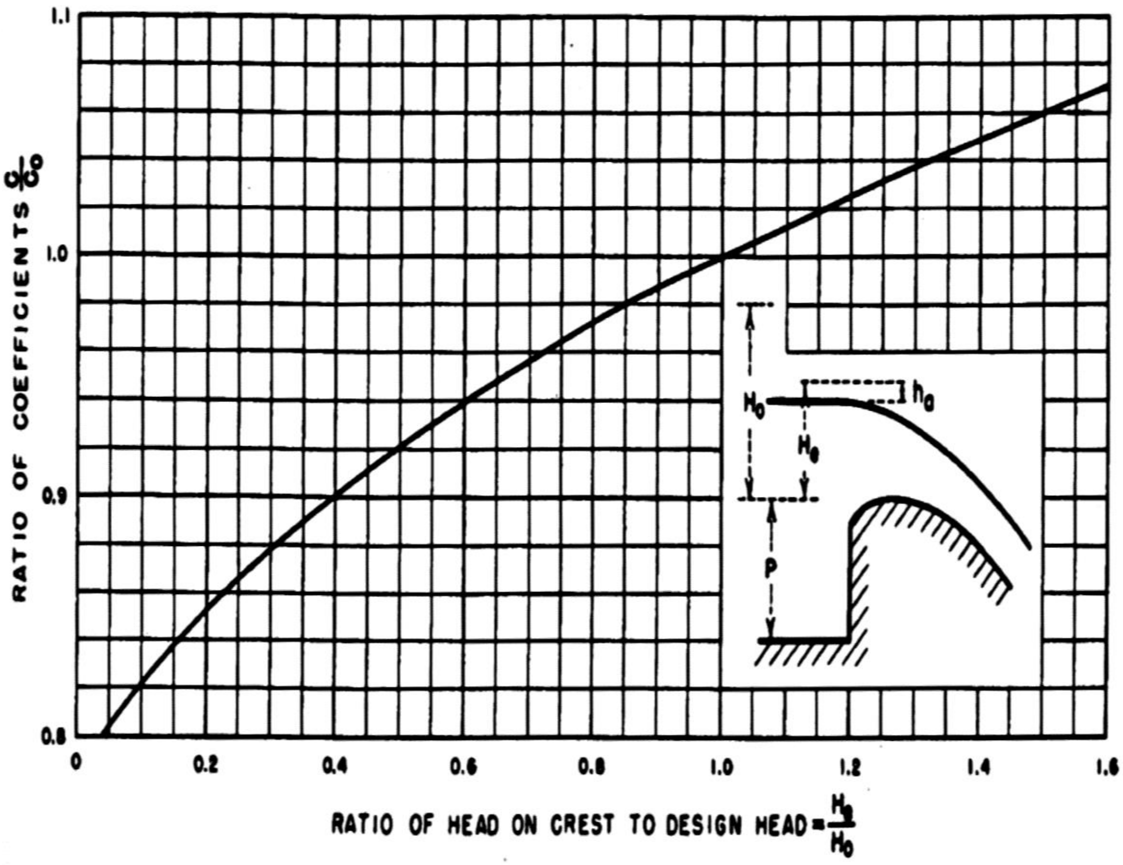


Figure 9-24.—Discharge coefficients for other than the design head. 288–D–2410.

Effect of Upstream Face Slope

USBR (1987) presents figure 9-25 below for the coefficient's adjustment.

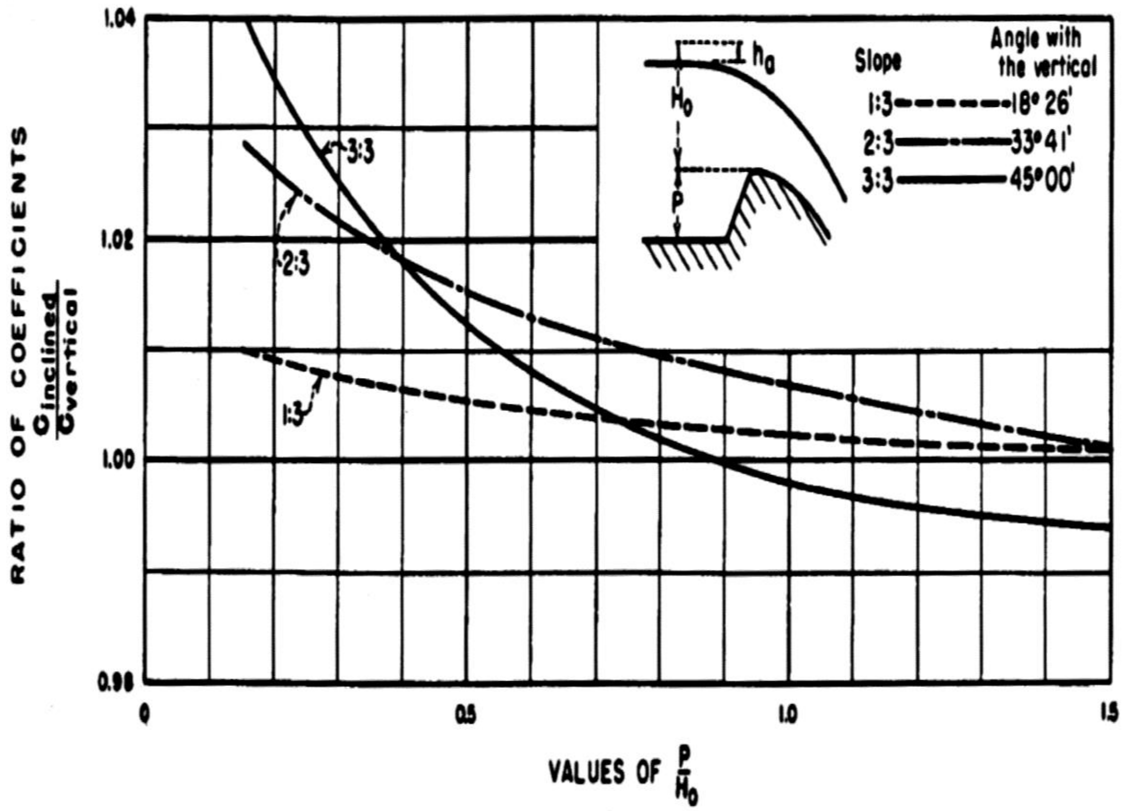


Figure 9-25.—Discharge coefficients for ogee-shaped crest with sloping upstream face. 288-D-2411.

Effect of Downstream Submergence

It is not recommended to design an offline basin's spillway for submerged conditions. If this cannot be avoided, USBR (1987) provides figure 9-27 (not shown herein) for the reduction of the coefficient.

Cavitation Considerations

Cavitation is defined as the formation of water vapor within a liquid resulting from excessively low localized pressures. Cavitation damage results when these low-pressure vapor voids are swept into a high pressure zone where the vessel collapses (implodes). Small pits are created that cause more separation of flow at the pit and lower negative pressures. This increases the size of the pit and the damage continues. Cavitation and its resulting damage to the spillway structure is dependent upon the following factors:

- Boundary shape
- Damage resistance characteristics of the boundary
- Flow velocity
- Flow depth
- Elevation of the structure above sea level
- Length of time the cavitation occurs

As a general rule, velocities exceeding 35 fps should be investigated for cavitation potential.

The following derives a guide for the determination of cavitation potential. Once the σ (Cavitation Index) has been calculated, relationships (COE 1992) may be checked for different shaped surface irregularities.

From the energy equation where section *v* is where vaporization would occur (cavitation) and section $_0$ is the actual head. P is the absolute pressure (atmospheric and gage) and H_0 is the actual head. The atmospheric pressure at sea level is 14.7 psi. The absolute pressure required for vaporization (sea level and 60 deg. F) is 0.26 psia. Since $P_{abs}=P_{atm}+P_{gage}$ then the gage pressure required for vaporization is P_{gage} = 0.26 - 14.7 = -14.44 psi.

$$\frac{P_v}{\gamma} + Z_v + \frac{V_v^2}{2g} = \frac{P_0}{\gamma} + Z_0 + \frac{V_0^2}{2g}$$

re-arranging and setting $\frac{P}{\gamma}$ = H

$$H_v + Z_v + \frac{V_v^2}{2g} = H_0 + Z_0 + \frac{V_0^2}{2g}$$

then

$$H_v - H_0 = Z_0 + \frac{V_0^2}{2g} - \frac{V_v^2}{2g} - Z_v$$

dividing both sides by $\frac{V_0^2}{2g}$

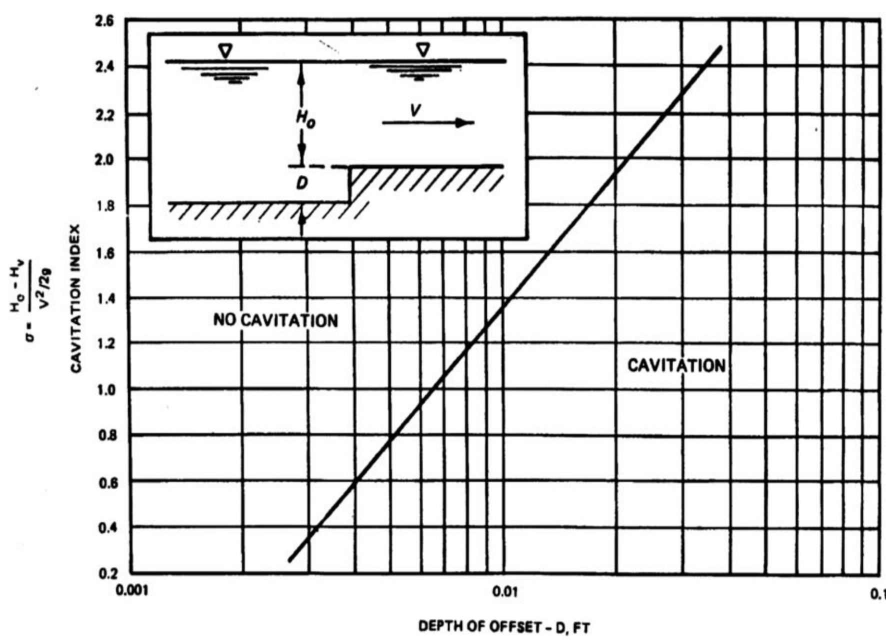
$$\frac{H_v - H_0}{\frac{V_0^2}{2g}} = 1 - \frac{\frac{V_v^2}{2g}}{\frac{V_0^2}{2g}} + \frac{Z_0 - Z_v}{\frac{V_0^2}{2g}}$$

for high velocities the elevation term on the right may be ignored

$$\frac{H_v - H_0}{\frac{V_0^2}{2g}} = 1 - \frac{V_v^2}{V_0^2}$$

finally the Cavitation Index, $\sigma = \frac{H_v - H_0}{\frac{V_0^2}{2g}}$ (note that the terms have been rearranged to yield a positive number).

The following graph (COE 1992) provides a guide to the cavitation potential of a "edge offset into flow" type of surface irregularity. For example, if the actual head above a weir crest is 8 feet and the velocity is 35 fps, the σ is 0.41. From COE plate 2-6 for an "edge offset into flow", an absolute pressure for vaporization of 0.26 psia (sea level at 60^0 F) cavitation should occur at an edge offset of 0.0032 feet.



CAVITATION CHARACTERISTICS - SQUARE EDGE OFFSET INTO FLOW

APPENDIX F - OPTIMIZATION OF OFFLINE SYSTEMS

Optimization of the choke

In all cases, the water surface in the main channel needs to be lifted high enough to overtop the side-weir. The most structurally economic type of choke is the classic open-channel transition. However, hydraulically this type of choke begins to lift the water higher at lower discharges and causes earlier flows to enter the basin and thus depleting some of its storage capacity. A better approach is to construct a RCB at the same width as the channel. The height of the RCB, then, determines when the flows begin to be choked and rise high enough to spill over the side weir. This positively results in passing the early portion of the hydrograph, without causing early side weir flows into the basin and reducing its available storage.

Optimization of the weir

In general, to convey the same discharge over a long weir as that over a short weir, the weir length would need to be increased. The advantage of a high (and long) weir is that early hydrograph flows would not enter the basin and deplete its storage prior to the peak arrival. However, the disadvantage is that the long weir is more costly due to the need for construction of a long splash pad that receives weir flows into the basin. Another disadvantage is that the weir may not be effective for lower frequency storms.

Optimization of the outlet

Whenever possible, the outlet slope should be steep. This ensures more efficient conveyance through the outlet. Further, at low flows, the velocity within the outlet should be high enough to perform self-flushing (cleaning) action. In the event that the outlet delivers the basin's water to an unimproved surface, then a mild slope could be considered to slow down the velocity at low flows.

Optimization of the channel d/s of choke

To eliminate (or reduce) the reduction in basin capacity due to reverse-flows through the outlet, flap-gates may be used. However, a better option is to include a supercritical short channel segment downstream of the choke, at the outlet. This will reduce the depths and reduce the likelihood of reverse-flows. Another, even more significant, benefit is that the basin would be emptied much faster due to the lower tail water control for its outlet.

APPENDIX G – GENERAL SUGGESTIONS ON USE OF BAS

Copying hydrographs from one project file to another

To copy hydrographs from one BAS project file to another, follow the following steps:

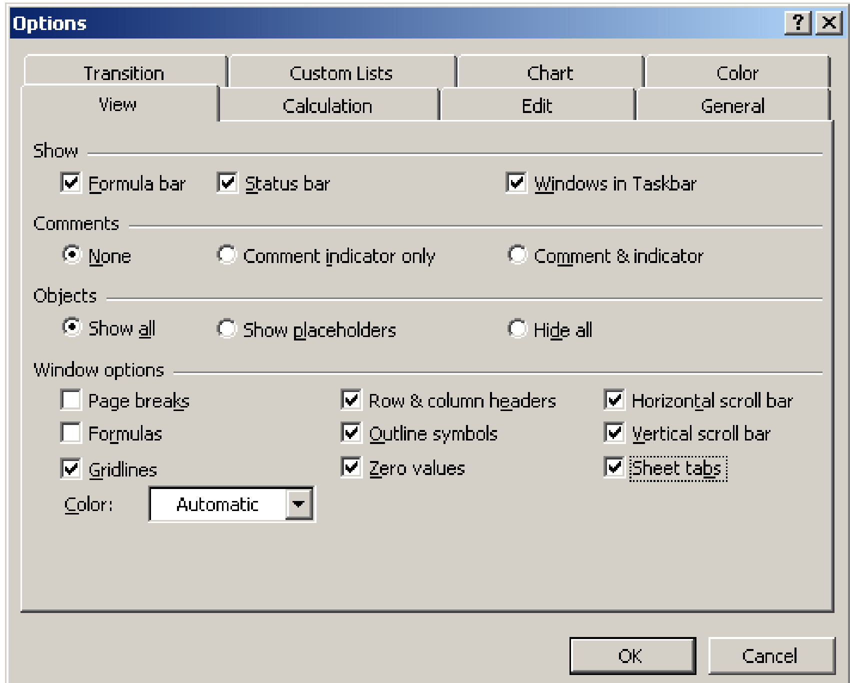
1. Close any open project file and exit BAS

2. Activate Excel and go to the directory which contains the source hydrograph **(Project A)**

3. Open **Project A** by double clicking the file

4. It is likely that Excel **Sheet tabs** (at bottom) are not displayed. Also, it is likely that the **Vertical and Horizontal Scroll** bars and **Row & column headers** are also not displayed.

5. Display the **Sheet tabs**, the **Vertical and Horizontal Scroll** bars and **Row & column headers** by going to (see figure below) **Tools/Options** and selecting those options.

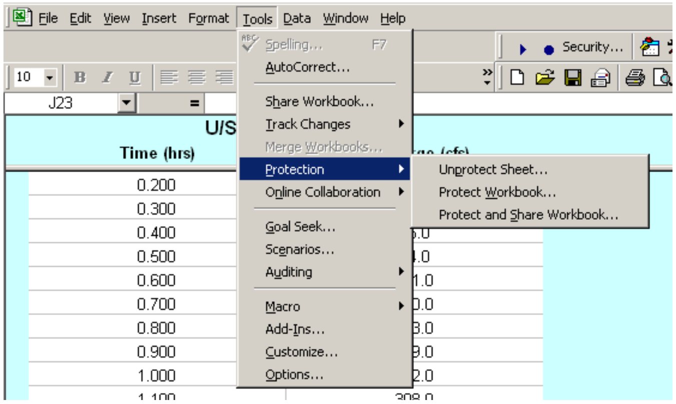


6. Now that the **Sheet tabs** are shown, press the **Sheet tabs** arrow at the lower left-hand corner to display the **Hydrograph** sheet. Then, press the Hydrograph sheet tab. You may or may not be at the hydrograph location within that sheet. Therefore, slide the **Horizontal Scroll** bar (bottom right of screen) all the way to the left. The hydrograph should be displayed now.

7. Now it is time to open the project file that is to receive the hydrograph in **Project A**. We'll call this **Project B**.

8. Open **Project B** and display the **Row & column headers**, **Sheet tabs** and **Vertical and Horizontal Scroll** bars per the above instructions.

9. BAS project files are usually sheet-protected. Therefore, you'll not be able to modify or add data within a sheet without un-protecting that sheet. To do so, go to **Tools/Protection** and select **Unprotect Sheet**.



10. Now you are ready to copy and paste the hydrograph. Return to **Project A** by pressing **Window/Project A.** Highlight the whole hydrograph (both columns. Data only), right click and select **Copy**.

11. Return to **Project B** by pressing **Window/Project B.** First you need to clear all the hydrograph values in **Project B**. Do so by selecting all the existing data and right-clicking the mouse. Choose **Clear Contents**.

12. Place the cursor on the first Hydrograph-Time cell , right-click then **Paste**. The hydrograph should now be copied completely into **project B**.

13. Save the modified project file and exit. Note: you do not need to return the project file to the same settings that existed when you opened the file. Those will be automatically accomplished the next time you open the project file.

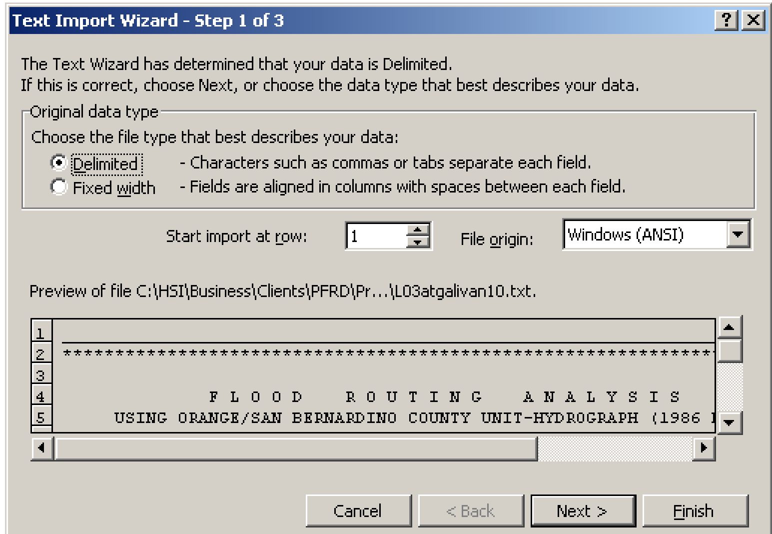
Copying hydrographs from a none - Excel data file

To copy hydrographs from a none-Excel data file such as files with .txt extensions, follow the following steps:

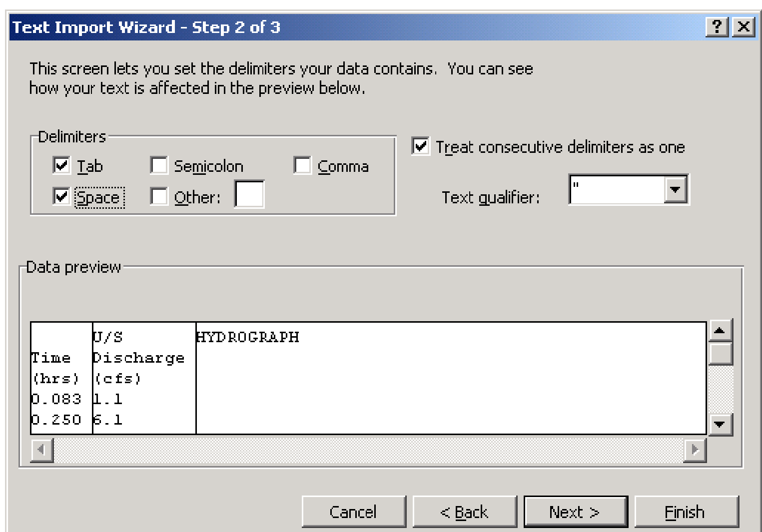
1. Close any open project file and exit BAS

2. Activate Excel and choose **File/Open** and select the file from its home directory.

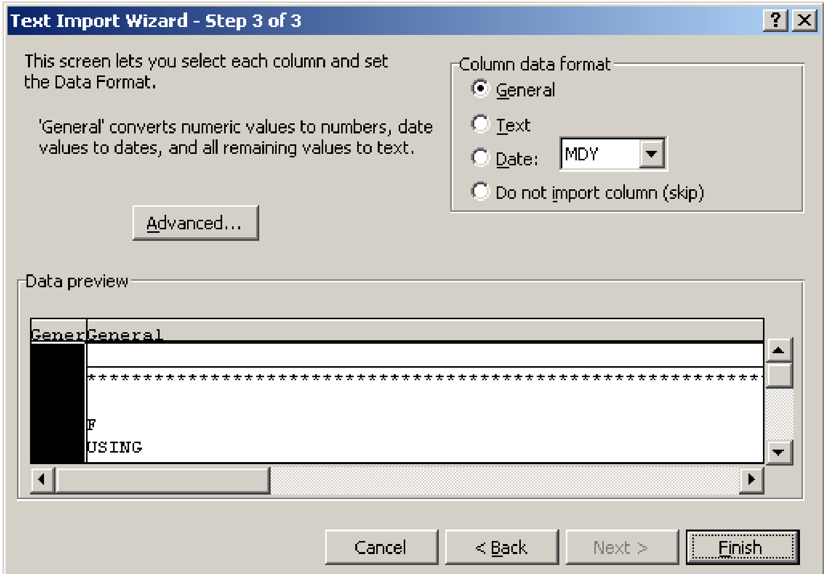
3. The following form will be displayed:



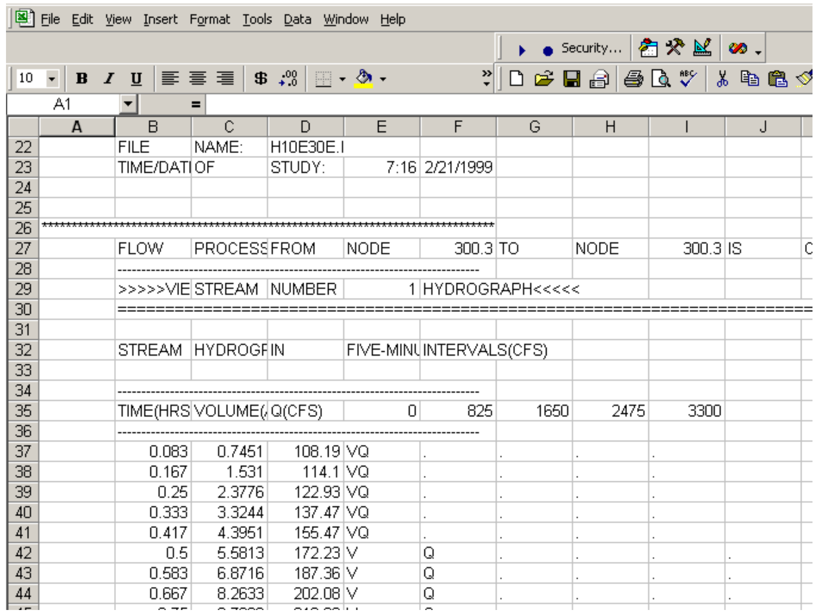
4. Press **Next**. The following form will be displayed:



5. Select **Space** and **Tab** then press **Next**. The following form will be displayed:

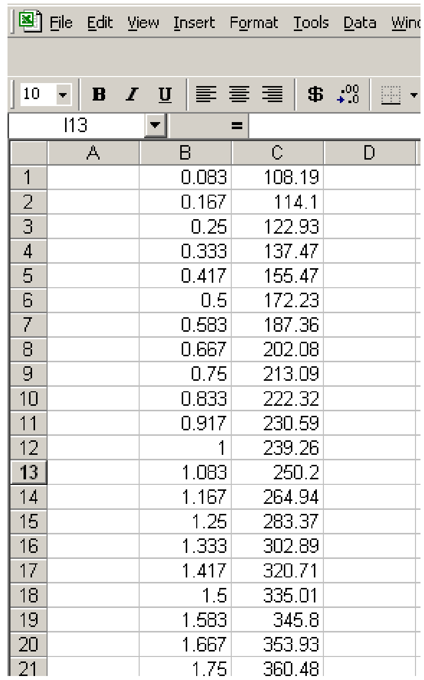


6. Press Finish. You will now have all the data in that file in Excel format. You'll need to clean up the information by deleting all the data and values that are not associated with the hydrograph. The following figure is a typical **.txt** file that was opened in Excel.



7. Delete all rows and columns that are not hydrograph values.

8. Make sure you end up with two adjacent columns. The left column is the Time and the right column is the Discharge. You should end up with columns similar to the figure below.



	A	B	C	D
1		0.083	108.19	
2		0.167	114.1	
3		0.25	122.93	
4		0.333	137.47	
5		0.417	155.47	
6		0.5	172.23	
7		0.583	187.36	
8		0.667	202.08	
9		0.75	213.09	
10		0.833	222.32	
11		0.917	230.59	
12		1	239.26	
13		1.083	250.2	
14		1.167	264.94	
15		1.25	283.37	
16		1.333	302.89	
17		1.417	320.71	
18		1.5	335.01	
19		1.583	345.8	
20		1.667	353.93	
21		1.75	360.48	

9. Now you are ready to copy the data into a BAS project file. To do so, please follow the instructions in the previous section of this appendix.

LIST OF SYMBOLS

A	Cross-sectional area of flow
b	Bottom width of channel
C_d	Side-weir coefficient of discharge
d	depth of flow
$\bar{d}$	Average depth of flow
d_c	Critical depth
d_h	Hydraulic depth
d_n	Normal depth
d_{ds}	Depth downstream of a Hydraulic Jump
d_{us}	Depth upstream of a Hydraulic Jump
E	Specific Energy
EGL	Energy Grade Line
F	Force
F_1	Froude No. at upstream of weir
F_r	Froude Number
F_{rds}	Froude Number downstream of a Hydraulic Jump
F_{rus}	Froude Number upstream of a Hydraulic Jump

f	Darcy-Weisbach friction coefficient
Ff	Force due to friction
g	Acceleration of gravity
h	Average depth of flow
γ	Specific Weight
GVF	Gradually-varied-flow
HD	Hydraulic Drop
HGL	Hydraulic Grade Line
HJ	Hydraulic Jump
h_l	Head loss
h_{lf}	Head loss (Major) due to friction
h_{lm}	Head loss (Minor) due to changes in direction and/or magnitude of velocity
Km	Minor loss coefficient
L	Length
M	Momentum
n	Manning's friction coefficient
P	Pressure
P_w	Wetted Perimeter
Q	Discharge

Q’	Side-weir lateral outflow intensity
R	Hydraulic Radius
RVF	Rapidly-varied-flow
S_0	Physical Slope of a channel
S_f	Slope of the Energy Grade Line
$\bar{S}_f$	Average friction slope
S_{HGL}	Slope of the Hydraulic Grade Line
T	Top width of a channel at the water surface
U	Longitudinal component of the lateral outflow velocity
V	Velocity
W	Weight or as defined in a particular section
y	Depth of flow or as defined in a particular section
z	Side slope of a trapezoidal channel (ratio of the horizontal to vertical)
$\bar{Z}$	Depth (measured from water surface) to the centroid of a flow area
ΔX	Spatial increment between cross-sections

REFERENCES

Ackers, P., “A Theoretical Consideration of Side Weirs as Storm-Water Overflow”, Proceedings, Institution of Civil Engineers, London, Vol. 6, Feb. 1957.

ASCE, The American Society of Civil Engineers, Stormwater Detention Outlet Control Structures, Task Committee on the Design of Outlet Control Structures., 1985.

Babbit, H. E., Sewerage and Sewage Treatment, John Wiley, New York, 7th Ed., 1953.

Brater & King, H, “Handbook of Hydraulics”, published by Mc Graw-Hill Book Company, 1963

Borghei, S. M. Jallili, M. R. and Ghodsian, M., “Discharge Coefficient for Sharp-Crested Side Weir in Subcritical Flow” Journal of the Hydraulics Division, ASCE, Vol. 125, No. 10, October, 1999.

Cheong, H. F., “Discharge coefficient of lateral diversion from trapezoidal channel,” Journal of Irrigation and Drainage Engineering, ASCE, Vol. 117, 1991.

Chow, V.T., Open Channel Hydraulics, McGraw-Hill Book Co., New York, 1959.

Coleman, G. S. and Smith, D., “The Discharge Capacity of Side Weirs,” Selected Engineering Paper No. 6, Institution of Civil Engineers, London, 1923.

Collinge, V. K. “The Discharge Capacity of Side Weirs”, Proceedings, Institution of Civil Engineers, London, Vol. 6, Feb. 1957.

COE, United States Department of the Army, Corps of Engineers, “Hydraulic Design of Spillways,” August, 1992

Coulomb, R., de Saint Martin, J.M, and Nougaro, J., “Etude theorique de la repartition du debit le long d’un deversoir lateral,” le Genie Civil, Vol. 144, 1967, 4, 311-316.

De Marchi, G., “Saggio de Teoria del Funzionamento degli Stramazzi Laterali” (Essay on the Performance of Lateral Weirs), L’Energia Eletrica, Milano, Vol. 11 Sept. 1934.

El-Khashab, A. and Smith, V. H., “Experimental Investigation of Flow over Side Weirs,” ,” Journal of the Hydraulics Division, ASCE, Vol. 102, No. HY9, Sept, 1976.

Engels, H., “Mitteilungen aus dem Dresdener Flussbau Laboratorium” (Report on the Dresden Hydraulics Laboratory), Forschungsarbeiten auf dem Gebiete des Ingenieurwessen, Berlin, #200 and 201, 1917.

Forchheimer, P., Grundriss der Hydraulik (Basics of Hydraulics), Leipzeg-Berlin, 1926.

Frazer, W. “The Behaviour of Side Weirs in Prismatic Channnels”, Proceedings, Institution of Civil Engineers, London, Vol. 6, Feb. 1957.

Gentilini, B., “Ricerche Sperimentali sugli Sfioratori Longitudinali” (Experimental Research on Side Weirs),), L’Energia Elettrica, Milano, Vol. 15 Sept. 1938.

Hager, W. H. and Volkert, P. U., “Distribution Channels,” Journal of the Hydraulics Division, ASCE, Vol. 112, No. 10, October, 1986.

Hager, W. H., “Lateral Outflow Over Side-Weirs,” Journal of the Hydraulics Division, ASCE, Vol. 113, No. 4, April, 1987

Hinds, J., “Side Channel Spillways, “ Transactions, American Society of Civil Engineers, Vol. 87, 1926.

Mostafa, M. G., “Current Theories and Needed Research for Canal Protection by Diversion of Excess Flood Waters to Retarding Basins”, Report prepared for Orange County Flood Control District, California, October 1972.

Mostafa, M. G. and Chu, H. L., “Experimental and Mathematical Investigation of Flow over Side-Weirs in Open Channels,” Engineering Research Center, CSULB, Report prepared for Orange County Flood Control District, California, June 1974.

Mostafa, G, “Hydraulics of Spatially-Varied Flow”, Engineering Workshop on Flow Reduction, Calif. State University-Long Beach, 1987

Nadesamoorthy, T., and Thompson, A., Discussion "Spatially Varied Flow over Side Weirs," by Subramanya, K. and Awasthy, S. C., Journal of the Hydraulics Division, ASCE, Vol. 98, No. 12, 1972.

Natsuhara, L. M., "A Case Study of Rapid Flow with Side-Weir," Thesis, California State University, Long Beach, December 1987.

Nimmo, W. H. R., "Side Spillways for Regulating Diversion Canals," Transactions, American Society of Civil Engineers, Vol. 92, 1928.

Parmley, W. C., "The Walworth Sewer, Cleveland, Ohio," Transactions, American Society of Civil Engineers, Vol. 55, 1905.

Ranga Raju, K. G., Prasad, B. and
Gupta, S. K., "Side Weir in Rectangular Channel," Journal of the Hydraulics Division, ASCE, Vol. 105, No. HY5, May, 1979.

Sing, R., Manivannan, D. and Satyanarayana, T., "Discharge Coefficient of Rectangular Side Weirs", Journal of Irrigation and Drainage Engineering, ASCE, Vol. 120, No. 4, July/August 1994.

Subramanya, K., and Awasthy, S. C., "Spatially Varied Flow Over Side Weirs," Journal of the Hydraulics Division, ASCE, Vol. 98, No. HY1, Proc. Paper 8627, Jan, 1972.

Swamee, P. K., Pathak, S. K., Mohan, M., Agrawal, S. K. and Ali, M. S., "Subcritical Flow over Rectangular Side Weir," Journal of Irrigation and Drainage Engineering, ASCE, Vol. 120, No. 1, January/February 1994.

Tults, H., "Flood Protection of Cnals by Lateral Spillways," Journal of the Hydraulics Division, ASCE, October 1956.

USBR, United States Department of the Interior, Bureau of Reclamation, "Design of Small Dams," Third Edition, 1987

Weisz, D. M., "Experimental Investigation of Flow over Side-Weirs in a Rectangular Open Channel," Thesis, California State University, Long Beach, December 1973.

SUBJECT INDEX

C

T

U

W