PANJ-AMU RIVER BASIN (AFGHANISTAN)

USER'S MANUAL FOR THE DESIGN OF A CROSS-REGULATOR AND A HEAD REGULATOR ON PERMEABLE FOUNDATIONS USING A MS EXCEL SPREADSHEET

Prepared by Claude de Patoul PhD, with funding from the European Union

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(Second edition revised and expanded)

Preamble of the Director General of Water Affairs Management, Ministry of Energy and Water

The Panj-Amu River Basin Programme (P-ARBP) is one of the key programmes for proper management of water resources through the introduction and implementation of Integrated Water Resources Management (IWRM).

The objective of the Programme, started in 2004 and carried-out jointly by the Ministry of Energy and Water (MEW), the European Union (EU) and Landell Mills (LM), is poverty and unemployment alleviation as well as food security through water security in the basin.

The implementation of such a programme requires the development of staff capacities, including the basin engineers and technicians regarding survey, design and construction of hydraulic infrastructure.

The User's Manual for the Design of a Cross-Regulator and a Head Regulator and the attached spreadsheet developed by Dr. Claude de Patoul, P-ARBP Team Leader from LM is a very valuable self contained manual. The practical approach makes it a useful guide to good practices for all concerned with the design and operation of hydraulic structures. It brings consistency and uniformity of approach to the design of the most common irrigation structures in new and existing canal systems of the Panj-Amu River Basin and the other river basins in Afghanistan.

Therefore, I recommend this manual to the engineers working in the design department of the River Basin Agencies and the Ministry of Energy and Water and other experts involved in water resources affairs of river basins of the country. The analysis, method and process described in this manual should be adopted for the design, rehabilitation and maintenance of hydraulic structures of irrigations systems in our country .

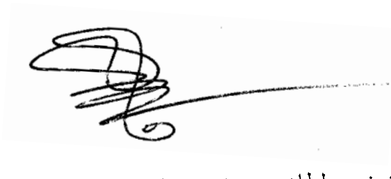
I am sincerely grateful to Dr. Claude de Patoul, Design Engineer/Team Leader of Panj Amu River Basin Programme (P-ARBP) for preparing this manual with the financial support given by the EU. We wish him further success.

Eng. Sultan Mahmoud Mahmoudi
General Director Water Affairs Management
Ministry of Energy & Water
Kabul

تقریظ:

پروگرام حوزه دریایی پنج-آمو (PARBP) یکی از برنامه های مهم جهت مدیریت مناسب منابع آب از طریق معرفی و تطبیق پروسه مدیریت همه جانبه منابع آبی (IWRM) با توجه به روش حوزه دریایی در حوزه دریایی پنج آمو به شمار میرود.
هدف این پروگرام که مشترکاً توسط وزارت انرژی و آب ، جامعه اروپا و کمپنی لندل میلز پیش برده میشود عبارت از کاهش فقر و بیکاری و مصئونیت غذایی به وسیله مصئونیت آب در حوزه مذکور بوده که در سال 1384 به فعالیت آغاز نمود.
تطبیق چنین پروگرام ایجاب می نماید تا ظرفیت کارکنان بشمول انجنیران و متخصصین فنی حوزه مذکور در بخش سروی ، دیزاین و امور ساختمانی تاسیسات آبی ارتقاء یابد. به همین سبب، کتاب رهنمود دیزاین سربند تنظیم کننده که محاسبات دیزاین در برنامه اکسل ضمیمه آن بوده ، یک رهنمود کامل و ارزشمند می باشد. روش های که در آن بکار رفته، قابل استفاده و رهنمای خوب، در امور دیزاین و امور عملیاتی ساختمان های هایدرولیکی می باشد. این کتاب، زمینه روش هماهنگ و همسان را در دیزاین ساختمان های معمولی آبیاری در سیستم های موجود و جدید حوزه دریایی پنج – آمو و سایر حوزه های دریایی کشور مهیا می سازد که از طرف داکتر کلود دی پتول (Dr. Claude de Patoul) تیم لیدر پروگرام حوزه دریایی پنج آمو از کمپنی لندل میلز، تهیه گردیده است.
این کتاب دارای اهمیت و ارزش خاص می باشد. بنابرین، مطالعه و کاربرد این رهنمود را برای انجنیران بخش دیزاین ادارات حوزه دریایی، وزارت انرژی و آب و سایر دست اندرکاران امور مدیریت منابع آب در حوزه های دریایی کشور سفارش می نمایم. امید است تحلیل ها، روش ها و پروسه های که در این رهنمود استفاده شده است در امور دیزاین، احیای مجدد و حفظ و مراقبت سیستم های آبیاری کشور در نظر گرفته شود.
بنابرین ، از آقای داکتر کلود دی پتول (Dr. Claude de Patoul) انجنیر دیزاین و تیم لیدر پروگرام حوزه دریایی پنج - آمو بخاطر تهیه این رهنمود که به کمک مالی اتحادیه اروپا صورت گرفته، قدردانی نموده؛ موفقیت هر چه بیشتر شان را در امور مربوطه خواهانم.

با احترام



انجنیر سلطان محمود محمودی
رئیس عمومی تنظیم امور آب
وزارت انرژی و آ

Foreword

The double purpose of this USER'S MANUAL FOR THE DESIGN OF A CROSS-REGULATOR AND A HEAD REGULATOR ON PERMEABLE FOUNDATIONS USING A MS EXCEL SPREADSHEET is to serve as a guide to good practices for all concerned with the design and operation of structures. It brings consistency and uniformity of approach to the design of the most common irrigation structures in the new and existing canal systems of the Panj-Amu River Basin and other river basins in Afghanistan.

This Manual presents instructions, standards and procedures for the selection and design of a cross-regulator and a gated head regulator. It is fairly self-contained and based on the publications listed in the references. It does not include the calculation of structure stability (shear etc.) of the structure which is dealt with in other publications.

To practically implement the technique, the author developed a stand-alone MS Excel spreadsheet computer programme that allows one to design and compute the flow rate through a complete check structure. This spreadsheet is an interactive computer program that generates all the necessary data for the design of a cross-regulator, a gated head regulator and its discharge calibration (vertical slide gates). A cross-regulator is used for passive control of the water level in the river or parent canal. The structure is normally applied for free flow only. The User's Manual considers only rectangular cross-sections to facilitate the analysis. In this manner, the author can develop the design procedure more fully for rectangular cross-sections, thereby providing a better basis for later investigations by others regarding other geometric sections (e.g., trapezoidal, triangular, or circular).

This application also provides simple graphs to help define dimensions and hydraulic properties of the structures. A worked out example of the design of a standard irrigation structure of the basin is presented in the attached MS Excel sheet. Never blindly enter data in the spreadsheet and accept the result at face value; one may get incorrect results and not even realise it.

The User's Manual is not intended to cover sophisticated or peculiar irrigation structures linked to particular technical problems and high discharges. In cases such as these, a different approach may be used with the help of specialists. Furthermore, the designer should not overlook the use of alternative methods, if seen to be more appropriate in peculiar situations.

This second revised and expanded edition has taken into account additional field experience and the remarks received during the training sessions and classroom courses given to participants and students. Although this spreadsheet has been extensively tested to eliminate errors and inaccuracies, the author cannot guarantee its suitability for any purpose. The river basin and sub-basin agencies or company's designers must ensure that safe and adequate designs are properly prepared when using the User's Manual. The use of this material in any manner whatsoever shall only be done with competent professional assistance. The author provides no expressed or implied warranty that this material is suitable for any specific purpose or project and shall not be liable for any damages including but not limited to direct, indirect, incidental, punitive and consequential damaged alleged from the use of this Manual.

Any comments which may lead to improve the next edition of this Manual are welcomed and can be addressed to inibreh@hotmail.com.

Claude de Patoul, PhD
Team leader
Panj-Amu River Basin Programme

Contents

PART I:

WATER FLOW AND DESIGN APPROACH

1. FLOW CLASSIFICATION

1.1. INTRODUCTION

In the User's Manual, a channel is a natural conduit with a free surface open to the atmosphere and a canal is a water human-made or artificial conduit with a free surface open to the atmosphere.

However, in engineering practice, the words are interchangeable. Depending on the circumstances, they also receives different name such as channel, chute, aqueduct etc. These names are used fairly loosely and can be defined only in a very general manner.

1.2. JARGON AND CRITERIA CONSIDERED

The flows are classified according to the variation in the parameters of flow in relation to space and time. The state of flow is classified according to the range of the invariants of flow in relation to (dynamic) viscosity (friction force which exists inside a fluid as it flows) and gravity in the direction of flow.

1.2.1. **Normal flow**

The normal flow is the flow where it wants to be with friction forces exactly balancing gravitational acceleration.

1.2.2. **Steady and unsteady flow**

The criterion considered is time. The flow is said to be steady if the fluid properties (depth of flow, velocity, density and discharge) don't change during the time interval under consideration. The flow is unsteady if one of the properties (the depth of flow etc.) changes with time.

1.2.3. **Uniform and non uniform flow**

The criterion considered is space (or depth) ($\delta y/\delta x = 0$). The flow is said to be uniform if the depth of flow is the same at every section of the canal or channel[1]. In other words, there is a balance between the frictional loss and drop in elevation of the channel: the friction slope S_f is equal to the bottom slope S_0 when the head loss h_L is equal to the elevation drop, that is $S_f = S_0$ when $h_L = Z_1 - Z_2$.

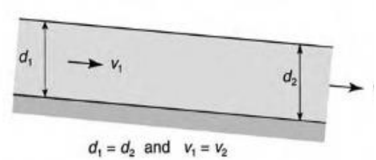


Figure 1.1: *uniform flow*

For a given roughness coefficient (resistance to flow), discharge and slope, there is only one possible depth for maintaining a uniform flow (empirical Manning´s equation). This flow depth is the normal depth which corresponds

[1] A uniform flow may be steady or unsteady, depending on whether or not the depth changes with time.

to the minimum energy level. The water will always to try to flow at normal depth.

The basic requirement of a non uniform flow is the slope of the water surface and the slope of the bed are different.

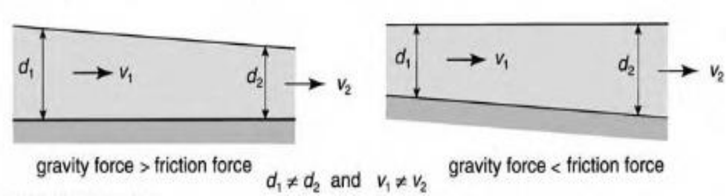


Figure 1.2: *non uniform flow*

- Gradually varied flow ($\delta y/\delta x < 1$): uniform flow whose water depth varies gradually along a <u>long distance</u> in space with stream wise distance because of an imbalance between gravitational forces and friction forces. This may occur as the result of a change in channel conditions (slope, cross-section or roughness) or as an adjustment brought about by u/s or d/s disturbances[1]. Because the variation is gradual, the flow can still be treated as one-dimensional (varying only with space) and the pressure[2] as hydrostatic Furthermore, the u/s and d/s specific energy are still assumed as equal.
- Rapidly varied flow ($\delta y/\delta x > 1$): the rapidly varied flow has a very pronounced curvature of the streamlines and all the parameters are modified in a very <u>short distance</u>. The change of curvature may be so pronounced that the flow profile is virtually broken (jump). The u/s and d/s specific energy are no more assumed as equal ($E_1 \neq E_2$) but the u/s and d/s specific force are the same ($M_1 = M_2$) (refer to figures 9.1a and 9.1b).

[1] Backwater curve if depth of flow increases in the direction of flow; drawdown curve if decreases in the direction of flow.

[2] Engineers often refer to pressure in terms of metres of water rather than as a pressure in kN/m^2. They can do this because of the unique relationship between pressure p and water depth h ($p=\rho*g*h$). It is called the pressure head or just head and is measured in metres using the pressure-head equation.

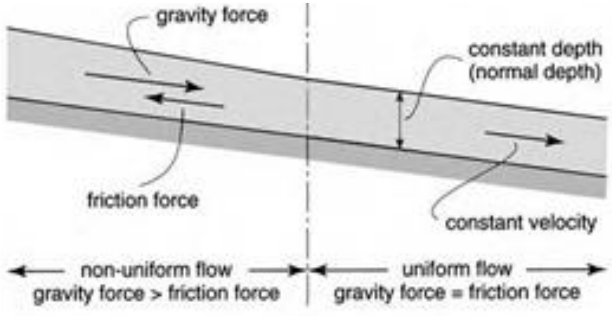


Figure 1.3: *non uniform and uniform flow forces*

1.2.4. Laminar, transitional and turbulent flow

The criterion considered is the <u>viscous forces</u> and the gravity relative to the interne inertia of the flow (Reynolds number)[1].

This type of flow rarely exists in nature and so is not of great practical concern in hydraulics; only turbulent flow is considered in this Manual.

1.2.5. Critical, sub and super-critical flow

The criterion is <u>gravity</u> whose effect is represented by a ratio of inertial forces to gravitational force and given by the number F_r (refer to article 10.3.3).

A small surface wave occurs when the flow in the open channel is disturbed. Sub-critical[2] flow occurs if the small surface wave can propagate u/s as well as d/s. The velocity of flow is less than that of the wave propagation. It will lead to a backwater zone of great distances before the obstacles[3].

If the wave can only propagate d/s and cannot propagate against the flow, this flow is called super-critical[4] flow, in which the flow velocity is greater than that of the wave propagation (wave celerity). The backwater zone occurs only around the obstacles

If the velocity of the small surface wave propagating d/s is zero, it is just the critical situation to distinct the super-critical flow and the sub-critical flow. This flow is called the critical flow.

1.3. EQUATIONS

1.3.1. Basic equations

[1] The laminar flow is not considered in this manual due to the low water kinematic viscosity.
[2] Sub because y_c is below the flow depth considered.
[3] Backwater and draw-down curves are formed where flow is influenced by cross-regulators, drop structures, gate and measuring flumes. The water surface gradually rises when flow is backed-up with at the same time a diminishing of the flow velocity. A draw-down profile gradually reduces water depth while the flow velocity gradually increases.
[4] Super because y_c is above the flow depth considered.

The hydraulic theory is based on three basic equations:

- Continuity equation: links u/s and d/s discharges which means also areas ad velocity.
- Energy equation: links pressure force (head, kinetic and potential) to velocity.
- Momentum equation: links force (mass) and velocity.

1.3.2. **Flow classification**

The empirical Manning's equation is used for free flow in open channels. It cannot be used where obstacles such as gates, weirs etc. are present.

In order to analyse situations where obstructions are to be considered, Bernoulli's energy equation must instead be used in certain conditions[1]. For Bernoulli's energy equation to hold, the streamlines must be parallel (usually applicable to steady flow). In certain flow phenomena, however, we simply can no longer ignore the mechanical energy losses due to the canal banks or such as in a hydraulic jump (where streamlines are curved) and we must look to the momentum principle as an alternative ways of describing the flow[2] (momentum (kgm/s) = mass (kg) * velocity (m/s)).

Some described flow classifications are shown in the following figures:

[1] In the Bernoulli's energy equation, the solutions depend upon the assumption that the total energy head at the u/s section should be equal to the total energy head at the d/s section plus the loss of energy between the two sections: the mechanical energy is conserved (**principle of mechanical energy conservation**). In fluid mechanics, it is found convenient to separate mechanical energy from thermal energy and to consider the conversion of mechanical energy to thermal energy as a result of frictional effects as mechanical energy loss. Then the energy equation becomes the mechanical energy: the sum of the gravitational (elevation Z), potential and kinetic energies. If energy actually leaks from the system via frictional head loss, the Bernoulli equation will overstate the energy available to the flow and the related predictions of velocity and depth will proportionately be in error.

[2] The momentum principle is an alternative way of describing the flow (principle of u/s and d/s forces corresponding to mass flow rate times velocity conservation). The momentum equation is brought to bear on problems involving high internal energy changes such as the task of describing the hydraulic jump where the flow is far less than perfectly energy conservative. If the energy equation is applied to such problems, the unknown internal energy loss ΔE is indeterminate and the omission of this term would result in considerable errors (refer to article 9.3). Indeed, all real flows in nature dissipate energy in overcoming frictional resistance. If we want to use Bernoulli's equation, we minimize this error by considering only short reaches of channel and only gradual transitions in order to ignore these energy losses. However, the energy equation is similar to the momentum equation when applied to certain flow phenomena such as the gradually varied flow and the two principles will produce practically identical results.

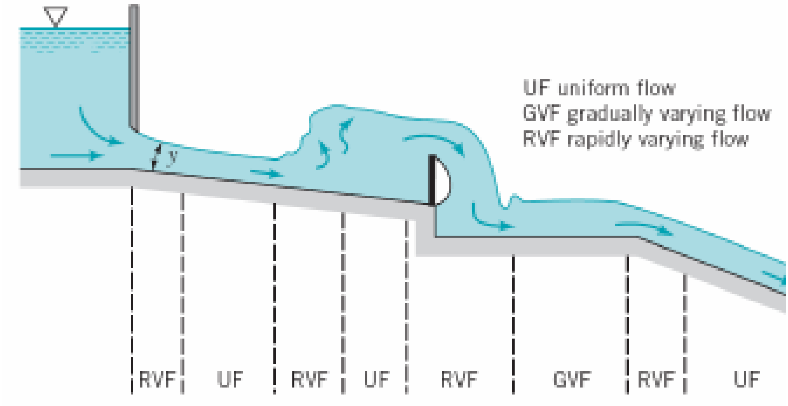


Figure 1.4: *flows classification downstream a sluice gate*

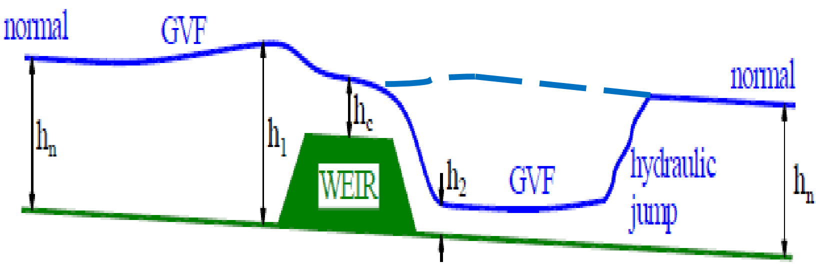


Figure 1.5: *flows classification with the presence of a weir*

If the sill level of the weir rises sufficiently, then, as the specific energy cannot be less than the critical specific energy E_c, the u/s flow must back up, increasing the depth and the energy in the sub-critical flow immediately u/s the weir, as pictured in the above figure (refer to table 14.3 and figures 25.1a & 25.1b).

Once critical conditions are established over the weir, it can be used as a flow-metering devises (refer to article 10.3.1).

2. HYDRAULIC STRUCTURE FOR FLOW CONTROL AND DISTRIBUTION

In the User's Manual, the head work consists of two modules: a passive water level regulator in the parent canal or in the river (usually an ungated broad crested weir) and a manual gated head regulator in the canal (off-take or intake).

The head regulator is of the gated breast wall type while the structure is of the open channel type. Cross-regulator (usually ungated) is of the open channel type. The head regulator performs two functions at the same time: flow discharge measurement function and water level control function.

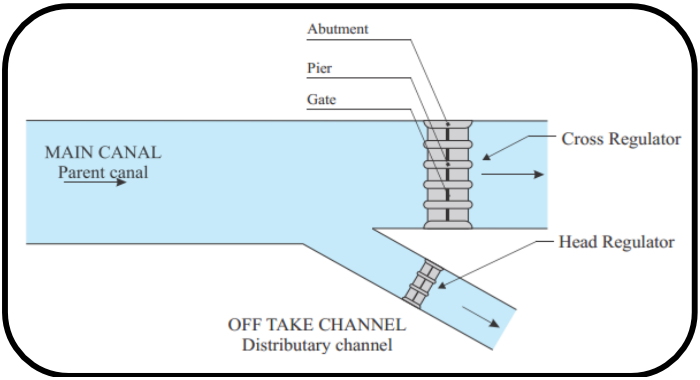


Figure 2.1: *plan of head work with cross-regulator and head-regulator (off-take) modules*

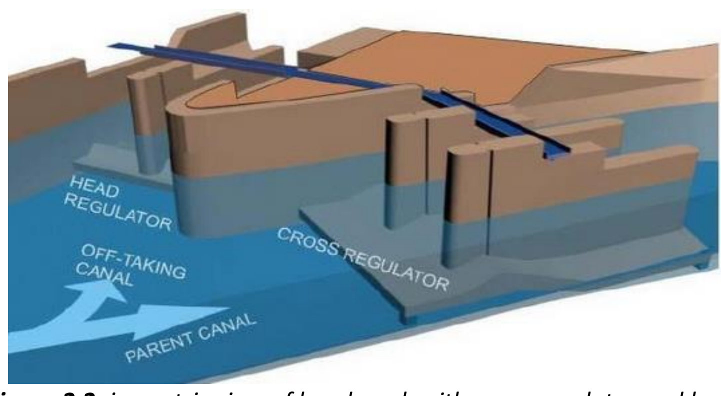


Figure 2.2: *isometric view of head work with cross-regulator and head regulator (off-take) modules*

From u/s to d/s, the different parts of the hydraulic structures are as follows:

- U/s transitional structure to guide the flow from the channel into the structure.
- Scour protective works u/s of the structure.
- U/s protected transition to guide the flow from the channel into the structure.
- Water level (head) control section (broad crested weir).
- D/s conveyance structure (sloping glacis).
- Energy dissipation reach (USBR type of stilling basin).
- Scour protective works d/s of the structure.
- D/s transitional structure to guide the flow from the structure into the channel.

The locating the main head work along a stream has to be carefully planed. The best location is on the outside of a river bend as shown in the following figure. If located on the inside of a band, it will be continually silted up as a result of the actions of secondary flows in the river.

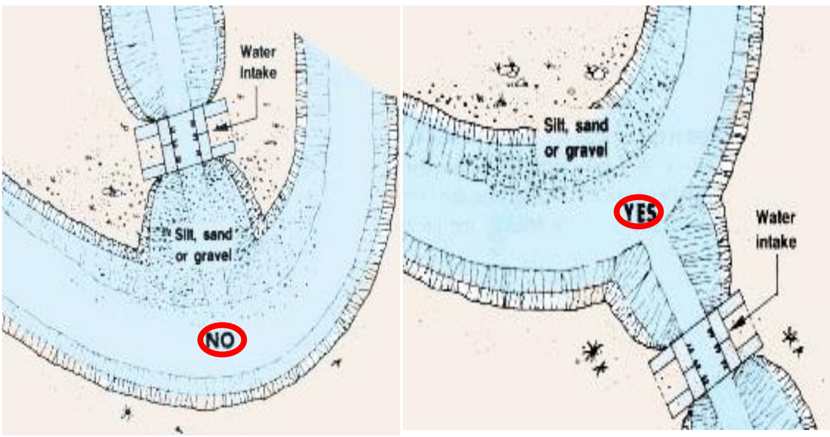


Figure 2.3: *view of head work location along a river*

Figure 2.4: *bird view of Selaba head work with ungated cross-regulator and rating curve on the right and gated head regulator with canal hydrograph and corresponding gate(s) openings on the left (Yatim Tepa canal)*

3. HYDRAULIC AND STRUCTURAL DESIGN

3.1. LONGITUDINAL AND CROSS-SECTION

The sections of a cross-regulator structure are given below (refer to figure 14.6):

1. Section A: converging flow.
2. Section C-C: water level (head) control.
3. Section 1-1: super-critical flow.
4. Section 2-2: energy dissipation.
5. Section 3-3: super-critical flow.

In this User's Manual, we consider the structure with a rectangular cross-section.

3.2. HYDRAULIC DESIGN

The hydraulic design comprises of fixing the overall dimensions and profiles of the structure through analytical equations and available empirical formulas[1].

The hydraulic design determines the (rectangular) cross-sections of the structure, the total mechanical energy of water (expressed as the total head in metre of water), the permissible velocities, the cut-off wall (refer to glossary) considered u/s and d/s of the structure, the glacis, the type of stilling basin and the u/s and d/s protected transition works.

The hydraulic design of the structure involves two sets of hydraulic conditions for sub-surface flow and for surface flow.

3.2.1. Hydraulic design for sub-surface flow

The sub-surface or seepage flow occurs due to the hydrostatic head H_S difference from the u/s to the d/s sections of the structure. It includes piping and the associated sand boil phenomenon.

In this User's Manual, the effects of the sub-surface flow on the stability of the hydraulic structure are computed with the Khosla's empirical method of independent variables which determines the strength of the up-thrust (uplift) pressure.

3.2.2. Hydraulic design for surface flow

In certain conditions, the dynamic action of the water flow may create the formation of the hydraulic jump causing an uplift unbalanced head in the jump basin which may be larger than the sub-soil seepage.

3.3. STRUCTURAL DESIGN

[1] If necessary, the dimensions set by the hydraulic design can be further refined by testing a scale model of the structure in an appropriate laboratory.

The structural design is the dimensioning of the super-structures and sub-structures and the manner in which the load bearing (structural) members of a hydraulic structure support each other in sharing the load (stress). The various parts of the structure are analysed for stresses under different loads and reinforcement[1] or other structural details are worked out. The stability analysis is not carried out in details in this Manual for the type of structures considered and can be found in most hydraulic books.

In the User's Manual, larger concrete head regulator (also called discharge regulator) are divided in 3 parts to allow movement due to settlement[2].

3.4. DESIGN CONDITIONS FOR GATES

In the design of the hydraulic structures, the following criteria have to be taken into account for the gates in closed position:

- The sliding gates for the head regulators (usually breast wall type) are designed to withstand and operate against the full unbalanced head, with the u/s water level above the gate crest level with no water d/s.
- The sliding gates for the structures (open channel type) are designed to withstand and operate against the full unbalanced head, with the u/s water level at the top of the gate with no water d/s.
- The cross-regulator sliding gates (open channel type) are designed to withstand and operate against a head of water 0.30 m above the top of the gate assuming no water downstream. The gates are ether the open channel type or breast wall type. Since the open channel type is subject to overtopping in times of flood they shall be designed, for safety reasons, as it were of the breast wall type, thus giving the worst case loading condition.

[1] The amount of rebar used in typical structures is a small percentage of the amount of concrete; mostly, as a rule of thumb, we use about 1% to 0.5 % rebar for carrying the tension forces in bending. Columns may use up to 6% rebar, partly because the rebar carries both tension and axial forces. Since rebar costs much more than concrete, efficient engineering design minimizes rebar use. In order for reinforcing bars to be in the required location in reinforced concrete, the bars must often be fabricated to special shapes. Shop drawings take the schematic information from the structural drawing and show the actual bar lengths, bends, clearances, etc. For a rough estimation of the reinforcement steel in construction, the following thumb rules may be adopted:

Construction	Unit	Quantity
Slab	kg/m^3 of concrete	50 to 80
Lintel	kg/m^3 of concrete	80
Beam	kg/m^3 of concrete	100 to 150
Column	kg/m^3 of concrete	150 to 225
Footing slab	kg/m^3 of concrete	80

[2] An expansion joint of 3 cm between the parts is made watertight by a 3-bulb rubber or PVC water stop cast in both concrete ends.

3.5. RIVER TRAINING WORKS

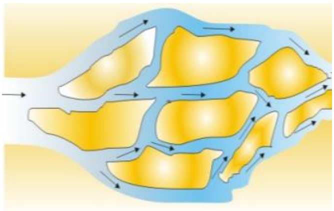


***Figure 3.1**: braided river*

River training works are necessary to achieve as follows:

- Prevent out flanking of the structure.
- Prevent u/s land flooding by the river.
- Provide favorable curvature of flow u/s of the structure
- Guide the river to flow axially through a structure.

Channel migration may occur naturally or as a result of human activity, and may be associated with any of the causes that give rise to degradation and aggradation. Migration of the entire river channel as part of the process of meander progression, or movement of the deep-water channel within the same overall channel banks, can affect the scour exposure of a structure whose foundations may have been set in relation to an earlier channel position. In some cases, migration may occur rapidly in response to a particular flood event, but in other cases it may be gradual. In a braided river, the channel positions are continuously changing, as shown in the opposite picture.

As a general rule, if there is potential for channel migration, the foundations should be designed or assessed on the basis of any credible shifts of the deep-water channel or channels. Alternatively, training works may be carried out to limit the possible movement of the deep-water channel.

At many instances, it is necessary to guide and/or restrict the course of the river through the hydraulic structure and it is achieved by the use of river training works.

4. GENERAL FIELD DATA

4.1. SITE INVESTIGATION

The general layout of the structure is mainly influenced by the topographical features, the approach conditions of the river or the parent canal and environmental considerations.

The topographic study of the flood plain allows defining an average cross-section of the river or parent canal and obtaining several field data essential for the design of the hydraulic works, such as the bottom slope of the river, the height of the banks, the type of material from the river bed etc.

The irregular river cross section should be simplified to a representative trapezoidal section for simplifying calculations as shown in the following figure:

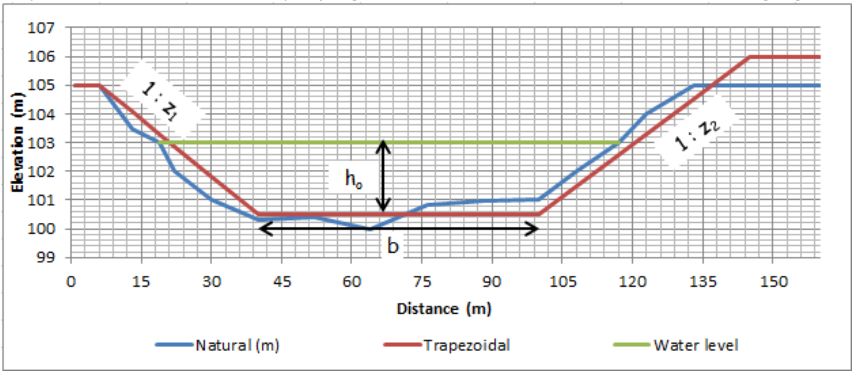


Figure 4.1: *irregular river cross section and trapezoidal cross-section*

The d/s reach should be investigated if the hydraulic structure causes flood risks (backwater curve) which would means additional embankments.

4.2. MAIN FIELD DATA

The main field data to be collected from the site for the design of the structure are listed below.

1. Index map of the site.
2. Contour plan of the area.
3. Command area of canal (ha).
4. Discharge rates of:
    a. Flood or maximum discharge of river or parent canal Q_{max}[1].
    b. Minimum discharge in the river or parent canal Q_{min} of river.

[1] The flood discharge rate or design volumetric flow rate considered in the river has an annual exceedance probability (AEP) of 1 in 100.

c. Irrigation water need or water duty Q_{crop}. (irrigation water need) as shown in the following figure:

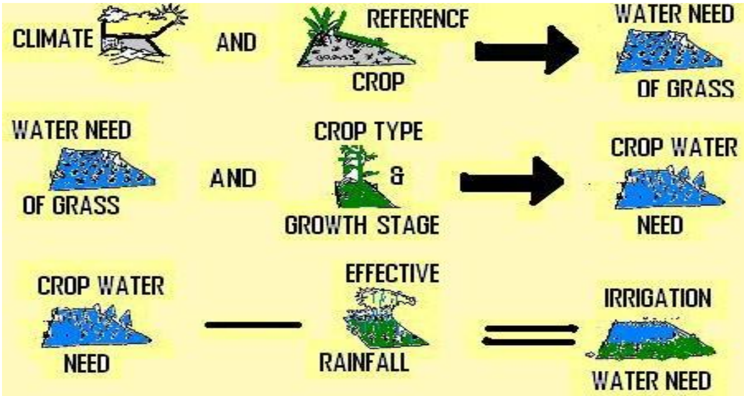


Figure 4.2: *climate to irrigation water need*

5. Cross-sections of the river or parent canal and off-taking canal.
6. Side slope of river or parent canal bank (H: V).
7. Upstream investigation for additional flood risk by backflow.
8. Test pits log chart and bed materials (d_{50} mean diameter).
9. Permeability coefficients and soil mechanical parameters.
10. Rainfall data of the location.
11. Location and accessibility of quarry areas for coarse and fine aggregates.

PART II:

HYDRAULICS OF THE CROSS-REGULATOR STRUCTURE

5. CROSS-REGULATOR (WEIR)

5.1. BROAD CRESTED WEIR

A cross-regulator or weir is defined as a barrier over which the water flows in an open channel. The edge or surface over which the water flows is called the crest or sill. The overflowing sheet of water is the nappe. If the nappe discharges into the air, the weir has a free discharge. If the discharge is partially under water, the weir is submerged or drowned.

In the User's Manual, a broad crested weir with a rectangular control section is usually adopted for its greater structural stability. The calculation of the rate of flow relies on the assumption that the fluid will pass through the critical depth somewhere near to its d/s edge, as shown in the following figure.

Furthermore, the crest is sufficiently long for parallel streamlines to be established (hence broad crested) to obtain parallel streamlines (refer to figure 6.2) but insufficiently long for significant frictional losses[1]. Consequently this makes the calculation of the discharge relatively straightforward.

In the User's Manual, a standard shape is used, then there is a large body of literature available relating to their design, operation and coefficient of discharge according to known relationships[2] (refer to article 6.4).

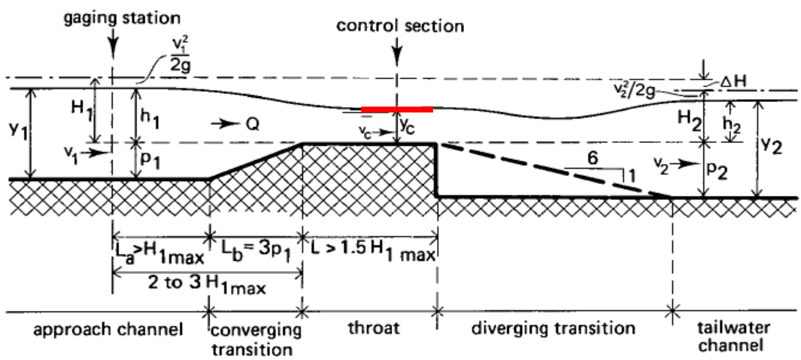


***Figure 5.1**: illustration of broad crested weir terminology*

5.2. WEIR ELEVATION

5.2.1. In the river or parent canal cross-regulator structure

[1] The disadvantage of a short crest is that it does not ensure a constant coefficient of discharge for varying heads, hence, the fall cannot be used as a meter; but it provides a higher coefficient of discharge and thus a greater discharging capacity.

[2] The ability to calibrate the structure using equations instead of measurements is based on the existence of parallel streamlines in the control section over the weir crest; nowhere above the short/sharp crest can the curvature of the streamlines be neglected.

One of the most important design parameters is the height of the crest above the channel bed. This height should be sufficient to provide modular flow for the entire range of discharges that the broad crested weir is intended to measure. In order to ensure critical flow over the cross-regulator for all flow conditions, the maximum anticipated flow rate through the channel should be used to calculate the required cross-regulator height. Furthermore, the crest height should at least cover the head losses at the intake and in the approach to the intake.

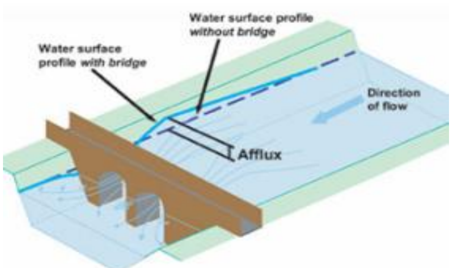


Figure 5.2: *afflux*

Afflux and discharge are related to the crest level. The operation of cross-regulators is often misunderstood and it is believed that they cause the flow to always back up and so raise water level u/s as shown with the presence of a bridge in the opposite figure. This only happens once critical conditions are achieved on the sill (refer to article 25.2). A lower crest level results in lesser afflux, in an increased height of gates and a decrease in floor thickness. By providing a high afflux, the width of the structure can be narrowed but the cost of training works will go up and the risk of failure by out flanking will increase. The crest level is also function of the required water surface elevation in the branching canal.

5.2.2. In the head regulator of the branching canal

In the User's Manual, the crest elevation is based on the lower water discharge rate in the river or parent canal (Q_{min}) and generally kept 0.1 m to 0.2 m higher than the crest level in the river or parent canal; it is also function of the size available and number of gates to be provided in the branching canal.

Furthermore, the crest elevation should at least cover the head losses at the intake and in the approach to the intake.

6. DISCHARGE AND ENERGY OVER A CROSS-REGULATOR

6.1. DISCHARGE RATE

The main cross-regulators sections and plans are summarized in the below sections.

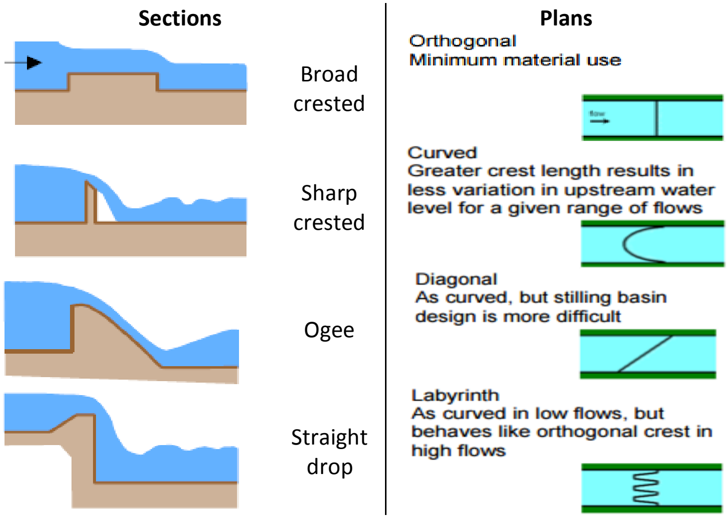


***Figure 6.1a**: sections and plans of cross-regulators*

In the User's Manual, a rectangular broad crested weir is usually used for passive control of the water levels in the river or parent canal.

6.1.1. **Flow rate equation**

The presence of the weir in the open channel introduces an area constriction to an otherwise uniform flow in a prismatic channel of mild slope (refer to article 25.2 and table 25.1). The ratio H_0/L (refer to article 6.4.2) indicates if the weir behaves as a broad crested weir or a short crested weir.

The head-discharge equations based on ideal flow must be corrected for energy losses, velocity distributions and streamline curvature by the introduction of a discharge coefficient C_e. The resulting head-discharge equation for a rectangular channel is as follows:

$$\boxed{Q = C_e * B_{weir} * H_0^{3/2}} \quad (6.1a)$$

Where:

Q = flow (discharge) rate (m^3/s).

C_e = effective discharge coefficient or weir coefficient.

Refer to list of symbol.

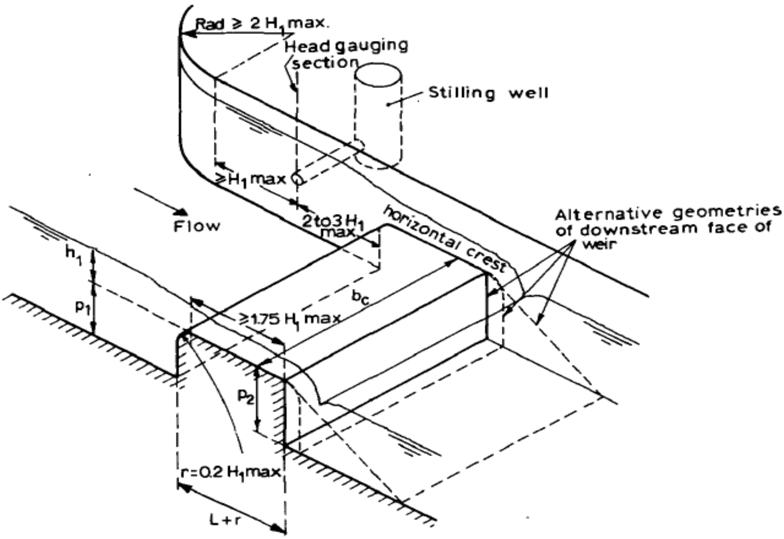


***Figure 6.1b**: illustration of broad crested weir terminology*

In an open channel, it is not possible to measure the energy head H_0 directly and it is therefore common practice to relate the flow rate to the u/s sill or crest referenced water head h_0 by introducing the velocity coefficient C_v.

$$\boxed{Q = C_e * B_{weir} * h_0^{3/2}} \quad (6.1b)$$

<u>Where</u>:
Q = flow (discharge) rate (m^3/s).
C_e = effective discharge coefficient or weir coefficient.
B_{weir} = overall width of the weir across the flow (m).
Refer to list of symbol.

In the case of an ungated suppressed weir, the overall width of the weir across the flow B_{weir} has the same value as the overall width of the structure between abutments $B_{structure}$ or as the clear waterway B_{cl} (refer to article 14.4 and table 14.2).

6.1.2. Effective discharge coefficient and streamlines at control section

When using the velocity coefficient C_v, the effective discharge coefficient is given by the following equation and depicted in the following figure:

$$\boxed{C_e = C_d * C_v * 2/3 * (2/3 * g)^{1/2}} \quad (6.1c)$$

Where:

Refer to list of symbols.

The discharge coefficient C_d, hence C_e, is affected by the friction on the channel wall and bottom between the gauging and the control section, by the velocity profile in the approach channel and control section and by the changes in pressure distribution caused by streamline curvature as shown in the below figure[1].

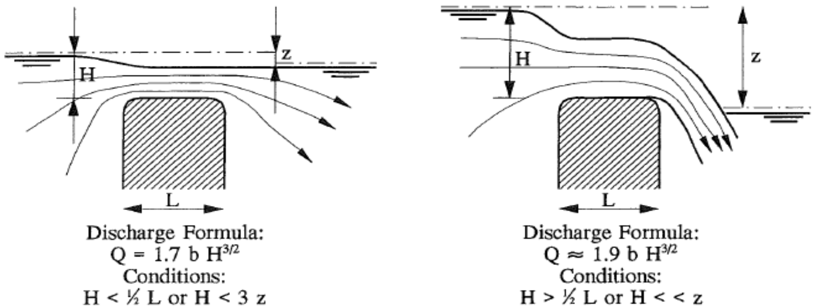


***Figure 6.2**: illustration of conditions of streamlines at control section affecting the discharge coefficient*

In equation 6.1b, the semi-empirical effective discharge coefficient (or weir coefficient) C_e equals in practice:

- 1.7 for rectangular broad crested weir state; C_d remains fairly constant[2] with a smooth and sufficiently long broad crest creating parallel streamlines at the water level control section (refer to figure 6.2 and article 6.4.2 on how to obtain it).
- 1.84 to 1.9 for short crested weir state; the streamlines (flow lines) at the control section are curved as shown in figure 6.2. Since the discharge coefficient C_d increases if the streamlines curvature at the control section increases, C_d should be multiplied by a corrector factor which is always greater than unity (varying between 1.1 and 1.3). This phenomenon explains the change of value of the effective discharge coefficient to approximately 1.9 (1.84 in the User's Manual) for short crested weir state (refer to article 6.4.2).

6.1.3. **Velocity coefficient**

[1] A minimum distance for the approach channel is necessary for the development of uniform and symmetric flow conditions and the establishment of a stable water surface (refer to article 6.4.1).

[2] Changes of C_d as a function of H_0 are usually insignificant.

In the field, it is not possible to measure the energy head H_0 directly and it is common practice to relate the discharge to the u/s water head h_0 as shown in equation 6.1.b. A positive velocity coefficient correction is therefore necessary for neglecting the velocity head $V_a^2/2g$ in the approach channel in order to have the true magnitude of C_e for the above equation 6.1b. C_v may be approximated from the following equation for a rectangular cross-section:

$$\boxed{C_v = (H_0 / h_0)^{1.5}} \quad (6.1d)$$

Where:

Refer to list of symbols.

Finally, C_v varies from 1.01 up to 1.30 and usually is equal to 1 with low approach velocity[1]. The cases given above show that traditional discharge equations are often a mixture of rational analysis and experimental coefficient evaluation.

6.2. UPSTREAM FLOW VELOCITY AND KINETIC ENERGY HEAD

6.2.1. Approach flow velocity

The velocity to be considered is the approach (or accelerated) velocity V_a at the entry of the structure (Manning's equation). For solving the problems in hydraulic engineering, the velocity used is the average velocity of flow over a section.

With a rigid section, the permissible limits of velocity are given in the following table:

Table 6.1: *Maximum permissible average velocities of water in canals*

Type of soil or lining	Max. permissible average velocity (m/s)
Unlined canal	
Soft clay or very fine clay	0.2
Very fine or very light pure sand	0.3
Very light loose sand or silt	0.4
Coarse sand or light sandy soil	0.5
Average sandy soil and good loam	0.7
Sandy loam, small gravel	0.8
Average loam or alluvial soil	0.9
Firm loam, clay loam	1.0
Firm gravel or clay	1.1

[1] During flood stages, the velocity varies greatly; hence the velocity head is usually included in the total energy head.

Stiff clay soil, ordinary gravel soil, or clay & gravel	1.4
Broken stone and clay	1.5
Coarse gravel, cobbles, shale	1.8
Conglomerates, cemented gravel, soft slate	2.0
Soft rock, rocks in layers, tough hardpan	2.4
Hard rock	4.0
Lined canal	
Cast-in-place cement concrete	2.5
Precast cement concrete	2.0
Stones	1.6-1.8
Cement blocks	1.6

However, the conveyance reach u/s of the weir should have a sub-critical flow as to limit the head loss (Froude number $F_r < 1$). The Froude number F_r can be calculated with the formula 10.3a or 10.3b (V_1 becomes the approach velocity V_a at the entrance of the structure; y_1 is the u/s water depth within the structure and B is the open-water width of the structure). In the User's Manual, it is often limited to < 0.5 (refer to article 6.4.1) to avoid standing waves in the conveyance reach. Hence, the recommended maximum approach velocity V_a at the entrance of the structure is given by the following equation:

$$\boxed{V_{a\ max} = 0.5\ (g * y)^{0.5}} \quad (6.2a)$$

Where:

$V_{a\ max}$ = maximum approach velocity at entrance of structure (m/s).

y = u/s water depth in the structure (m).

6.2.2. Kinetic energy head

When all parts of a system move with the same velocity, the kinetic energy per unit mass is expressed as:

$$\boxed{V_a^2 / 2g = (Q / A)^2 / 2g} \quad (6.2b)$$

Where:

$V_a^2/2g$ = kinetic energy head due to u/s velocity (m).

Q = low water discharge rate (m^3/s).

Refer to list of symbols.

6.2.3. Upstream total energy elevation HE_0

The elevation of the total u/s mechanical energy HE_0 is the sum of the u/s water depth in relation to the weir sill, the kinetic energy head due to u/s flow velocity and the crest elevation.

$$\boxed{HE_0 = \text{crest sill height} + V_a^2 / 2g + h_0} \quad (6.3)$$

Where:
Refer to list of symbols.

6.3. DOWNSTREAM FLOW VELOCITY AND KINETIC ENERGY HEAD

6.3.1. D/s elevation HE_2

$$\boxed{HE_2 = HE_0 - \Delta E} \quad (6.4)$$

Where:
Refer to list of symbols.

6.3.2. Kinetic energy head $V_3^2/2g$

$$\boxed{V_3^2 / 2g = (Q / A)^2 / 2g} \quad (6.5)$$

Where:
Q = low water flow rate (m^3/s).
Refer to list of symbols.

6.4. DISCHAGE EVALUATION AND FLOW STATE

Once critical conditions are established over the weir, it can be used as a flow-metering device.

6.4.1. Discharge measurement station

The approach of the User's Manual is to combine the water level control and the flow measurement functions by establishing the corresponding rating curves (refer to article 17).

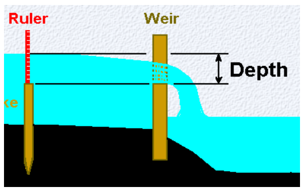


***Figure 6.3**: ruler location*

The energy head H_0 measurement (staff gauge or ruler) should be located at a distance between 2 or 3 times $H_{0\,max}$ u/s of the weir toe (refer also to figures 5.1 and 6.1). In the case of gates with piers (open channel or breast wall type), the distance is considered from the weir or the u/s nose of the pier whichever is located more upstream.

The Froude number should not exceed 0.5 at the staff gauge location at maximum flow. This is a primary design criterion intended to ensure that the water level can be measured with reasonable accuracy at the gauge and to avoid standing waves as the water surface and flow to become critical due to possible decreased canal roughness from the river or parent channel into the canal[1].

6.4.2. Flow state of gauging weir

The discharge coefficient C_d corrects for such phenomena as the energy loss between the gauging and the water level control section, the non uniformity of the velocity distribution and the streamline curvature in these two sections. These phenomena are closely related to the value of the ratio H_0/L.

In terms of the ratio H_0/L (refer to figures 6.1 & 6.2), four different flow states over the weir may be distinguished:

$H_0/L < 0.08$: over this range, the weir cannot be used as a measuring device.

- The depth of flow over the weir crest is such that sub-critical flow occurs above the crest.
- The water level control section is situated near the d/s edge of the weir crest.
- The energy lost through friction above the sill is a relatively large part of H_0 (the thin layer of water above the sill is very close to the rough boundary).
- C_d is determined by the resistance to flow characteristics of the crest surface (roughness coefficient).

$0.08 \le H_0/L \le 0.33$: over this range, the weir may be described as broad crested.

- A region of parallel flow occurs somewhere midway above the crest.
- The control section is located at the end of the section where parallel flow occurs.
- C_d has a constant value over this H_0/L-range, provided that the approach velocity V_a has no significant influence on the shape of the separation bubble

$0.33 < H_0/L < 1.5$ to 1.8: over this range, the weir should not be termed as broad crested but as short-crested[2].

- The two downward slopes of the water surface merge and parallel flow will not occur above the crest.
- Streamline curvature at the water level control section has a significant positive effect on the discharge, resulting in higher C_d values.

$H_0/L > 1.5$: over this range, the weir acts as a short crested weir

- The ratio H_0/L has such a high value that the nappe may separate completely from the crest.

[1] To eliminate this error, increase the size of the approach channel, reduce the control section width, or increase the height of the crest relative to the invert of the approach channel.

[2] Frequently, in this range of H_0/L values, the weir is also classified as broad crested.

6.5. FLUMING OF WATERWAY

The contraction of the waterway of the channel (i.e. fluming of the waterway) will reduce the open water width of the cross-regulator, which is likely to produce economy in many cases.

The maximum fluming is generally governed by the extent that the velocity in the trough that should remain sub-critical (refer to article 6.4.1).

However, the greater is the fluming, the greater is the length and height of the d/s transition wings and the length of the d/s transition wings.

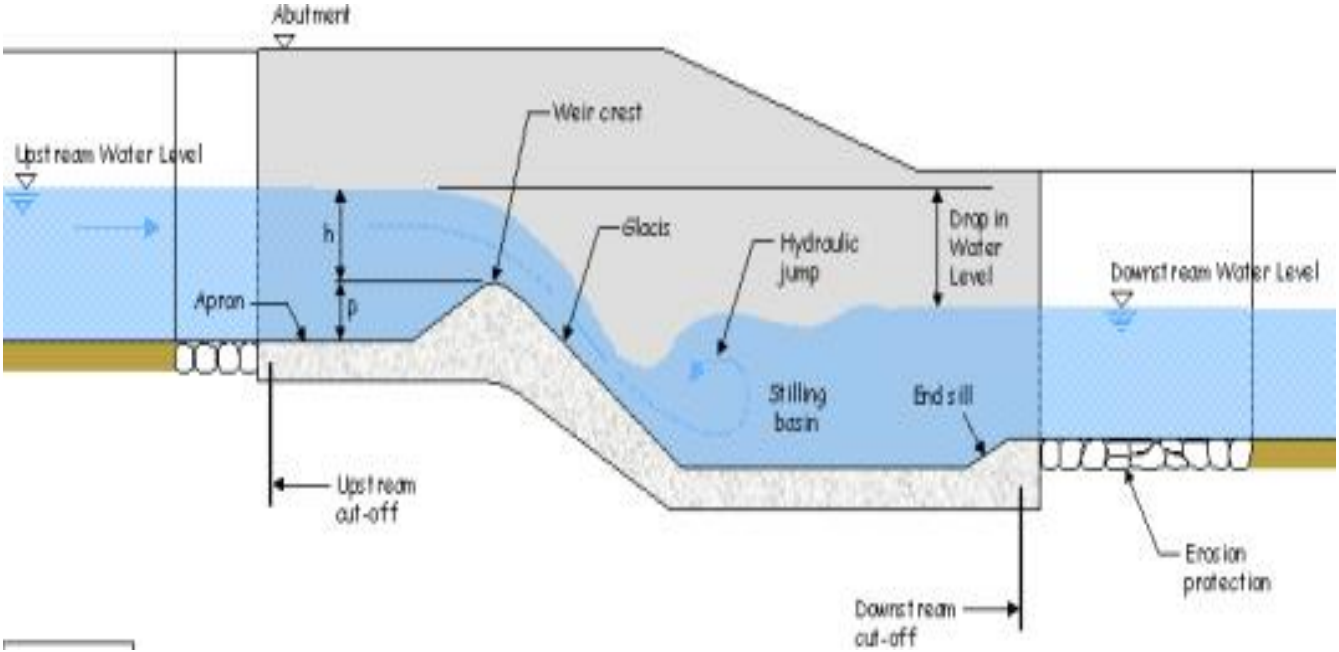


Figure 6.4: *illustration of a typical ungated cross-regulator structure with glacis*

7. MODULAR FLOW AND SUBMERGENCE RATIO

7.1. MODULAR FLOW

The following figure shows the different flows over a cross-regulator:

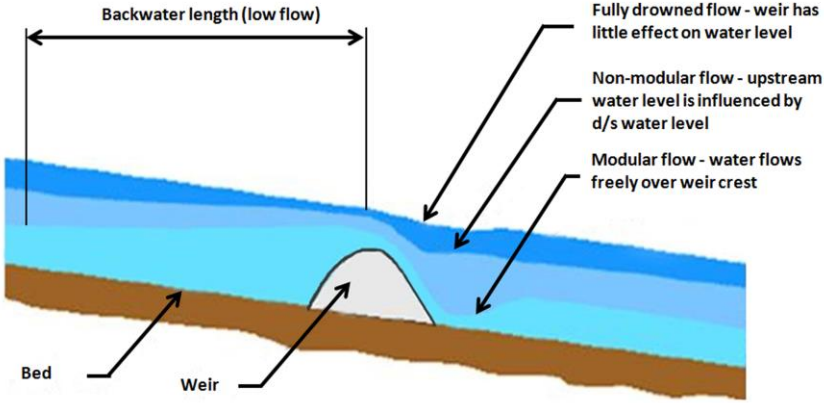


Figure 7.1: *modular and drowned (submerged) flow conditions*

The flow over a channel feature is modular when it is independent of variations in tailwater surface elevation. The flow is usually contracted and passes through a critical depth y_c. To maintain a modular flow through a structure (i.e., critical depth at the control section), there must be some head loss through the structure due to the energy losses caused by friction and the expansion of the flow in the sudden or gradual protected transition d/s of the throat (weir)[1].

7.2. MODULAR LIMIT

The modular limit is often expressed as the submergence ratio h_2/h_0 (or H_2/H_0). In the User's Manual, the adopted value of the modular limit for a broad crested weir to operate satisfactorily in free flow conditions is 0.75 (or 75 %), that is when $h_2 = 0.75*h_0$[2]. The submergence ratio is given by the following equation:

[1] When the d/s water surface elevation rises above a critical point, the resistance to flow in the d/s channel becomes sufficient to reduce the d/s velocity, increases the d/s flow depth, and causes a backwater effect in the structure; the broad crested weir will cease to operate in modular conditions. The head discharge relationship can no more be applied once the head of water above the crest of the tailwater (h_2) exceeds the critical depth y_c on the crest (the flow no longer passes through a critical depth). Further increases in d/s depth cause the d/s depth to increase without a change in discharge.

[2] For sharp crested weirs, the head-discharge relationship becomes inaccurate at a submergence ratio of around 0.22, so the broad crested type has a wider operating range.

$$\boxed{S = 100 * h_2 / h_0} \quad (7.1)$$

Where:
S = submergence (%).
Refer to list of symbols.

The u/s water head h_0 in relation to weir sill is computed with the following equation:

$$\boxed{h_0 = H_0 - V_a^2 / 2g} \quad (7.2)$$

Where:
Refer to list of symbols.

The d/s water head h_2 in relation to weir sill may also be taken as the difference between the tailwater surface elevation and the weir sill elevation.

If the tailwater channel or canal is relatively wide or if the tailwater surface elevation is affected by a d/s structure, it may occur that the weir as a measuring structure is modular at its maximum design capacity, but non modular with lesser discharges. Under such circumstances, a decrease in the u/s head means an increase in the submergence ratio h_2/h_0. The crest of the control section should then be raised so that h_2, and thus the ratio h_2/h_0, decrease to below the modular limit[1].

7.3. DISCHARGE RATE CONSIDERATIONS

With modular flow, the water user with his own canal inlet cannot increase the discharge by lowering the tailwater. On the other hand, in case of a submerged flow, the water user can increase the discharge by lowering artificially the tailwater level.

The decrease in the u/s velocity due to the submergence may lead to aggravate sedimentation problems, in addition to the obvious effects of increasing the complexity of determining the reduced discharge rate and raising the d/s flow depth (which could lead to overtopping the bank channel).

It is important to remember that d/s water levels can change with changes in d/s flow resistance, which frequently varies with sediment deposits, debris, canal checking operations etc. Increased d/s flow resistance can result in structures originally designed for free flow to experience submergence.

[1] A common mistake done by the designer is to calculate the head that will occur over a weir at a particular discharge without considering at all the heights of weir required to obtain a modular flow. The approach taken in the User's Manual is to allow the user to analyze alternatives to reach a more economic design by determining the minimum crest height for which modular flow can be obtained.

7.4. HEAD LOSS AND TRADEOFFS

Rectangular weirs have proven to be an effective option since they can be built easily with a simple design process. Some important design criteria have been considered in article 6 (Froude number, submergence etc.). Some options are given in the below table to satisfy the three primary design criteria for a gauged weir:

1. The Froude number must be less than 0.5 at the gauge for Q_{max} (refer to article 6.4.1).
2. The gauged weir must not be submerged at Q_{min}.
3. The gauged weir must not be submerged at Q_{max}.

Table 7.1: *options for satisfying design requirements for gauged weirs*

Design requirement	Options
At Q_{max}	
Modular flow (avoid submergence)	Raise the crest
	Narrow the control section
	Add or modify d/s slope ramp to regain potential energy
	Increase d/s expansion ratio (refer to article 25.3)
	Choose a location where more drop is available
Low Froude number	Increase approach channel top width
	Deepen the approach channel
At Q_{min}	
Modular flow (avoid submergence)	Raise the crest
	Narrow the control section
	Add or modify d/s slope ramp to regain potential energy
	Increase d/s expansion ratio (refer to article 25.3)
	Choose a location where more drop is available

8. FLOW VELOCITY AND SEDIMENTATION

The hydraulic structures should be designed to prevent sedimentation as much as possible.

The faster the water travels, the larger the particles of silt it can carry with it. The following figure shows the relationship between the size of sediment and the velocity required to erode (lift it), transport it and deposit it (Hjulström curves). The critical deposition curve shows the maximum velocity at which a river or parent canal can be flowing before a particle of a certain size is deposited. The zone in-between is the zone of transportation in suspension[1].

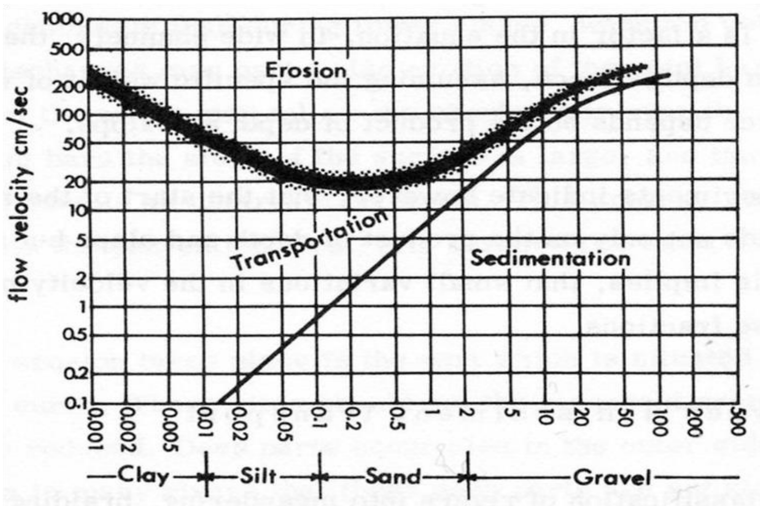


Figure 8.1: *erosion susceptibility chart (Hjulström curve)*

For example, in the above figure, it can be seen that, when the flow velocity reaches 50 cm/s, the current would pick up particles of 0.02 mm diameter from the natural bed of the channel and erosion commences.

For preventing sedimentation (deposition) from taking place, the water around the structure should no slow down enough to drop its sediment. If this is not possible, a mechanism for flushing the area subjected to sedimentation should be provided (desilting or flushing).

[1] Velocities for transportation are lower than that for erosion, because it takes much more energy to lift sediment than to maintain it. The other strange pattern is that it takes more energy to erode some of the smallest particles. This is because they are clay particles which are clogged or bonded together, therefore require a lot of energy to be eroded.

PART III:

FLOW ENERGY DISSIPATION

9. ENERGY DISSIPATION

9.1. TOTAL AND SPECIFIC ENERGY

9.1.1. Total energy

The total mechanical energy (Bernoulli) is measured from some fixed datum (refer to below figure) and its value can only reduce as energy is lost through friction[1].

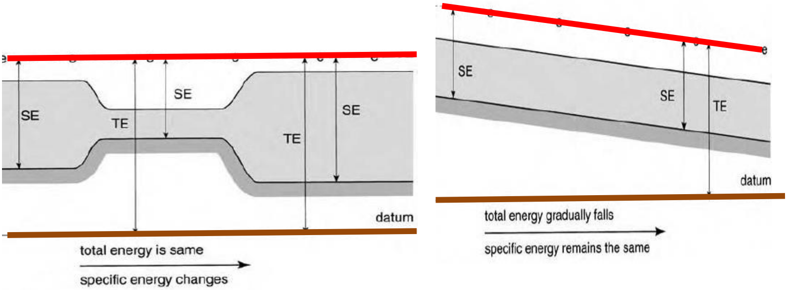


***Figure 9.1**: datum, total and specific energy*

9.1.2. Specific energy

The specific energy (Bakhmeteff) is the average energy per unit weight of water as expressed in relation to the channel bottom (datum); it is the sum of the water depth and the velocity head and measured from the bed of a channel[2] (refer to above figure).

When the bed level changes, the specific energy also changes. It also means that specific energy can rise as well as fall, depending on what is happening to the channel bed: when the flow moves from the channel over a cross-regulator, the specific energy falls; when it comes off the cross-regulator it rises again.

The total mechanical energy of a channel flow refers to a datum; if the datum coincides with the channel bed elevation at the section (datum = 0 on horizontal floor), the resulting expression is known as specific energy.

The difference between total and specific energy can be illustrated by the uniform flow. The total mechanical energy falls gradually as energy is lost through friction. However, the specific energy remains constant along the channel because there are no changes in water velocity and depth (no raised bed, hump etc.).

[1] When there is a change in the bed level of a channel (e.g. when water flows over a weir), there are also changes in the energy components but the total mechanical energy remains the same.

[2] Simply stated the specific energy is the energy in a channel measured from the bed of a channel.

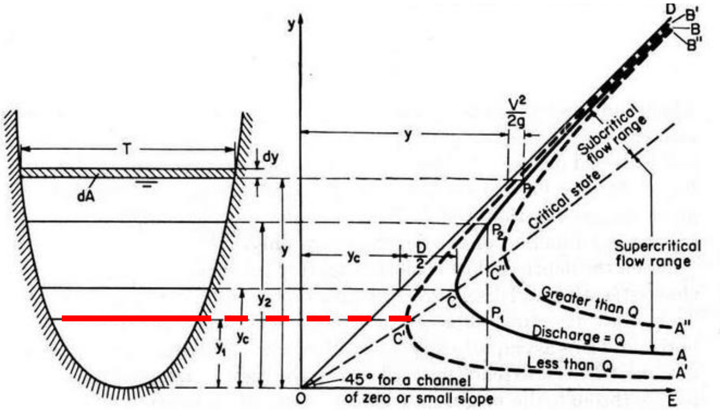


***Figure 9.2a**: specific energy curve and alternate depth*

For a given unit flow q, there are two possible depths in the channel for the same specific energy, which are called alternate depths y_1 and y_2.

The critical flow condition (y_c) is considered the most efficient, although not necessarily the most desirable[1]. To the critical depth y_c corresponds the critical energy or minimum energy E_{min}. Unique critical values exist for any discharge. For a given flow rate per unit width q_0 in a rectangular channel, to the critical depth corresponds the minimum specific energy with:

$\boxed{E_{min} = y_c * 3 / 2}$ (9.1a) or $\boxed{y_c = E_{min} * 2 / 3}$ (9.1b)

Where:

Refer to list of symbols.

The Froude number = 1 (refer to article 10.3.3).

9.2. HYDRAULIC JUMP

A hydraulic jump is to quickly reduce the excess energy (velocity) of the flow, passing from a predominant kinetic energy term $v^2/2g$ on a paved apron to a point where the flow becomes incapable of scouring the d/s channel bed with a predominant heat and potential energy. Other practical applications of the hydraulic jump are many such as:

- Prevention or containment of the scour d/s of the structure.

[1] Critical flow is unstable and, generally, it cannot be maintained over a long distance; it is rather a local phenomenon. Due to the shape of the specific energy curve close to the critical point, a small change in energy E (possibly as a result of small channel irregularities) can result in significant fluctuations in water depth as the flow oscillates between sub- and super-critical flow. In other words, especially in super-critical flow, a slight change of water depth may correspond to a great change of specific energy dissipated.

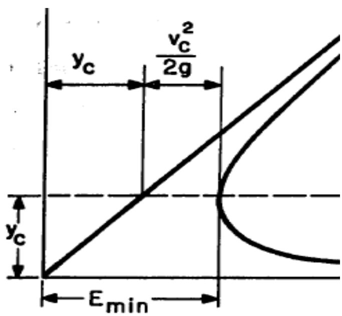
- Increase in the discharge of a sluice gate by holding back the tailwater[1].
- To recover head or raise the water level in the channel for irrigation or other water distribution purposes.
- To increase weight on an apron and thus reduce direct uplift pressure under a solid floor[2].

9.3. HYDRAULIC JUMP VARIABLES

The design of a hydraulic jump involves the following 8 variables (also shown in the following figure):

- E_1 or E_{f1}: calculated or estimated specific energy head (Bakhmeteff) in super-critical (subscript 1) or in sub-critical (subscript 2) flow range of jump in relation to canal bottom (m).
- V_1: super-critical velocity (m/s).
- y_1: super-critical initial depth of flow (m).
- V_2: sub-critical velocity (m/s).
- y_2: sub-critical sequent depth of flow[3] (m).
- q: unit discharge (m^3/s.m).
- ΔE: specific energy loss or dissipation in the standing wave (m).

The following figure shows a hydraulic jump on horizontal bed interpreted by the specific energy and the specific force (due to fluid pressure and gravitation acting on the fluid particle) curves for a constant unit discharge q.



[1] The effective head is reduced if the tailwater is allowed to drown the hydraulic jump.
[2] The raising water depth on the apron reduces the uplift pressure.
[3] In fluid dynamics, the conjugate depths or sequent depths refer to the depth (y_1) u/s and the depth (y_2) d/s of the hydraulic jump whose momentum fluxes are equal for a given unit discharge (volume flux) q but specific energy E_1 and E_2 are different due to the energy losses in the jump. It is important to note that the conjugate depths are different from the alternate depths that have the same specific energy (no losses in the system due to friction and slope absolutely flat).

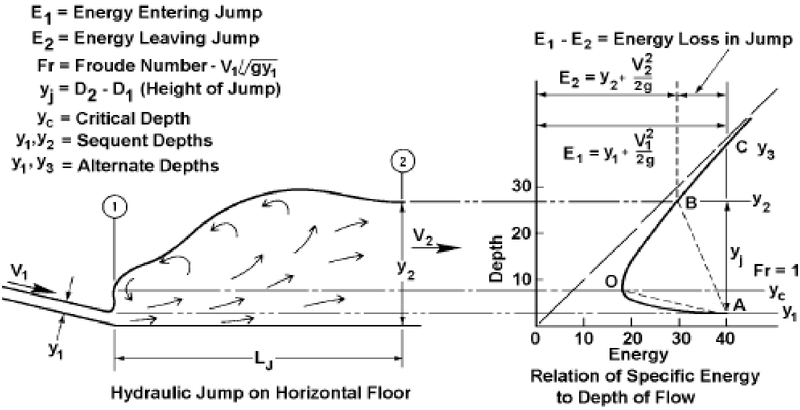


***Figure 9.2b**: jump on horizontal floor (or very mild slope), water depths and specific energies for a constant flow rate*

The depth before the jump is called initial depth y_1 and that after the jump is called sequent or conjugate depth y_2; energy loss is involved[1]. The values shown are for a unit discharge q_1 only. Different q values are not plotted on the specific energy diagram above[2].

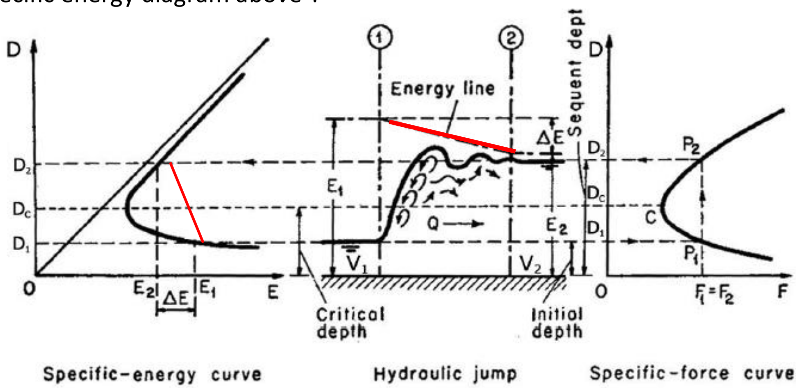


***Figure 9.2c**: hydraulic jump on horizontal floor interpreted by specific energy and specific force curves*

[1] Sequent depths are depths in which specific energy loss is involved. Alternate depths have the same specific energy. If there is no energy losses, the initial and sequent depths would be identical with the alternate depths in a prismatic channel.

[2] On the diagram, there is a distinct asymptotic relationship as the top part of the curve approaches the E = y line which means that $v_2^2/2g$ decreases (refer to equation 10. 1 in terms of unit flow rate q and y) and the bottom part of the curve tends toward the x axis.

The u/s and d/s specific energy are not equal ($E_1 \neq E_2$) (Bernoulli energy equation not applicable) but the u/s and d/s momenta are the same (momentum equation applicable with $M_1 = M_2$).

The following figure shows side by side the specific energy curve (plotted for a constant unit discharge q) and the specific discharge curve (plotted for a constant specific energy) in a rectangular section.

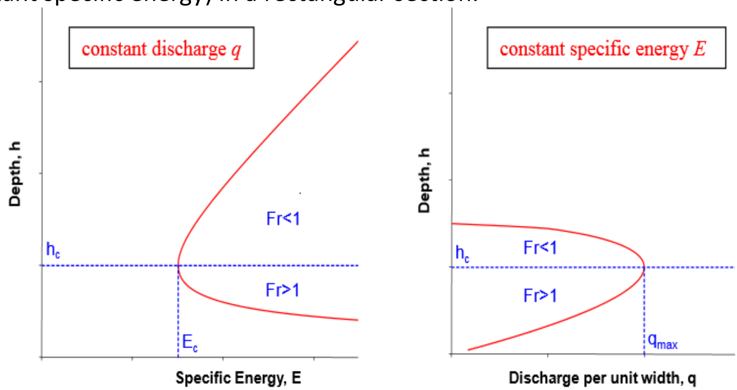


Figure 9.3: *graphs of specific energy and specific discharge curves*

It must be pointed out that the last two variables (q and ΔE) are variables of the hydraulic design:

1. For a fixed (constant) unit discharge q:
   - E minimum at y_c.
   - For all other values of E, there are two possible depths (sub-critical and super-critical depths).
2. For a fixed (constant) specific energy E:
   - q maximum at critical depth y_c.
   - For all other values of q, there are two possible depths (sub-critical and super-critical depths).

The above diagrams also indicate another difference between super-critical and sub-critical flow. For a particular value of discharge, a decrease in E head in sub-critical flow ($F_r < 1$) results in a decrease in the water depth[1]. On the contrary, with super-critical flow (Fr > 1), the figure indicates that the opposite happens: a decrease in E results in an increase in the water depth[2].

[1] For instance, the water surface is drawn down as the flow passes an obstruction in the channel such as over a weir or through a gate chamber or a bridge opening.

[2] Due to this difference of behaviour, it is indispensible for the design engineer to know which type he is being dealt with.

9.4. JUMP FORMATION

When the flow changes from super-critical state (where the kinetic energy predominates) to sub-critical state (where the thermal and potential energy predominate), the jump takes place. The change of state is due to an abrupt break in the bottom slope where the glacis slope suddenly turns flat when reaching the basin[1] or due to the presence of a gate, as shown in the opposite figure.

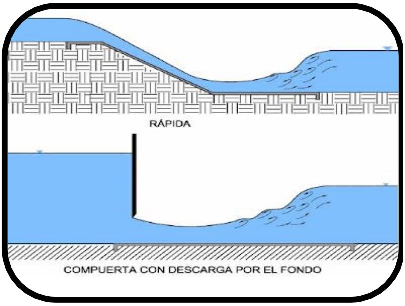


***Figure 9.4**: hydraulic jump*

Once there is super-critical flow in a channel, it is the d/s depth of flow that determines if a jump will occur. To create a jump the d/s depth must be just right. If the depth is too shallow a jump will not form and the super-critical flow will continue down the channel.

The following table summarizes the necessary conditions for the formation of a hydraulic jump.

***Table 9.1:** conditions for the formation of a jump at a considered section*

Jump	Condition	Flow	F_{r1}
Formation at considered section	$y_1 < y_c$	Super-critical	> 1
No formation at considered section	$y_1 > y_c$	Sub-critical	< 1

[1] No hydraulic jump with if $y_1 = y_c$.

10.ANALYTICAL DESIGN OF THE HYDRAULIC JUMP

10.1. HYDRAULICS OF THE STILLING BASIN

The equations proposed below are applicable if:

1. The jump takes place abruptly.
2. The flow remains streamlined throughout (refer to article 1.3).
3. The friction loss is negligible (phenomenon takes place in a short distance)[1].
4. Rectangular cross-section.

Always take care to no apply a particular equation to the wrong channel shape.

10.2. SPECIFIC ENERGY EQUATION

Energy is the most convenient parameter for determining the flow characteristics. In uniform flow, the total mechanical energy falls gradually as energy is lost through friction[2]. But specific energy remains constant along the channel because there are no changes in depth and velocity.

The value of the specific energy E_2 of the sub-critical reach of the stilling basin and of E_1 of the super-critical reach is calculated as the sum of the pressure head term[3] and the velocity head term measured with respect to the channel bottom as shown with the following equations (with Z = 0) (from Bernoulli's energy equation)[4]:

$$\boxed{E_2 = y_2 + \frac{V_2^2}{2g}\ ;\ E_1 = y_1 + \frac{V_1^2}{2g}\ ; E = y + \frac{q^2}{2gy^2}} \quad (10.1)$$

Where:

y = static pressure head (flow depth) (m).

$v^2/2g$ = dynamic pressure head (kinetic energy) (m).

Refer to list of symbols.

With the required tailwater, the velocity leaving a properly designed stilling basin (V_2) should equal the velocity V_3 of the receiving canal or channel.

10.3. COMPUTATION OF THE CRITICAL AND SEQUENT DEPTHS OF FLOW

10.3.1. Determination of critical water depth y_c

[1] For chute (glacis) less than 9 m long, the friction in the chute can be neglected.

[2] In other words, the friction slope is equal to the bottom slope when the head loss is equal to the elevation drop.

[3] = (ρ*g*h) where p is pressure (kN/m²); ρ is mass density of water (kN/m³); g is gravity constant (m/s²); h is depth of water (m).

[4] By reordering the equation, we can have the specific energy in terms of a flow rate per unit width and the channel water depth (**$E = y + q^2/(2 * g * y^2)$**). In other words, we removed the velocity term which, sometimes, is not useful, and we replaced it with a flow rate term. We now have rewritten the equation in terms of only one parameter: depth of flow (refer to graphs of figures 9.2).

The point of inflection on the specific energy curve marks the approximate position of the critical depth y_c where the specific energy is minimum for a given discharge or where the unit discharge is maximum for a given specific energy (refer to figure 9.1 and 9.2). The Froude number equals unity and the minimum energy is given by equation 9.1.

In a rectangular cross-section, the critical depth y_c is determined by applying Bernoulli's energy equation.

$\boxed{\mathbf{y_c = (q^2 / g)^{1/3}}}$ (10.2a) with $\boxed{\mathbf{q = Q / B_{structure} = y * V}}$ (10.2b)

Where:

Refer to list of symbols.

From the above equation, the critical depth is influenced only by the discharge per unit width (which means also the velocity) and the structure geometry: y_c increases with increasing discharge and, for a constant discharge, increases with decreasing channel width. On the contrary of y_n, it has nothing to do with the slope or the roughness coefficient of the channel (refer to figures 14.1 and 14.4).

10.3.2. Determination of super-critical water depth y_1

The determination of the super-critical depth y_1 and the velocity V_1 before the jump (usually at the toe of glacis) is based on the assumption of no energy loss between the upper pool and the toe of the glacis (theoretical base point formation of the jump)[1], as shown in the opposite figure. Therefore, the energy equation (Bernoulli equation) applies. Knowing the upper pool elevation, the u/s velocity head (if significant) and the discharge, the super-critical depth y_1 and entering velocity V_1 can be solved by trial and error for an assumed stilling basin floor elevation (in the User's Manual, the assumed floor elevation initially equals the d/s channel bed elevation)[2]. As an important hydraulic characteristic, the flow depth at the bottom of the glacis is also the flow depth preceding the hydraulic jump in a properly designed stilling basin.

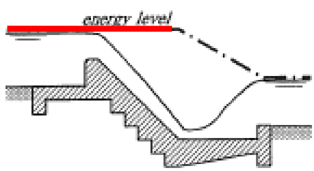


***Figure 10.1**: energy grade line in glacis and jump*

[1] For short and smooth transitions, the energy losses are negligible and the Bernoulli equation may be applied quite successfully. Short transitions may be gates (e.g. or radial gates), weirs and glacis.

[2] Glacis are not functioning as a chute having sub-critical uniform flow because the energy grade line is not parallel to the slopping glacis; the condition for functioning as a chute with sub-critical uniform flow is to have the energy level parallel to the slopping chute dissipated by friction. The glacis is to short or too smooth so that the friction loss is very small and ignored. Thus, the energy dissipation takes places not in the glacis reach but only in the stilling basin thanks to the hydraulic jump and eddies.

However, the energy equation cannot be used between y_1 and y_2 because there is an unknown energy loss at this portion of the flow. The momentum equation is used instead to describe the flow between those two positions.

10.3.3. **Determination of the u/s Froude number**

The Froude number is the ratio of the velocity of a stream divided by the speed of a small wave on the water surface relative to the speed of the water, called wave celerity[1] (F_r = water velocity/wave celerity).

The Froude number F_r (or F_{r1} before the jump) is computed according to the following equation (rectangular cross-section)[2]:

$$\boxed{\mathbf{F_{r1} = V_1 / (g * y_1)^{1/2}}} \quad (10.3a) \qquad \text{or} \qquad \boxed{\mathbf{F_{r1} = q / (g * y_1^3)^{1/2}}} \quad (10.3b)$$

<u>Where</u>:
V_1 = stream velocity before the jump.
Refer to list of symbols.

V_1 is usually considered at the toe of the glacis if any. At critical flow, the wave celerity equals the flow velocity and the Froude number = 1. Any disturbance to the surface will remain stationary.

10.3.4. **Relationship between the sequent depths**

The ratio of the flow depths across the hydraulic jump is computed thanks to the momentum conservation principle and the continuity equation[3] across the jump. The u/s and d/s specific energies are no more assumed as equal ($E_1 \neq E_2$) but the u/s and d/s specific forces are the same[4]. The forces of this momentum equation ignore the forces of the baffle blocks if any in the stilling basin.

Theoretically speaking, in a rectangular channel, a hydraulic jump takes place at a point where the u/s and d/s sequent depths and the approaching

[1] The celerity wave is typically equal to $(y*g)^{1/2}$ with y = the water depth in the channel at the considered point.

[2] There are several ways to calculate the Froude number.

[3] The continuity equation relates mass rate of flow along a streamline. It links u/s and d/s discharges (or areas and velocities) and means that the amount of water flowing into a system must be equal to the amount of water flowing out of it.

[4] The momentum equation, stated as the sum of all external forces, is equal to the rate of change of momentum. This equation links force (or mass) to velocity. The forces of this momentum equation are shown in the figure below.

Momentum Equation

$F_1 - F_2 + F_g - F_f = \rho Q(V_2 - V_1)$

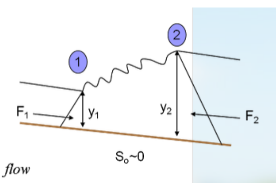
Where

F_1 = *Force helping flow*

F_2 = *Force resisting flow*

F_f = *Frictional* Resistance

Fg = Gravitational component of flow



Assumptions: the friction can be neglected (length very small) and the slope close to 0 (gravitational component of flow may be neglected)

Froude number satisfy the below equation. This relationship between the initial depth y_1 and the sequent depth y_2 of a hydraulic jump on a horizontal floor in a rectangular channel is a quadratic equation in terms of the entering Froude number and is solved by iteration. This equation is applicable even when the flow enters the jump at an appreciable angle to the horizontal.

$$\boxed{\mathbf{y_2 / y_1 = 1 / 2 * [(1 + 8 * F_{r1}^2)^{1/2} - 1)]}} \quad (10.4)$$

Where:

Refer to list of symbols.

This equation can be swapped around and be in terms of not F_{r1} as above but critical depth y_c[1]. There is no hydraulic jump formation when $y_c = y_1$.

10.4. CHARACTERISATION OF THE HYDRAULIC JUMP

10.4.1. **Concentration and degradation**

In the Manual, the jump characteristics are determined taking into account the following hypothesis:

- No concentration of flow[2].
- No degradation (retrogression) of d/s level because of the protective works (refer to article 24).

10.4.2. **Base point and surface point elevation of jump formation**

For a given energy head loss ΔE and a discharge intensity q, there is a definite value of the quantity of d/s specific energy (E_2 or E_{f2}) in sub-critical flow range required for the jump formation. This quantity of d/s specific energy E_2 or E_{f2} indicates the theoretical position of the base point of the jump formation, hence the elevation of the stilling basin floor.

The elevation of the position of the base point of jump formation is computed as the difference between the total d/s mechanical energy elevation HE_2 and the calculated or estimated specific energy head E_2 or E_{f2} in sub-critical flow range of the hydraulic jump as shown in the following equation:

$$\boxed{\textbf{Elevation of base point formation} = \mathbf{HE_2 - E_2}} \quad (10.5)$$

Where:

Refer to list of symbols.

The elevation of the surface point of jump formation is obtained by adding to the elevation of the base point of jump formation the super-critical depth y_1.

[1] The equation becomes, taking y_1 as an example: $\mathbf{y_1 = (y_2 / 2) * [(1 + 8 (y_c / y_2)^3)^{0.5} - 1]}$ or $\mathbf{y_1 = (y_2 / 2) * [1 + 8 * q^2 /(g * y_2^3)^{0.5} - 1]}$. The 2 conjugate depths y_1 and y_2 may replace each other in the equations.

[2] While calculating the flow, the possibility of non uniform flow is not taken into account by providing a suitable concentration factor chosen arbitrarily (usually 20 % which means that the calculated maximum discharge rate is increased by 20 % during the design phase).

10.4.3. **Hydraulic jump efficiency**

The dimensionless ratio of the specific energy after the jump (E_2) to that before the jump (E_1) is defined as the efficiency of the jump and indicates how much of the original energy stays after the jump. It is calculated with the following equation:

$$\boxed{\frac{E_2}{E_1} = \frac{(8Fr_1^2+1)^{\frac{3}{2}}-4Fr_1^2+1}{8Fr_1^2*(2+Fr_1^2)}} \quad (10.6a)$$

Where:

Refer to list of symbols.

The above equation shows that the efficiency depends only on the u/s Froude number (F_{r1}). A greater F_{r1} corresponds to a lower ratio E_2/E_1 and a higher relative loss or energy dissipation in the jump as shown in figure 10.2 and as calculated with the following equation:

$$\boxed{\textbf{Relative loss in energy} = 1 - E_2 / E_1 = \Delta E / E_1} \quad (10.6b)$$

Where:

Refer to list of symbols.

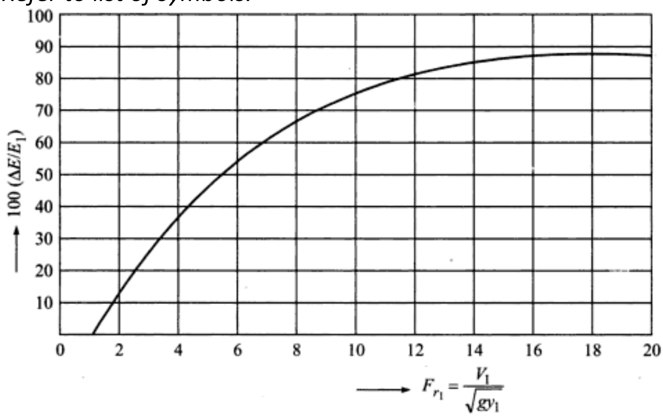


***Figure 10.2:** relative loss in specific energy head of hydraulic jumps in horizontal rectangular structure*

10.4.4. **Hydraulic jump energy head loss**

The total mechanical head loss ΔE in the basin equals the difference of the specific energies before and after the jump as shown in the following equation:

$$\boxed{\Delta E = E_2 - E_1 = \frac{(y_2-y_1)^3}{4*y_2*y_1}} \quad (10.7a)$$

Where:

Refer to list of symbols.

For energy to be lost, it requires $y_2 > y_1$.

This quantity of mechanical energy dissipated is converted into thermal, sound and potential energy as a result of frictional effects during the jump. Although the energy dissipated in the highly turbulent motion in the jump can be considerable, the difference in temperature is not great because the flow rate is usually quite high. The energy per unit width dissipated by the hydraulic jump at design flow conditions is calculated with the following formula:

$$\boxed{E_{lost} = \gamma * q * \Delta E} \quad (10.7b)$$

Where:

E_{lost} = energy dissipated per unit width in metre (kWs).

γ = specific or unit weight of water (9.81 kN/m^3)[1].

Refer to list of symbols.

10.4.5. **Hydraulic jump height**

The height of the jump h_j is the difference between the water depth after (y_2) and before the jump (y_1). The jump elevation is the sum of the elevation of the base point of the jump formation and the jump height[2].

The relative height of the jump can also be expressed as a ratio with respect to the initial specific energy E_1 as shown in the following equation:

$$\boxed{\frac{h_j}{E_1} = \frac{y_2}{E_1} - \frac{y_1}{E_1}} \quad (10.8)$$

Where:

Refer to list of symbols.

This relative height increases up to Froude number 2.77 and then decreases non linearly as the value of Froude number increases to eight (refer to figure 10.3).

10.4.6. **Hydraulic jump length**

The length of a hydraulic jump may be defined as the distance measured from the front face of the jump to a point on the surface d/s from the rollers where it gets back to normal depth again. This length cannot be determined easily by theory. Some empirical equations are proposed below (refer to article 12.3.2).

[1] Specific or unit weight of water $\gamma = \rho * g$ with ρ in kg/m^3(water density).

[2] The above calculation assumes that the complete jump is located upstream of the sill and that no additional energy loss takes place at the sill.

In the User's Manual, the length of the hydraulic jump equals the length of the stilling basin, which may include an end sill (refer to article 12.5).

10.4.7. **Characteristic curves of the hydraulic jump**

Since the relative loss, the efficiency, the relative height and the relative initial and sequent depth of a hydraulic jump in a horizontal (or nearly so) rectangular cross-section are function of F_{r1}, they can be plotted against the u/s F_{r1}, resulting in a set of characteristic curves shown in the following figure:

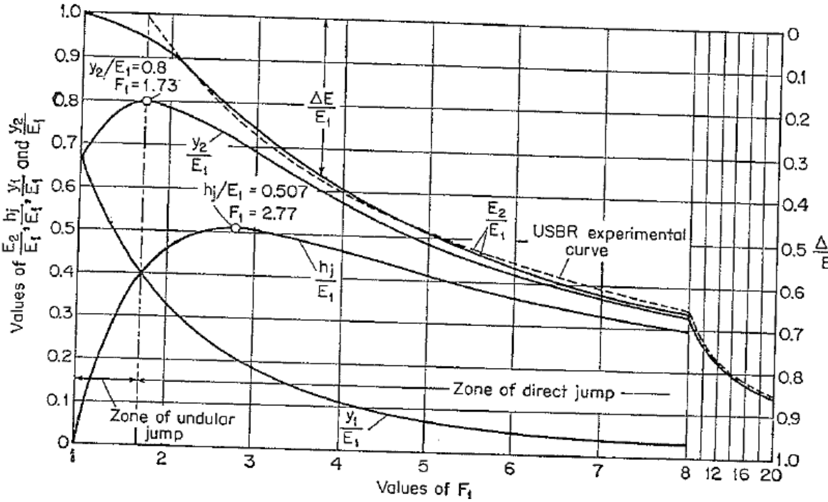


Figure 10.3: *characteristic curves of hydraulic jumps in horizontal rectangular structure*

Theses characteristic curves provide the designer with a general idea about the range and conditions under which the proposed structure is to be operated.

With reference to these curves, the following features may be noted:

1. The maximum relative jump height h_j/E_1 is 0.507, occurring at $F_{r1} = 2.77$ as per USBR standard.
2. The maximum relative sequent depth y_2/E_1 is 0.8, which occurs at $y_1/E_2 = 0.4$ and $F_{r1} = 1.73$. For this value of $F_{r1} = 1.73$, the transition from undulating jump to a direct jump takes place (refer to figure 12.1).
3. When $F_{r1} = 1$, the flow is critical and $y_1 = y_2 = 2/3*E_1$ (refer to article 9.3). It means also that the velocity of the stream is the same as the propagation of the wave (refer to article 10.3.3).
4. When F_{r1} increases, the changes of all characteristic ratios become gradual.

5. The efficiency E_2/E_1 decreases non linearly with Froude number from 1 to 7.

10.5. STABILITY AND CONTROL OF THE HYDRAULIC JUMP

Theoretically speaking, in a rectangular channel, a hydraulic jump occurs at a location where the sequent depths and the approaching Froude number satisfy the equation 10.4. However, there can be conditions in a channel, such as a d/s control, that can alter where the conjugate depths form. For instance, tailwater depth can play a very influential role on where the jump will occur in the channel and changes in this depth can shift the jump either u/s or d/s.

With the presence of a glacis, the jump may occur in either the steep glacis (preferably at the lowest) or the cistern floor.

10.5.1. Pattern of jump positions

There are 3 alternatives patterns of jump positions or locations.

1. **$y_n = y_2$:** jump rating curve (y_2 versus Q) coincides with the tailwater rating curve in the canal (y_n versus Q) at all discharges. This is the ideal condition for scour protection because the jump will form and remains at the toe of the glacis at all discharges. In such a case, a simple apron is generally sufficient to provide protection in the region of the hydraulic jump.
   However, a little difference between the assumed and actual values of the relevant hydraulic coefficients may cause the jump to move from its estimated position. Consequently, some devices to control the position of the jump are always necessary.
2. **$y_n > y_2$:** the tailwater rating curve in the canal (y_n versus Q) is lying above the jump rating curve in the basin (y_2 versus Q) at all discharges. The jump moves upwards and will drown out at the source by the tailwater, becoming a submerged (under water) jump. It is probably the safest design because the position of the submerged jump can be most readily set. However, the design is not efficient and very little energy will be dissipated[1]. The wave celerity may continue to move at high velocity along the channel bottom for a considerable distance.
   The remedy for this problem is to adjust the glacis slope to create the proper conditions for a jump to occur somewhere on the glacis at all discharges.
3. **$y_n < y_2$:** the tailwater rating curve in the canal (y_n versus Q) is lying below the jump rating curve (y_2 versus Q) at all discharge. The jump moves downwards to a point where the equation y_2/y_1 for a given specific energy is satisfied and a jump takes place. This case must be

[1] In fact, the super-critical flow will rush underneath and still cause erosion for some distance downstream.

avoided in design because the jump, repelled from the scour resisting apron of the stilling basin, will take place in the unprotected channel, resulting in severe erosion.

The below table summarizes the pattern of the jump positions or locations, taking into account that we must have the presence of a super-critical flow to assure the formation of the jump (refer to table 9.1).

Table 10.1: summary of the pattern of the jump positions or locations with super-critical flow

Jump location	Conditions
Standing wave at the considered cross-section	$y_n = y_2$
Moves upwards from the considered cross-section	$y_n > y_2$
Moves downwards from the considered cross-section	$y_n < y_2$

There is no simple remedy for a deficiency in tailwater depth, even with the aid of appurtenances. Increasing the length of basin, which is the remedy often attempted in the field, will not compensate for deficiency in tailwater depth. Baffle piers and sills are only partly successful in substituting for tailwater depth. Furthermore, if it is not possible to depress the basin floor because of difficulties in excavation, then a lateral expansion of the basin remains the only possibility for guaranteeing the required dissipation of energy. In such basins, there are mainly two problems faced by the designer: the determination of sequent depth and the estimation of energy loss.

10.5.2. Tailwater consideration

The tailwater depth[1] fluctuates owing to changes of the discharges in the canal or river. Usually, the tailwater levels need only be checked at Q_{min} and Q_{max}, since the flow should also be modular at intermediate flows, if the flow is modular at Q_{min} and Q_{max}.

In most of the design, it is assumed that the tailwater depth remains fixed. However, there are two situations of interest with regards to the d/s channel tailwater:

1. The tailwater level d/s of the structure may be influenced only by d/s channel friction. When this occurs, the flow is said to be uniform and the water depth is the normal water depth y_n (Manning's equation)[2]. However, owing to seasonal changes of discharge in the canal and of the hydraulic resistance of the d/s channel, the flow velocity changes the water depth (in doing so, the relation between the sequent depth

[1] Although flow is still non uniform in the diverging transition, the energy losses due to friction are estimated by applying the Manning equation.

[2] When the flow in the d/s channel is at normal depth, the roughness, cross-section and bed slope of the channel are equal for a sufficient distance such that the water level at the structure site is controlled solely by the frictional resistance of the d/s channel (i.e., no backwater effects).

y_2 and the tailwater depth y_n). Therefore, the tailwater levels should be determined for the seasonal maximum d/s channel roughness (n = 0.035 in the User's Manual)[1].

2. When there are obstacles in the channel d/s of the structure, the tailwater level is not governed by normal depth y_n, but by backwater (or drawdown) from of the obstruction (overfall etc.). In such cases, the tailwater level depends greatly on the properties and settings of this d/s obstacle. From a practical standpoint, the easiest way to determine the resulting tailwater level is to measure it during the worst-case conditions.

Therefore, even with a broad-crested weir, tailwater needs to be checked at both minimum and maximum flow, since backwater can cause high tailwater depths even at low flows. Also, a small difference between the actual and calculated values of hydraulic coefficients may move the jump formation point from its estimated or calculated position.

The usual case represents the conditions in which the jump rating curve is at a higher stage than the tailwater rating curve at low discharges but at a lower stage at high discharge. As a result, the hydraulic jump position moves up and down on the glacis and the channel, hunting for the right depth of flow. This unstable behavior is often undesirable. The safest method (adopted here) to ensure a more stable jump is:

- Check the stilling basin for a jump formation at low discharges.
- Combine with the basin a sloping apron for developing a satisfactory jump at high discharges.
- Ensure a hydraulic jump to be formed within the stilling basin for the maximum tailwater and discharge and ensure the energy dissipation is sufficient while the stilling basin is not in full capacity.

The choice of the glacis and the basin involves consideration of economics and hydraulic performance:

Tailwater rating curves are extremely important. The variation in a tailwater rating curve may shift toward more flow capacity, less flow capacity, or oscillate from one to the other and back again. The shift in rating may be abrupt, gradual, or sporadic[2]. The selected tailwater curve is used in design of stilling basins, wall heights, erosion protection and many other critical elements that make up a

[1] For broad crested weir with a uniform cross-section and with tailwater at normal depth y_n, the submergence need only be checked at Q_{max} because the tailwater level will generally decline faster than the u/s depth if the flow rate is reduced.

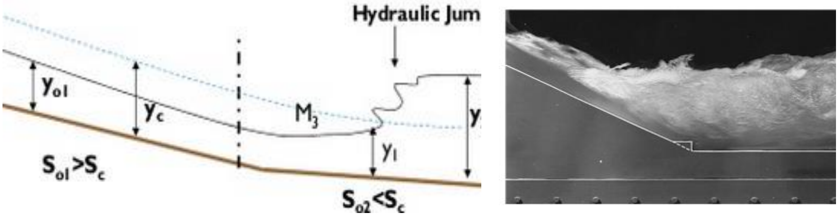
[2] The variation in a tailwater rating curve may be caused by sediment erosion or aggradation, deposition or excavation of channel bed or bank material, variations in hydrologic events, loops in rating curves as flow transitions from the raising to falling flood stages, inaccurate estimates of channel roughness (resistance to flow) or by man-induced events.

total project design. It has been observed however that the majority of the basins were designed for conjugate tailwater depth or less.

Therefore, it is imperative that the hydraulic engineer not only has an accurate estimate of what the tailwater curve will be after the project construction and throughout the life of the project, but designs a basin deep enough to provide for full conjugate depth (or some greater depth to include a factor of safety) at the maximum weir design discharge[1]. For example, for projects with loop rating curves, raising stages should be used for design of stilling basins and erosion protection and falling stages used for setting wall heights. The use of an average tailwater rating curve may yield inadequate design for both wall height and the high-velocity flow areas. In many cases, a cross-regulator structure is provided d/s to maintain the tailwater depth y_n.

10.5.3. **Control of the position of the hydraulic jump**

Depending on the corresponding value of y_1, the jump may occur either on, after or before the break in slope as shown in the following figure (with y_0 & $y_1 < y_c$):



$y_1 > y_0$ (y_0 = current depth on glacis) : jump d/s of break point
$y_1 < y_o$ (y_0 = current depth on glacis): jump u/s of break point
$y_1 = y_0$) (y_0 = current depth on glacis: jump on break point

***Figure 10.4**: theoretical position of the jump on the glacis*

The cistern elevation usually matches the elevation of the jump base point formation[2]. In the User's Manual, the formation of all types of jump is usually stabilized at the lowest end of the sloping glacis and not swept out of the stilling basin causing excessive scouring. Therefore, the floor of the basin is set at such a level that the tailwater surface elevation stays greater than the water surface elevation in the basin under varying conditions of head (at the higher flow range).

Taking into account that the tailwater rating curve at best is only approximate and the exact position of jump and the type of flow cannot be

[1] The designer is cautioned against spending too much effort in refining inconsequential parameters, such as spillway pier shape coefficients, without paying sufficient attention to potential shifts in tailwater rating curves that can, of course, have drastic influences on submerged structure capacity.

[2] As a first approximation, the cistern elevation is provided at the level of the d/s channel bed elevation.

determined only on an analytical way, a set of four conditions are adopted in the User's Manual during the design phase to stabilize the position of the hydraulic jump:

1. Condition Nº 1.A is met when the tailwater surface elevation y_n is 10 % greater than the sequent depth y_2 of the jump[1].
   Condition Nº 1.B is met when the depth of the cistern floor is increased by 25 % of the value of E_2.
2. Condition Nº 2 is met with the presence of a d/s end sill (refer to article 12.5) in cast with the cut-off wall in order to diffuse the residual portion of high velocity jet reaching the end of the basin, as shown in figure 12.1. The sizing of the end sill d/s of the dissipater is given by the following equation (the fore slope of the end sill should be set at 1H on 0.5V):

$$\boxed{\mathbf{h_{es} = y_1\,(0.0536 * F_{r1} + 1.04)}} \quad (10.9)$$

   Where:
   h_{es}: height of the end sill in relation to d/s channel or canal bed elevation (m).
   Refer to list of symbols.

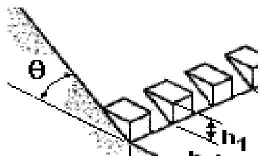


***Figure 10.5**: chute blocks*

3. Condition Nº 3 is met with the presence of the proposed glacis slope (2H or 3H on 1V) or other appropriate structure (refer to articles 10.5.4 and 12.2.2).
4. With a straight glacis creating a super-critical flow with Froude number F_{r1} between 2.5 and 9, a single row of triangle blocks (glacis blocks or chute blocks) as shown in the opposite figure are provided at the foot of the flumed glacis to ensure a more uniform flow. These chute blocks tend also to stabilize the jump and shorten the length.

In addition, a balance has to be stricken between adequate dissipation of energy and stability (fixed position on the glacis) of the hydraulic jump, taking into account that the jump formation depends upon the hydraulic conditions immediately after it.

At this point of the design, the assumed stilling basin elevation is checked against the existing tailwater surface elevation and a new stilling basin floor elevation is assumed until all the conditions for controlling the hydraulic jump are satisfied.

[1] 10 % for basin type IV; 5 % for basin type II. In the Manual, the 10 % increase is generally introduced for all the basin types as a FoS.

10.5.4. **Glacis slope**

The glacis type of fall is suitable up to a flow rate of 60 m^3/s and/or a topographical drop up to 2 m. In case of topographical drops greater than 2 m, drop structures are usually designed, such as vertical drop, chute drop, pipe drop or cascade drop (refer to article 12.2.2)

The slope of the glacis u/s of the stilling basin has little effect on the jump as long as the distribution of velocity and depth of flow are reasonably uniform on entering the jump. Furthermore, if the slope of the glacis is not too pronounced, the gravity component of the momentum equation can be neglected on the grounds that the error so introduced is practically cancelled by the errors involved in other assumptions. Therefore, in this User's Manual, a 2H on 1V or 3H on 1V slope is adopted for the sloping glacis and the equations derived for a horizontal bed may be applied to a sloping glacis.

This proposed glacis slope is usually sufficient to create on the glacis a flow velocity V_1 greater than the critical velocity V_c, triggering the formation of the jump, ideally at the toe of the glacis.

11.HYDRAULIC JUMP DESIGN WITH DESIGN CHART

11.1. DESIGN CHART

The hydraulic jump characteristics can be estimated with the following abaci:

- Blench's design chart for estimating the specific energy E_{f2} of the flow in sub-critical state d/s of the jump.
- Montague's design chart for estimating the sequent depths.
- Khosla's design chart for uplift pressure (refer to article 23).

11.2. ESTIMATION OF THE SPECIFIC ENERGY

Blench's curves shown in the following figure give the estimated specific energy E_{f2} d/s of the jump by taking the discharge intensity per unit width q and the head loss H_L (ΔE) as known variables[1].

At this level, a difference might occur between the computed and estimated energy ΔE due to the following:

- The specific energy E_2 is calculated with the sequent water depth y_2; the velocity head is calculated with the sub-critical velocity V_2.
- The specific energy E_{f2} is estimated by means of the tailwater depth y_n; the velocity head is calculated with the d/s velocity V_3.

This difference is summarized in the following table.

Table 11.1: *variables used for the calculation of E_2 and estimation of E_{f2}*

Energy	Water depth (m)	Velocity (m/s)
E_2	y_2	V_2 (sub-critical reach)
E_{f2}	y_n	V_3 (d/s of the stilling basin)

11.3. SEQUENT DEPTHS ESTIMATION WITH DESIGN CHART

Knowing the specific energy E_f, the conjugate depths y_1 and y_2 can be read directly from the energy of flow curves given in Montague's design chart, corresponding to certain discharge intensity per unit width of stream.

[1] The head loss considered may be the difference between the u/s and d/s water surface elevation.

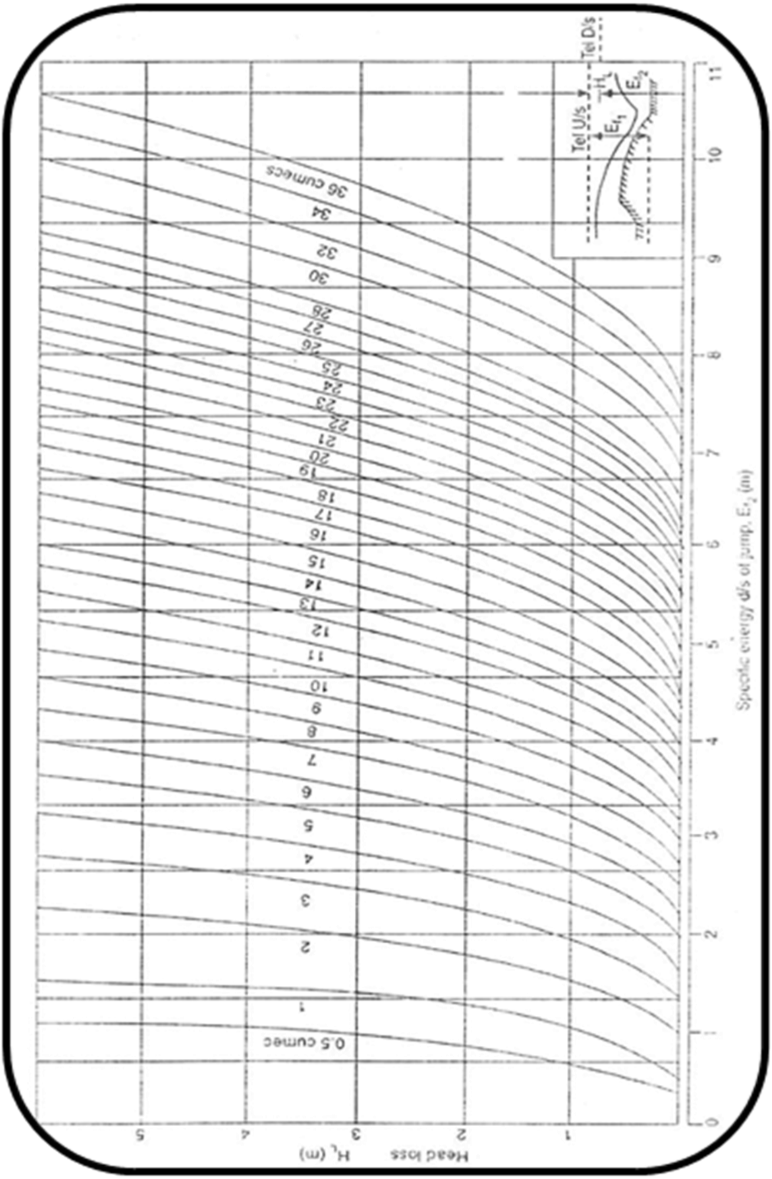


Figure 11.1*: Blench’s design chart for the estimation of the specific energy E_{f2} with known discharge intensity and energy loss*

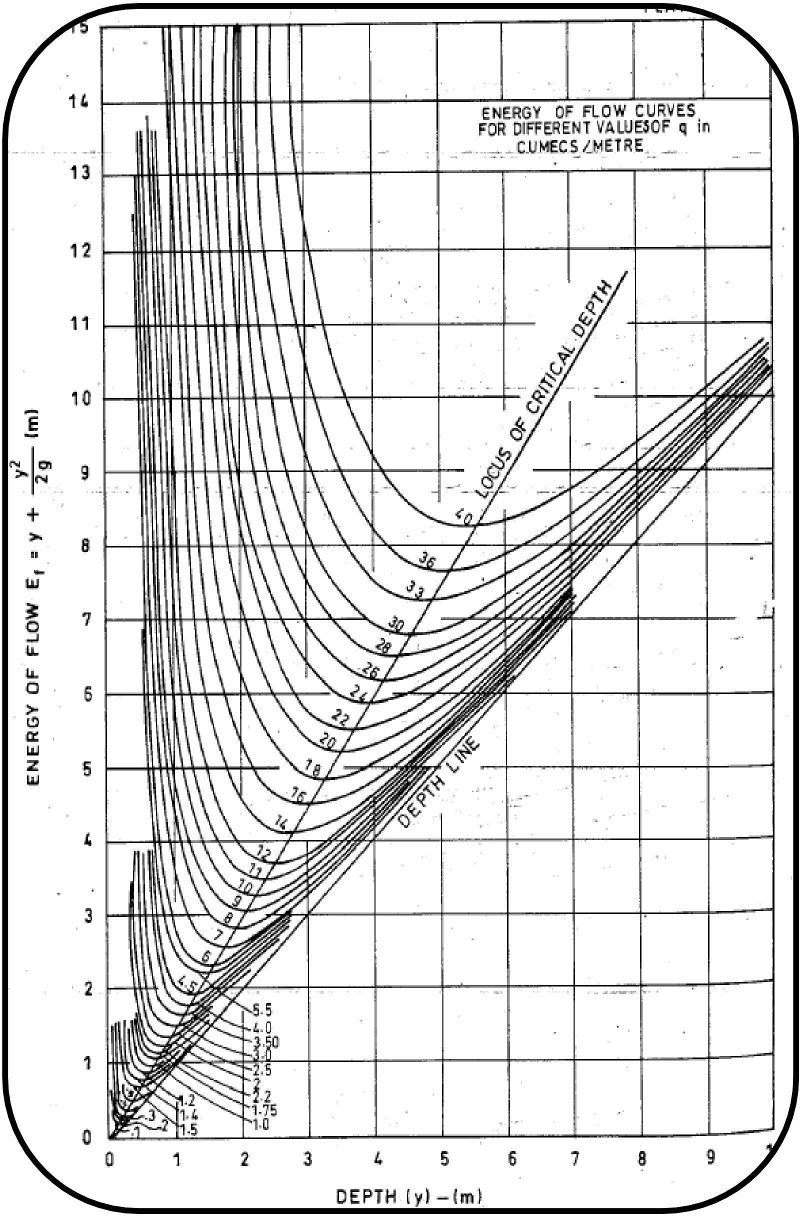


Figure 11.2: *Montague's design chart for the determination of energy flow curves for different values of discharge intensity per unit width q*

11.4. POSITION OF THE POINTS OF JUMP FORMATION

11.4.1. Elevation of the base point of jump formation

The position of the base point of jump formation is determined by checking the curves of the Blench's design chart (refer to the above figure) relating the energy head loss ΔE (H_L) and the unit discharge q to the estimated specific energy E_{f2}.

11.4.2. Elevation of the surface point of jump formation

The elevation of the surface point of jump formation is obtained by adding to the elevation of the base point of jump formation the estimated super-critical initial depth y_1.

11.5. CHARACTERISATION OF THE HYDRAULIC JUMP

The approach is the same as in the case of the analytical design (refer to article 10.4).

PART IV:

EXTERNAL FLOW ENERGY DISSIPATER

12.STANDARD ENERGY DISSIPATER STRUCTURES

12.1. INTRODUCTION

Energy dissipation can occur in different manners: (i) due to friction, (ii) by impact of the flow against the floor, (iii) by turbulence in a stilling basin and (iv) by heat and sound. Therefore, different types of energy dissipater structures can be distinguished.

Stilling basins are external energy dissipaters placed at the outlet of the structure. These basins are characterised by some combination of chute blocks, baffle blocks and end sill designed to trigger and/or maintain a hydraulic jump in combination with tailwater conditions. The stilling basin design guidance presented in this chapter is for free flow. Stilling basins designed for submerged flow normally require a model study.

USBR has developed various types of generalized water energy dissipater structures with rectangular cross-sections that will be used in the User's Manual. Each type of energy dissipater has to be investigated. In many cases, more than one solution to a particular problem will be possible. The choice will depend upon considerations of economics, material and complexity for construction.

Other forms of energy dissipater include the vertical drop structure with hydraulic jump or impact blocks where the dissipation takes place by impact of the inflow on a vertical baffle.

12.2. SELECTION OF THE ENERGY DISSIPATER

The stilling basin is the common type of dissipaters for weirs. The selection of a stilling basin depends upon:

- Approach flow conditions (Froude number).
- Hydraulic limitations.
- Basin size, constructability and cost
- Tailwater characteristics.
- Scour potential.
- Personal preference and experience.

12.2.1. Froude number

The u/s Froude number is the main criterion to select among the different types of standard stilling basins used for dissipating the energy.

In the following figure, the jumps and corresponding USBR stilling basin are classified according to the Froude number F_{r1} of the incoming flow. The higher the Froude number is at the entrance of the basin, the more efficient are the hydraulic jump and the shorter the resulting basin.

If the Froude number does not fall within an acceptable range, the following methods can be employed:

- Reduce the unit discharge by expanding the crest width.
- Increase the drop height by raising the crest elevation of lowering the stilling basin floor elevation.
- A combination of above.

To increase the Froude number, the potential[1] energy has to be converting into kinetic energy; opposite changes for decreasing the Froude number[2]:

12.2.2. **Topographical drop**

When the topographical ground slope exceeds a 2 m drop (and less than 6.1 m), this excess will be accounted in the User's Manual by providing an alternative to the glacis, with a vertical drop or a fall type of structure (refer to article 10.5.4 and 12.4).

12.3. CARACTERISTICS OF THE ENERGY DISSIPATER

12.3.1. **Classification of energy dissipater**

The classification of hydraulic jumps and dissipaters in figure 12.2 may be considered as a guideline for hydraulic jumps in rectangular horizontal channels.

12.3.2. **Total length of dissipater**

On horizontal floor, the total length of a stilling basin is considered in the User's Manual as related to the length of the hydraulic jump before the drop off caused by the d/s channel conditions (refer to article 10.4.6). This length may include an end still.

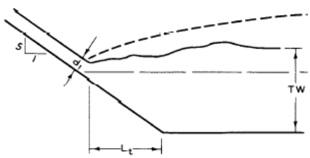
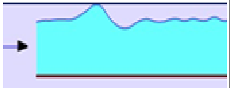
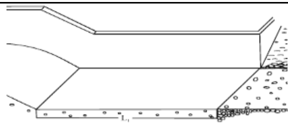
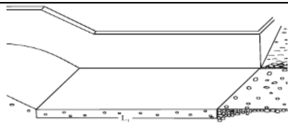
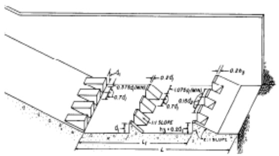


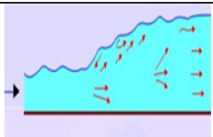
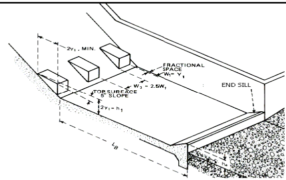
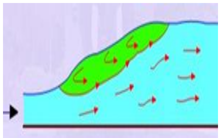
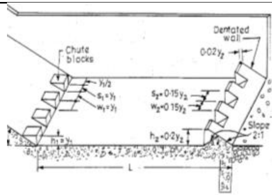
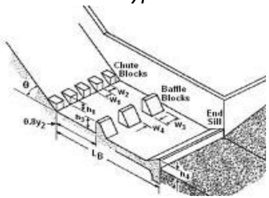
***Figure 12.1**: hydraulic jump length on glacis*

This length cannot be determined easily by theory and some empirical equations and graphs are proposed below. In case of a jump on sloping floor, the depth ratio y_2/y_1 increases with the slope of the floor as shown in the opposite figure. It means that the jump on sloping floor requires more tailwater depth than the corresponding horizontal floor jumps. However, in the User's Manual, the equations derived for a horizontal floor are applied to the sloping glacis (refer to article 10.5.4), considering also that a proper tailwater depth is provided for maximum flow (refer to article 10.5).

[1] When there is potential energy only, the flow rate is zero (the water is stagnant). When there is kinetic energy only, the flow rate is also zero because the water depth has to be nil The maximum flow rate lies between these two extremes.

[2] In practical design, the economies would be checked between the cost of the expanded crest or the cost of raising the crest or the cost of lowering the stilling basin floor.

Hydraulic jump classification	**Froude number**	**Stilling basin type (USBR)**	**Characteristics**	**Energy loss**
No jump occurs (sub-critical flow)	$F_{r1} < 1$			± 0 %
N jump occurs (critical flow)	$F_{r1} = 1$		$y_2 = \pm 2*y_1$; $V_2 = \pm V_1/2$	± 0 %
 Undular jump	$1 < F_{r1} < 1.7$	 *Basin **type I***	-Two conjugate depths are close - Flat floor with no appurtenances - Extensive length to contain the jump - Costly due to length - No control on jump	≤ 5 %
 Weak jump	$1.7 < F_{r1} < 2.5$	 *Basin **type I***	- $2.0 < y_2/y_1 < 3.1$ - Flat floor with no appurtenances - Extensive length to contain the jump - Costly due to length - No control on jump	≤ 15 %
		 *Basin **type LFN***	- $2.0 < y_2/y_1 < 3.1$ - Alternative for type IV or I - Chute blocks, baffle blocks and dentaded end sill - Less simple for construction - Decrease length of basin thanks to auxiliary devices	≤ 33 %

Hydraulic jump classification	**Froude number**	**Stilling basin type (USBR)**	**Characteristics**	**Energy loss**
 Oscillating jump (entering jet intermittently flowing near bottom and along surface	$2.5 < F_{r1} < 4.5$	 *Basin type **IV***	- $2.0 < y_2/y_1 < 3.1$ - Most common in our design - Jump difficult to handle - Try to change dimensions to be out of oscillating conditions and high d/s celerity waves	≤ 45 %
		*Basin type **LFN***	-Recommended instead of type IV (refer to characteristics above)	
 Steady jump (least sensitive in terms of its position to small fluctuations in the tailwater elevation)	$4.5 < F_{r1} < 9$	 *Basin type **II***  *Basin type **III***	- Incoming flow velocity V > 18 m/s - No particular difficulties to handle - No baffle blocks due to high velocity (cavitation) - For large hydraulic structure - $5{,}9 < y_2/y_1 < 12$ - Incoming flow velocity V < 18 m/s (no cavitations) - $q_{max} = 18\ m^3/s.m$ - Recommended because length basin type III < length basin type II and type I (up to 60% of type I) - Chute blocks, baffle blocks and end sill to decrease length (baffle blocks can also be cube shaped)	± 70 %

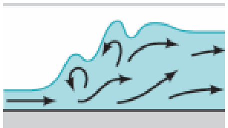
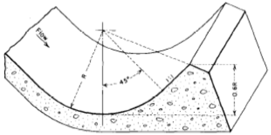
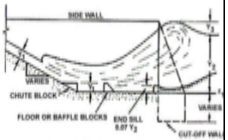
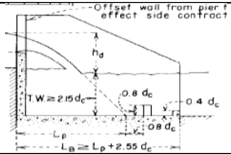
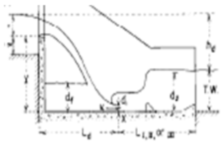
Hydraulic jump classification	Froude number	Stilling basin type (USBR)	Characteristics	Energy loss
 Strong jump	$F_{r1} > 9$	 *Basin **type V – bucket***	- $12 < y_2/y_1 < 20$ - High dam spillway with sloping glacis - Bucket dissipates energy (and not the hydraulic jump and basin because too large and expensive) - When the tailwater depth is too great for the formation of a hydraulic jump	± 80 %
	$1.7 < F_{r1} < 17$	*Basin **type SAF***	- Difficult to construct	
	2 m ≤ drop ≤ 6.1 m	***Straight drop with impact blocks***	- Drop < 6.1 m - Basin dimensions determined with design chart - Impact block basin length < hydraulic jump drop basin - $F_{r1} \leq 4.5$	
		***Straight drop with hydraulic jump and stilling basin***	- Drop < 6.1 m - Basin dimensions determined with design chart - Stilling basin type II, III or IV	

Figure 12.2: *classification of hydraulic jumps and types of stilling basins according to the incoming Froude number F_{r1} (USBR)*

1. Total dissipater length according to the type of stilling basin and the value of the Froude number.

Table 12.1: *empiric relations for the calculation of the total basin length according to the type of basin (USBR) and the value of the Froude number (rectangular cross-section)*

Type	F_{r1} *(dimensionless)*	Length of basin (m)
IV	$2.5 < F_{r1} < 4.5$	$5*(y_2 - y_1)$
III	$2.5 < F_{r1} < 4.5$	$2.7*y_2$
II	4.5	$3.6*y_2$
II	6	$4.0*y_2$
II	8	$4.2*y_2$
II	≥ 9	$4.3*y_2$
LFN	$1.7 < F_{r1} < 4.5$	$3*y_2$
I	$1.7 < F_{r1} < 2.5$	$6*y_2$

2 Total dissipated length according to graphs plotted from experimental data with the Froude number F_{r1} against a dimensionless ratio L/y_2).

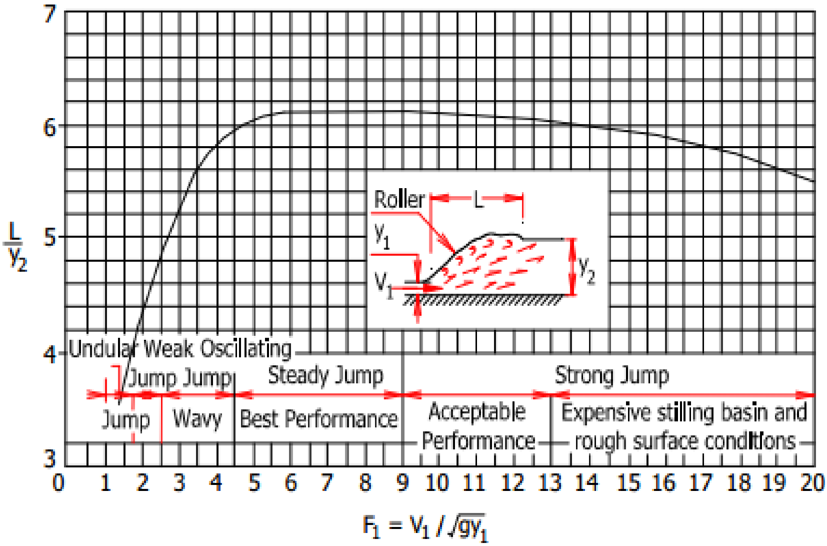


Figure 12.3: *dissipater length in terms of sequent depth y_2 of jumps in horizontal rectangular channels*

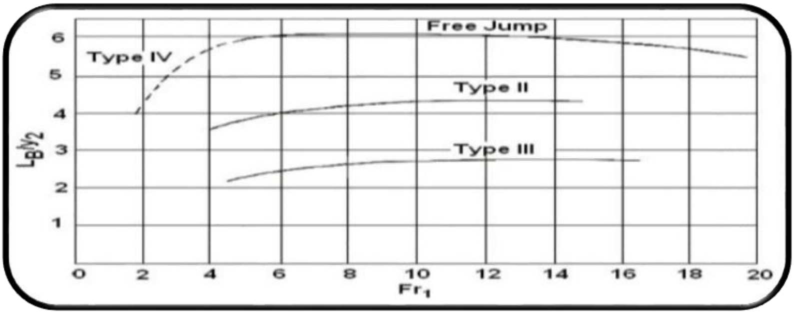


Figure 12.4: *dissipater length in terms of sequent depth y_2 (type II, III & IV)*

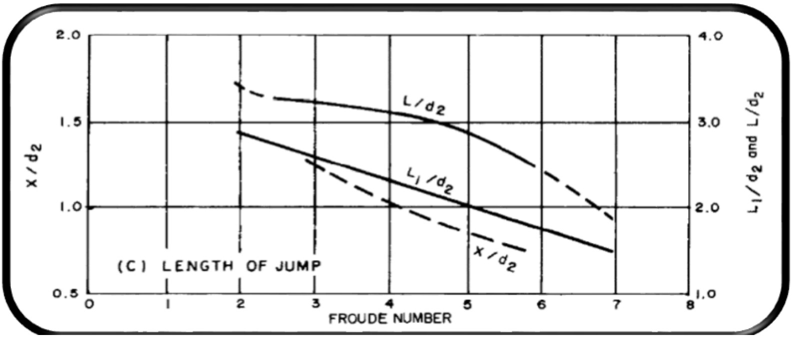


Figure 12.5: *dissipater length in terms of sequent depth y_2 (type LFN)*

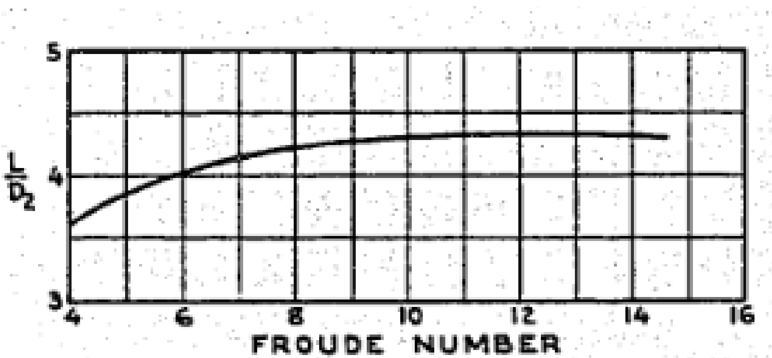


Figure 12.6: *dissipater length in terms of sequent depth y_2 (type II)*

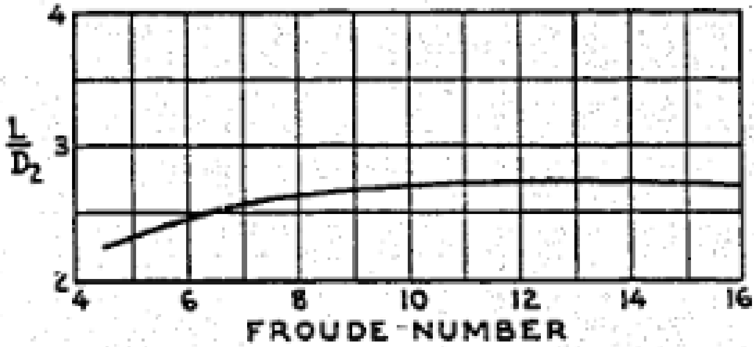


Figure 12.7: *dissipater length in terms of sequent depth y_2 (type III)*

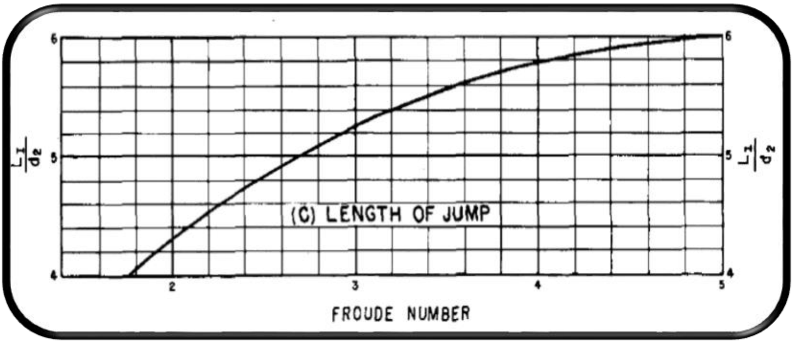


Figure 12.8: *dissipater length in terms of sequent depth y_2 (type IV)*

In practice, the stilling basin is seldom designed to confine the entire length of a free hydraulic jump on the paved apron because such a basin will be too expensive. Consequently, accessories to control the jump are usually installed in the basin (refer to figure 12.1).

12.4. STRAIGHT DROP

12.4.1. Introduction

The variables needed in the analysis for the inclined drop are much more complex than that of the vertical drop. The drop structures discussed here (refer to figures 12.1 and 12.9) require an aerated underside nappe to prevent a pulsating and fluctuating jet and, in general, for sub-critical flow in the u/s as well in the d/s channel.

A straight drop structure has a vertical wall between the control section and the stilling basin. The small portion of energy loss occurs by impact of the jet on the floor. The major portion of energy loss occurs by turbulence in the stilling basin.

The hydraulic problem is concerned with the dissipation of flow energy in the d/s basin. The aerated free falling nappe will strike the basin floor and turn d/s. Beneath the nappe, a pool is formed that supplies the horizontal thrust required to turn the nappe d/s (redirection of the flow). It is assumed first that there is no loss in specific energy head between the reservoir and the point where the jet strikes the basin floor (however, because of the impact of the nappe on the stilling basin floor once the jet strikes the floor, some energy is already lost). The main dissipation is obtained in the following hydraulic jump (for straight drop with jump) or the turbulence induced in a basin with the presence of impact blocks (straight drop with impact blocks).

12.4.2. **Aeration of nappe**

The negative pressure drags the lower side of the nappe towards the surface of the cross-regulator wall as shown in the opposite figure. If the atmospheric pressure exists beneath the nappe, it is known as a free nappe. A free nappe is obtained by ventilating a cross-regulator. With a partially ventilated nappe, the pressure below the nappe is negative (depressed nappe). Sometimes, no air is left below the water, and the nappe adheres or clings to the d/s side of the cross-regulator (clinging nappe or adhering nappe).

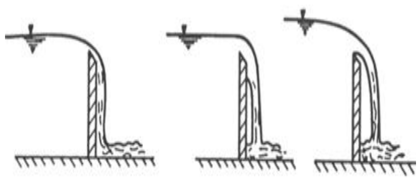


Figure 12.9: *weir ventilation and types of nappe: unaerated (clinging), partially aerated (depressed) and aerated (free) nappe*

The discharge depends upon the amount of ventilation and the negative pressure. Generally, the discharge of a depressed nappe is 6% to 7% more than that of a free nappe. If the flow clings to the d/s side, reducing the head on the cross-regulator (draw down), it gives a false value of discharge when it is put into the formula. The discharge of a clinging nappe is 25% to 30% more than that of a free nappe.

In a suppressed weir (weir width/$B_{structure}$ = 1), the sides of the structure may prevent air from circulating under the nappe (aeration is automatic in a contracted weir with weir width/$B_{structure}$ < 1). If used as a flow measurement, the air beneath the nappe may be exhausted, causing a reduction of pressure beneath the nappe with a corresponding increase in discharge rate for a given head[1].

12.4.3. **Drop number**

The flow geometry of the straight drop is described by functions of the dimensionless drop number as shown in the following equation:

$$\boxed{D = q^2 / g * Y^3} \quad (12.1)$$

Where:
D = drop number (dimensionless).
Y = drop height (crest elevation minus cistern elevation) (m).
Refer to list of symbols.

12.4.4. **Basin elevation**

The first approximation of the basin elevation for various basin widths is given in the design chart of the following figure. Because the basin is directly d/s from the crest, there is no loss in specific energy head between the reservoir and the point where the jet strikes the basin floor (scale A).

12.4.5. **Straight drop with hydraulic jump**

The aerated free-falling nappe of the jet strikes the floor of the stilling basin, reverses its curvature and turns into super-critical flow in the apron of the stilling basin. Consequently, a hydraulic jump can be formed d/s on the flat apron as determined by the tailwater depth (refer to figure 12.11).

The jump characteristics of the straight drop are basically the same as those for other jumps except that the position of the start of the jump cannot be determined as readily as it can for other basins. The position of the depth y_1 can be approximately determined by a straight line from the point on the axis of the nappe at the height of the pool depth joining the apron.

The value of y_1 and of F_{r1} at the start of the jump in relation to the drop number D are shown in the below figures. Theses relations are used in the User's Manual to determine the basin dimensions and characteristics. If the Froude number does not fall within an acceptable range, changes in either drop height or in the unit discharge should be considered.

12.4.6. **Straight drop with impact blocks**

The impact block type is considered when F_r is < 4.5. The dissipation of energy of the vertical drop structure with impact blocks is principally by turbulence induced by the impingement of the incoming flow upon the impact

[1] Suppressed weirs must have proper ventilation of the cavity underneath their nappes. This ventilation is commonly done by installing properly sized pipes in the walls to vent the cavity under the nappe. Standard equations and tables are valid only when sufficient ventilation is provided. The design of pipe size to introduce sufficient air depends upon the discharge, drop, and the loss of accuracy that is tolerable. Sizing air piping and air vents requires some knowledge of fluid mechanics and is difficult to do (outside the scope of the Manual).

blocks (refer to figure 12.11). The required tailwater depth is more or less independent of the drop height.

The length is considerably smaller than the straight drop with hydraulic jump. However, the foundation for an impact blocks basin must be of better quality because of the concentrated forces involved.

In practice, the straight drop with stilling basin has a much wider application than the straight drop with impact blocks.

12.5. END SILL

An end sill is effective in spreading the flow. The higher the end sill, the more effective it will be in spreading the jet. A higher end sill results in shallower depths in the d/s channel and possibly higher velocities over the d/s floor protection or channel bed.

The end sill should not be appreciably above the exit channel. For a sub-critical approach flow, the depth of flow is reduced. The flow drop may be enough to generate a critical flow transition and to form a secondary jump d/s.

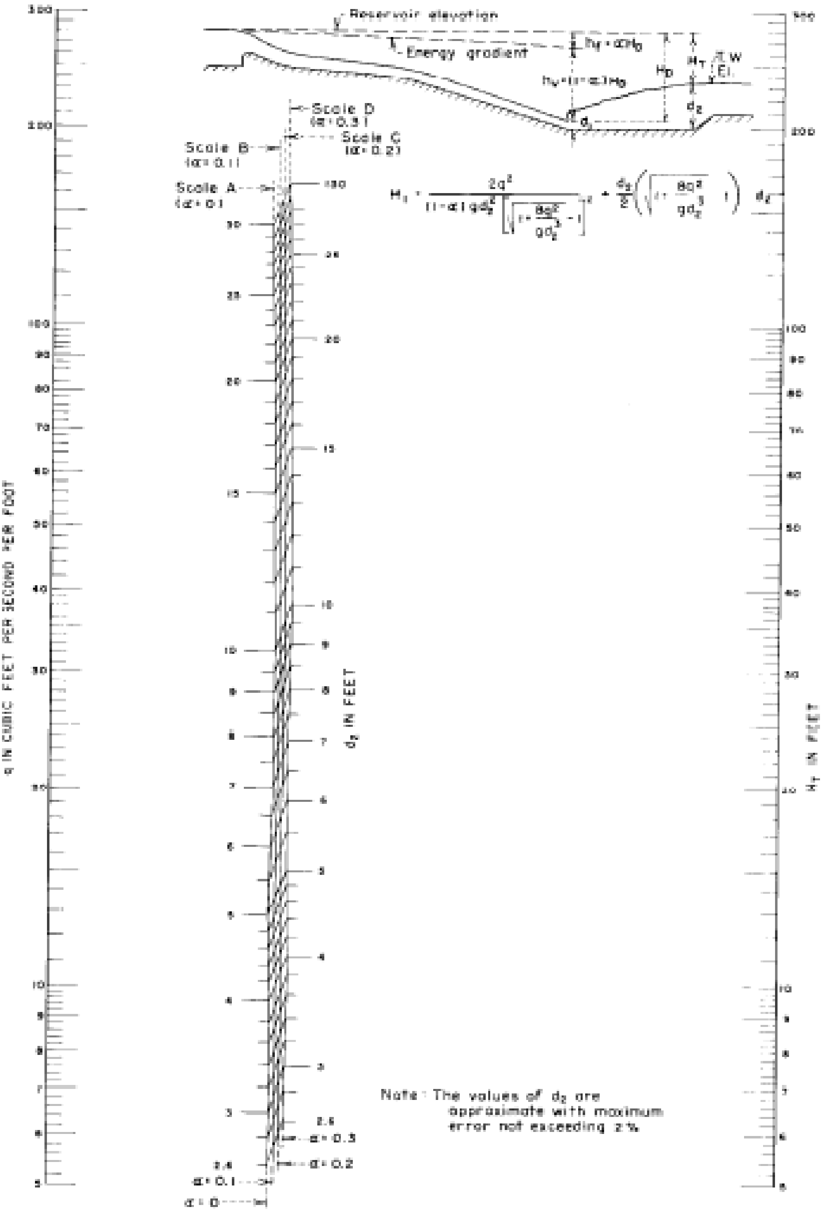


Figure 12.10: *USBR stilling basin depths versus hydraulic heads*

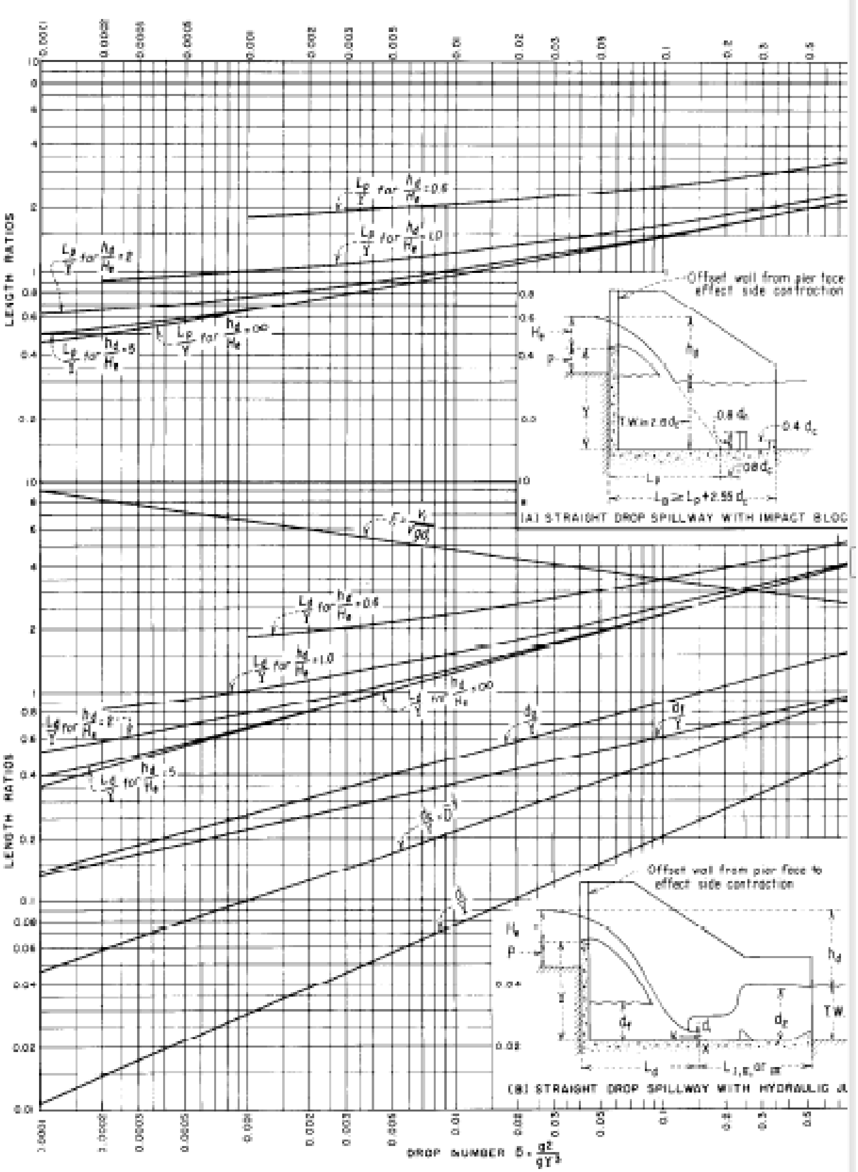


Figure 12.11: *USBR hydraulic characteristics of straight drops with hydraulic jump or with impact blocks*

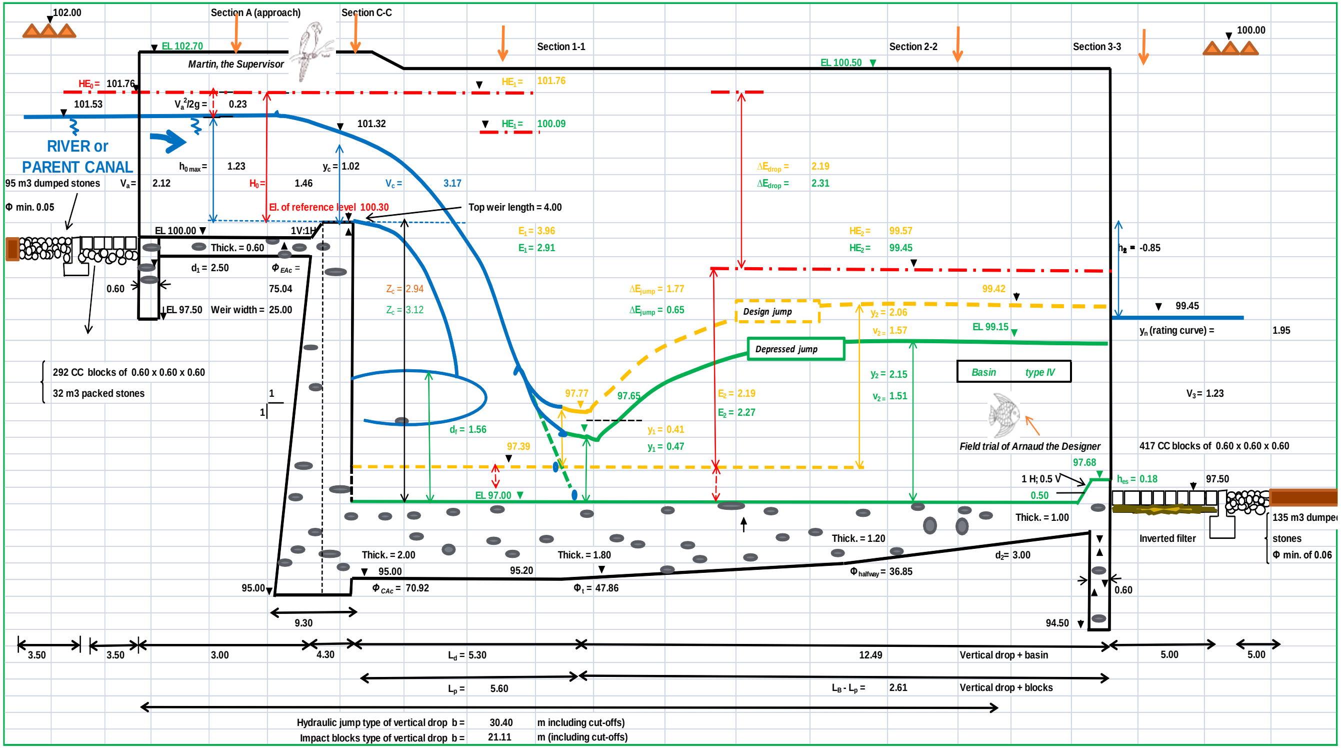


Figure 12.12a: *spreadsheet showing the general scheme for the hydraulic and structural design of a drop structure with straight drop*

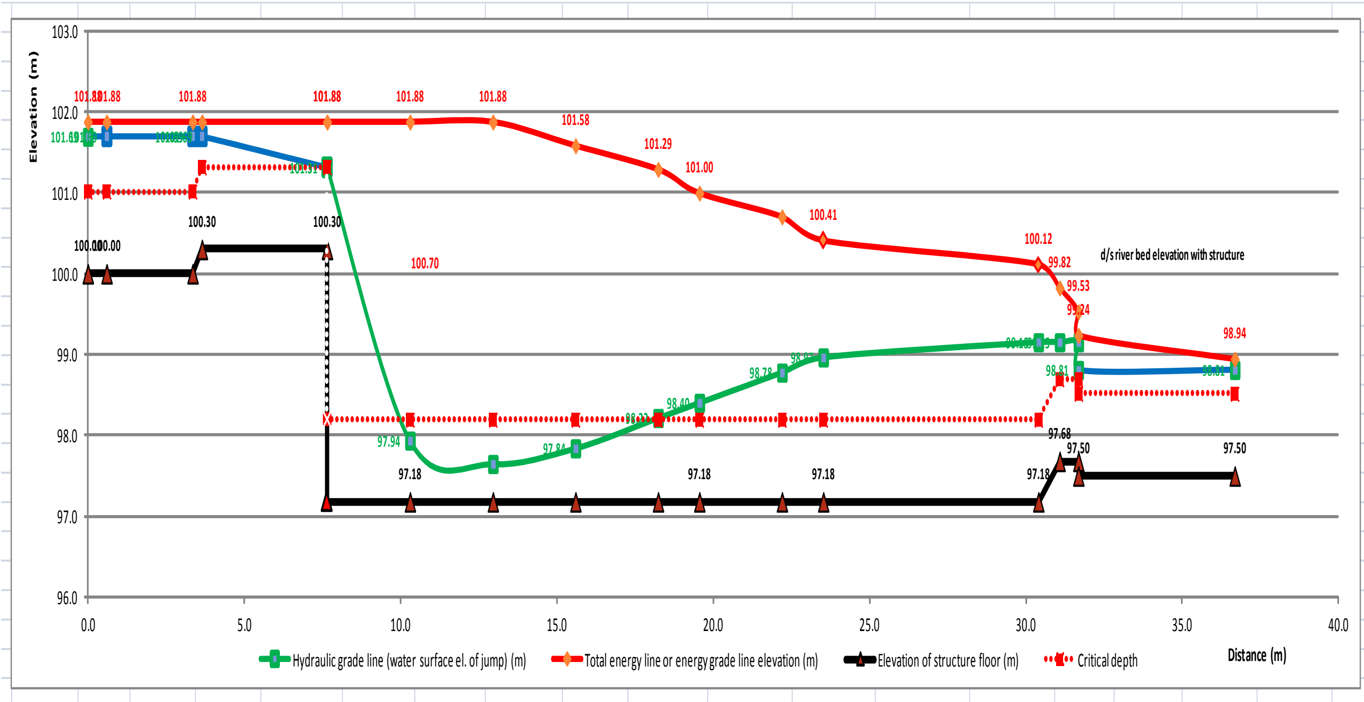


***Figure 12.12b**: spreadsheet showing structure floor (black), the water surface of jump (blue and green), the critical depth (dotted red) and the energy grade line (red) of a drop structure*

13.HYDRAULIC JUMP WATER SURFACE ELEVATION IN THE BASIN

13.1. INTRODUCCION

The knowledge of the free water surface profile of the jump is necessary to:

- Design the freeboard of the side walls of basin.
- Determine the water weight in the basin to counteract the uplift pressure under the structure.

In the User's Manual, the profile of the water surface of the hydraulic jump is determined for the high flood flow Q_{max} without taking into account the concentration and degradation factors.

13.2. JUMP WATER SURFACE ELEVATION IN THE SUB-CRITICAL REACH

The curves for plotting the post jump profile for different values of F_{r1} are given in the below figure

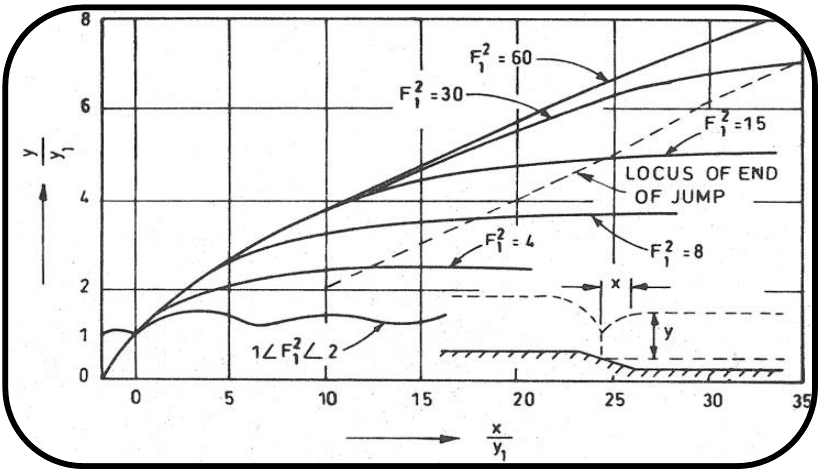


Figure 13.1: *design chart for plotting post jump water surface profile in the stilling basin*

By taking arbitrary values of x along the profile after the point of jump formation, x/y_1 can be tabulated. Corresponding to these values of x/y_1 for a known F_{r1}, different values of y_2/y_1 can be read out[1] of the design chart. Hence, different values of x and y_2 are known and the post jump profile can be slowly plotted.

[1] y from abacus corresponds to the sub-critical sequent depth of flow y_2 in the User's Manual.

It should be noted that the considered jump surface elevation occurs when the u/s flow has a Froude number of 2.5 or more. At lower F_r value, the jump tends to be either undular in nature or weakly developed (refer to figure 12.1).

13.3. JUMP WATER SURFACE ELEVATION IN THE SUPER-CRITICAL REACH

The water surface profile in the super-critical conveyance reach before the base point formation can easily be plotted with the help on the Montague's design chart (refer to figure 11.2).

At different points on the glacis corresponding to increasing values of x, the specific energy E_1 will go on increasing, being equal to the u/s specific energy level minus glacis level at the considered point (refer to table 25.1 with $\Delta Z \neq 0$ and $B_2 = B_1$).

For each value of E_1, a corresponding value of y_1 can be estimated from the curves. The values of y_1 will go on reducing till reaching the surface point of the jump formation.

PART V:

DESIGN OF CANAL HEAD REGULATOR AND FLOW CONDITIONS

14.WATER REQUIREMENT AND CONTROL AT HEAD REGULATOR

14.1. WATER SUPPLY

Water level and discharge control and measurement are required at all off-taking (branching) canals, as to control the discharge that leaves the parent canal or the river.

The discharge control method generally used is the supply oriented operation or d/s control. On the contrary, u/s control takes place when the water is released from the head work and distributes over the canals[1].

14.2. IRRIGATION REQUIREMENT AND DESIGN DISCHARGE

The required irrigation flow rate in the canal is calculated according to the cropped area, the water requirements of each crop and the effective rainfall.

The maximum irrigation water need adopted in the river basin is 2.51 l/s/ha for the design calculation, based on local practices and field losses.

The design of the head regulator (also called discharge regulators) considers the gate(s) fully open and takes into account the maximum flow rate passing down the cross-regulator in the river or the parent canal ($h_{0\,max}$ or $H_{0\,max}$).

14.3. GATE CHAMBER

In the User's Manual, the gate chamber comprises the piers and gate(s) system (open channel or breast wall type) and performs usually as an obstruction[2] in the open channel. The gate chamber is considered with a nominal horizontal (or nearly so) slope with or without the presence of a sill (weir)[3] below the gate (refer to article 16.4).

14.3.1. Pier

Piers are provided between each bay (refer to figure 2.2). The effect of the piers is to contract the flow and, hence, to alter the effective crest width of the weir. The piers are usually of mass concrete and founded on the floor.

Nominal reinforcement shall be provided at the faces as protection against surface cracking.

14.3.2. Under-flow gate

One spindle under-flow (under-shot) gate such as sluice gate is the most common type of gate.

[1] As opposed to a demand oriented operation or downstream control that maintains a constant water level at the d/s side of the structure, without regarding discharges. The effect is that the discharge at each regulator is automatically adjusted to the accumulated d/s demand for irrigation water.

[2] An obstruction in open channel flow presents a phenomenon very similar to that of a constriction, since both have the effect of reducing the cross-sectional area of the flow. However, the constriction reduces the cross-section into a single opening, whereas the obstruction creates at least two openings.

[3] The presence of a sill under the gate reduces the gate size and sediments flowing d/s. Water depth can be affected (refer to table 25.1).

The illustration of terminology of a sluice gate is given by the following figure:

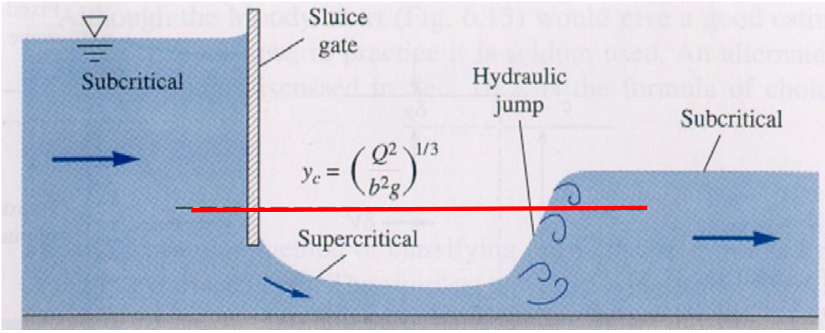


Figure 14.1*: illustration of the sluice gate terminology with mild slope or horizontal floor*

The advantages of the vertical gate with under-flow as a discharge or as a water level regulator are: the structure is relative cheap, simple and sturdy; it provides a reasonable constant discharge for varying u/s water levels in the parent canal.

The disadvantages of regulators with vertical gates are: the gate does not pass floating debris (refer to figure 16.4) and the discharge cannot be always measured accurately (refer to article 17.5.4).

14.3.3. Approach flow velocity

The velocity to be considered is the approach (or accelerated) velocity V_a at the entry of the structure (Manning's equation). For solving the problems in hydraulic engineering, the velocity used is the average velocity of flow over a section.

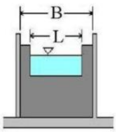
Permissible limits of velocity are given in the table 6.1. However, the conveyance reach u/s of the chamber should have a sub-critical flow ("normal" flowing water) as to limit the head loss. It means that $F_r < 1$ (refer to article 6.4.2 and equation 6.2a)).

14.4. EFFECTIVE OR CLEAR WATERWAY

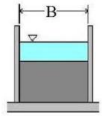
14.4.1. Pier(s) contraction

For a gated structure, the effective waterway B_{cl} has to be taken into consideration instead of the structure waterway $B_{structure}$ between abutments in order to account for the obstruction (horizontal contraction) due to the presence of the pier(s) of the gate chamber (refer to figure 2.2).

This horizontal contraction of the flow alters the effective crest width (transverse to the water flow) of the gated structure. A weir in which the crest width equals the channel width is referred to as suppressed and the number of side contractions N = 0. If both sides of the weir are far enough removed from the sides of the approach channel (end contraction of the weir), the weir is considered to be contracted (or unsuppressed), and N = 2; if one side is suppressed and one is contracted (or unsuppressed), N = 1.



Contracted Rectangular Weir



Suppressed Rectangular Weir

The effective or clear width of the crest is given by the following equation:

$$\boxed{B_{cl} = B_{structure} - W_{pier} - K * N_{sc} * h_0} \quad (14.1)$$

<u>Where</u>:
K = pier contraction coefficient.
W_{pier} = total with of piers (transverse to the water flow) (m).
N_{sc} = number of side contractions (2 for each gated bay; 0 for suppressed side contraction weir).
Refer to list of symbols.

The pier contraction coefficient K varies mainly with the shape and position of the pier nose and the head conditions. In the User's Manual, the u/s nosing and d/s end of pier are curved to ease the flow. Values of the contraction coefficient are given in the following table for different pier shapes:

Table 14.1: *pier contraction coefficients and shapes*

Pier shape	K
Square nosed pier without any rounding	0.10
Square nosed pier with rounded corners	0.02
Rounded nose pier	0.01
90° cut nosed pier	0.01
Pointed nose pier	0.04

With one open gate adjacent to closed gates, these values of K become roughly 2.5 times larger.

While designing the head regulator, the open water width of the structure is chosen according to the obstruction as shown in the following table:

Table 14.2*: obstruction and waterway*

Obstruction	Flow regulation	Open water width		Hydraulic coefficients
		Name	Width	
None	Unregulated	$B_{structure}$	Same or less than channel or river	Fairly constant
	Unregulated	B_{cl}		

Pier(s) & bay shape			$B_{structure}$ minus obstruction effects of pier(s) in gate chamber	Related to pier(s) shape (horizontal contraction)
Pier(s) & bay shape & gate(s)	Regulated	B_{cl}	$B_{structure}$ minus obstruction effects of pier(s) in gate chamber	Related to pier(s) shape and gate(s) lip and position (horiz. and vertical contractions)

However, a minimum width of the off-take has to be considered to avoid super-critical flow at the entrance and the possibility of an u/s jump (refer to figure 14.4b and table 14.3).

14.4.2. **Pier length and critical depth**

In the User's Manual, the minimum length of the pier is equal to 2 m, taking into account the gate chamber and the 1 m minimum width operating platform; the d/s edge or brink of the pier corresponds to the beginning of the glacis (u/s inflexion point).

With the presence of pier(s), the calculation of the critical depth y_c takes into account the effective or clear waterway B_{cl} and is given in the following equation.

$$\boxed{\mathbf{y_c = (q^2 / g)^{1/3}}} \text{ (14.2) with } \boxed{\mathbf{q = Q / B_{clear} = y * V}} \text{ (14.3)}$$

<u>Where</u>:

Refer to list of symbols.

Therefore, the d/s edge or brink of the pier must correspond to the beginning of the glacis (u/s inflexion point) from which $B_{structure}$ is taken into account in the calculations.

14.5. GATE OPENING, ENERGY AND MOMENTUM

Under-flow gates operate in a variety of flow modes including the weir flow when the gate(s) are out of the water and the orifice flow for normal operation.

The following figure of the hydraulic jump d/s of a gate outlet gives a clear idea about how conservation of energy and conservation of momentum apply in a gate chamber with rectangular cross-section and a constant unit discharge q (refer to article 9.3).

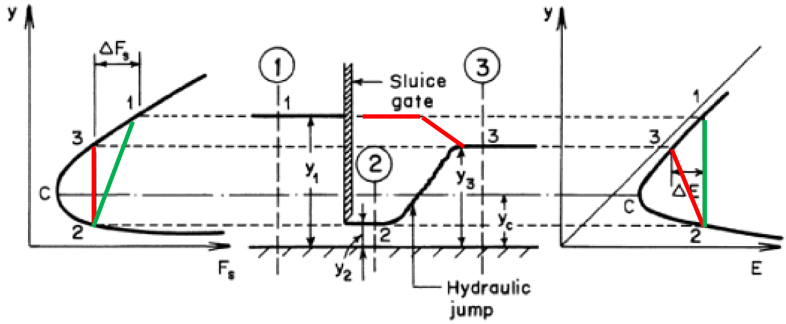


Figure 14.2: *hydraulic jump, specific energy and specific force diagrams of an under-flow gate with rectangular cross-section*

As shown in the centre of the above figure, a deep u/s flow (position 1) encounters a sluice gate. The gate imposes a decrease in flow depth at position 2 and a hydraulic jump occurs between the positions 2 and 3 (note that the gate opening w is lower than the critical depth y_c).

The left part of the figure shows the specific force diagram of these three positions, while the right part shows the specific energy diagram for these same three positions. The energy loss can be neglected between the positions 1 and 2 ($E_1 = E_2$)[1], but the external thrust on the gate causes significant specific force loss ($F_{s1} > F_{s2}$). By contrast, between the positions 2 and 3, turbulence in the hydraulic jump dissipates energy ($E_3 < E_2$), while the momentum is assumed to be conserved (same specific forces $F_{s2} = F_{s3}$), provided the channel bottom slope or the datum are the same.

In other words, the energy-momentum applies with the energy equation from position 1 to position 2 and the momentum equation from position 2 to position 3.

If we know the unit discharge q and the flow depth at position 1, by applying energy conservation between the positions 1 and 2 and momentum conservation between 2 and 3, the flow depths at position 2 (y_1) and 3 (y_2) can be computed.

From the above figures, it can be inferred:

- Total energy line (total mechanical energy): neglecting friction losses (short distance), there is no energy loss as water flows under the gate (refer to opposite figure) and the Bernoulli's equation applies[2]. This is reasonable because the flow is

[1] The specific energy remains constant for idealized gate(s) with negligible frictional effects.

[2] On the contrary, in the hydraulic jump, momentum principle applies (refer to article 10.3.4).

converging (contracting) under the gate and this tends to suppress turbulence which means little or no energy loss (turbulence is the main cause of energy loss).

- There is a significant change in the components of the total mechanical energy across the gate even though the total is the same. Upstream flow is slow and deep whereas d/s flow is very shallow and fast but the discharge is the same on both sides of the gate.

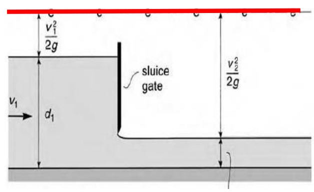


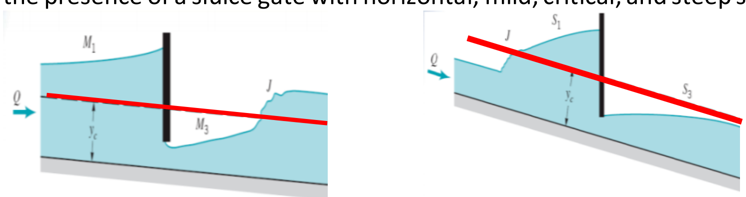
***Figure 14.3**: total energy line (or energy grade line) of a sluice gate*

- Energy dissipation d/s of gate(s): it takes place in the hydraulic jump[1]. The hydraulic jump has a tendency to move towards the gate because the normal depth y_n in the d/s channel is usually greater than the sequent depth y_2 (refer to article 10.5.3).
- Energy dissipation u/s of gate(s): with a super-critical flow u/s of the gate(s), the transition from super-critical to sub-critical flow takes place u/s of the gate(s). A hydraulic jump occurs before the gate(s) (refer to figures 14.4b and 14.4c). No hydraulic jump occurs d/s of the gate because the flow is going from one super-critical flow to another super-critical flow (steep slope); there is no d/s transition between super-critical and sub-critical flow.

14.6. WATER SURFACE PROFILES WITH GATE

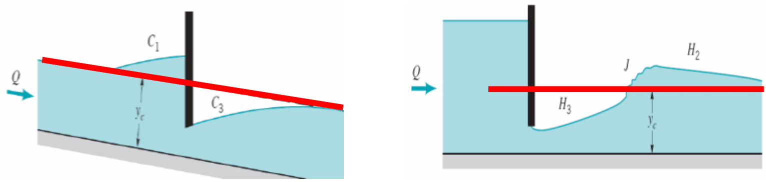
14.6.1. Introduction

The following figures show the different water surface profiles encountered with the presence of a sluice gate with horizontal, mild, critical, and steep slope:



(a) Mild slope (b) Steep slope

[1] It is usually not a well-formed jump. The basin often has entering Froude number less than 4, which means that the jump can be weak or oscillating between the bottom and the water surface, resulting in irregular wave formation propagating downstream.



(c) Critical slope (d) Horizontal slope

***Figure 14.4a, b, c, d**: individual water surface profile with a gate*

As shown in the above figures, the critical depth remains the same in the channel because it depends only on discharge and not on slope.

Generally of most interest to the design engineer is the situation where GVF occurs as a result of an obstruction (fallen tree, gates, weirs etc.) that raises the water surface above the uniform flow normal depth line u/s of the obstruction[1], as shown in the following figure (refer also to figures 25.1a & 25.1b):

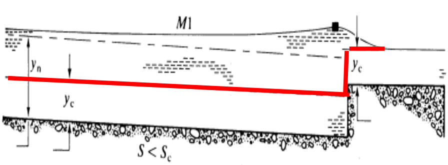


***Figure 14.5**: water surface profile due to obstruction*

This flow profile represents a backwater curve as the depth of flow increases continuously in the direction of flow. This happens only once critical conditions are achieved on the sill in the gate chamber (refer to article 25.2.1).

14.6.2. Surface flow profiles encountered with a gate

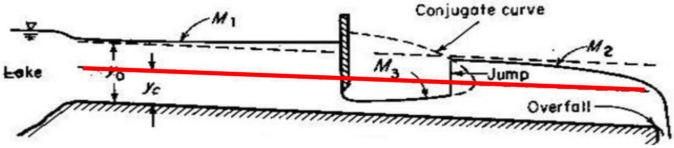
The following table and figures give the type of curves illustrated in the above figures that can be generated by a sluice gate placed across the channel, so that it produces a super-critical flow under the gate.

***Table 14.3:** description of flow profiles encountered with the presence of a sluice gate with regulated flow*

Profiles	Description
M_1	Sub-critical flow. Represents the backwater curve on a mild slope M (for example, occurs u/s of a gate)
M_2	Sub-critical flow. Represents the drawdown curve on a mild slope M (for example, occurs u/s of a sudden enlargement or overfall)
M_3	Super-critical flow. Starts at a vertical angle slope and terminates with a hydraulic jump[2]

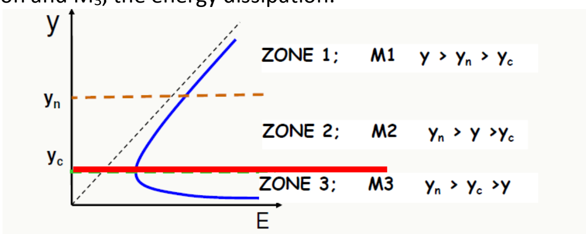
[1] When analysing the GVF with the Manning equation, the longitudinal energy slope must be used instead of the longitudinal bed slope.

[2] In other words, M_3 is the gradually varied flow (GVF) curve leading to the alternate depth y_1, followed by the rapidly varied flow (RVF) leading to the sequent depth y_2 (for example, occurs when a super-critical flow enters a mild slope or d/s of a sluice gate)

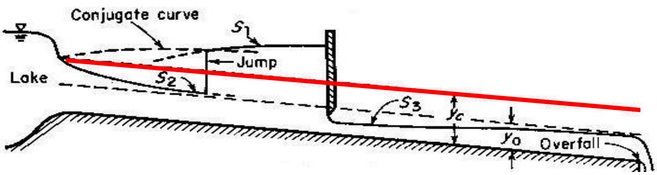


(reach with mild slope M)

The lowering of the gate on a sub-critical flow in a horizontal or mild reach frictionless rectangular section produces 3 surfaces profiles: M_3 immediately d/s of the gate which then undergoes a hydraulic jump to sub-critical M_2 GVF curve which depends upon the d/s control (overfall here). It could be a M_1 profile also (instead of the local overfall control, a very long channel gives a friction control ensuring uniform flow with the normal depth being controlled by the friction of the channel). Upstream of the gate, the initial/s portion of a M_2 curve evolves to an M_1 curve as the lowering the gate progresses. This M_1 curve represents in fact the energy built up in front of an obstruction and M_3, the energy dissipation.

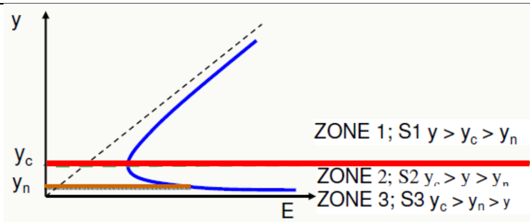


S_1	Sub-critical flow. Begins with a hydraulic jump at the u/s and becomes tangent to the horizontal pool level at d/s end.
S_2	Super-critical flow. Drawdown curve.
S_3	Super-critical flow. Formed between an issuing super-critical flow and the normal depth line to which the profile is tangent (for example, d/s of a sluice gate in a steep slope when the gate opening $w < y_c$).
J	Hydraulic jump.



(reach with steep slope S)

The lowering of the gate (artificial control) on a super-critical flow in a steep reach frictionless rectangular section backs up water to form a S_1 profile on the u/s side with the formation of a jump.



14.7. FLOW BEHAVIOURS THROUGH A GATE

The flow behaviour associated with gated structures (open channel or breast wall type) is often complex under real-time conditions and the rating curve is not straight forward.

14.7.1. Regulation and flow types

Hydraulically speaking, the under-flow gate(s) of the gate chamber acts like an orifice when regulating the flow[1] (refer to articles 16) and like a non orifice when not regulating the flow[2] (refer to articles 15). With unregulated flow, a critical water level control section may or may not exist in the chamber, depending of the magnitude of the specific energy at the contraction.

Basically, there are three types of flow behaviours through a gated structure, as shown in the following table. The identification of the flow type is crucial for the selection of the correct equation to be applied.

Table 14.4: *flow behaviours through* a structure with a gated discharge regulator

Regulation	Types	Remarks
Unregulated (non orifice flow)	Free	Rectangular broad or short crested weir state if presence of gate sill; horizontal constricted or obstructed open channel flow if no gate sill ($F_r > 1$)
	Submerged	Tailwater quite high ($F_r < 1$)
Regulated (orifice flow)	Free	Outflowing jet open to atmosphere ($F_r > 1$)
	Partially submerged	Outflowing jet still super-critical ($F_r > 1$); some influence of the tailwater
	Fully submerged	Outflowing jet sub-critical ($F_r < 1$); influence of the tailwater

14.7.2. Pivot table

The different criteria used in the Manual for flow regulation and for distinguishing the conditions, types and state of flows below a gate or over a weir are summarized in the following pivot table:

[1] The gate bottom interferes with the water flow.
[2] The gate bottom is above the water surface.

Table 14.5: *flow regulations, types, conditions and statess in relation to the adopted pivot*

<table>
<tr><th rowspan="2">Flow</th><th colspan="2">Pivot</th><th rowspan="2">Reference</th></tr>
<tr><th>Orifice flow</th><th>Non orifice or weir flow</th></tr>
<tr><td colspan="4">Regulation</td></tr>
<tr><td>Regulated</td><td>$h_0 > 1.1*w$</td><td>Not applicable</td><td rowspan="3">Articles 15 & 16</td></tr>
<tr><td>Unregulated</td><td>Not applicable</td><td>$h_0 \leq 0.9*w$</td></tr>
<tr><td>Semi-regulated</td><td colspan="2">$0.9*w \leq h_0 < 1.1*w$</td></tr>
<tr><td colspan="4">Types</td></tr>
<tr><td>Free</td><td>$h_2/w <$ modular limit*0.9</td><td>$h_2/h_0 < 75\%$</td><td rowspan="3">Articles 14.7 & 16.2.2</td></tr>
<tr><td>Submerged</td><td>$h_2/w >$ modular limit*1.1</td><td>$h_2/h_0 > 75\%$</td></tr>
<tr><td>Transitional</td><td colspan="2">0.9*modular limit $< h_2/w <$ 1.1*modular limit</td></tr>
<tr><td colspan="4">Conditions</td></tr>
<tr><td>Orifice</td><td>$h_0 > w$</td><td>Not applicable</td><td rowspan="2">Articles 15 & 16</td></tr>
<tr><td>Non orifice or weir flow</td><td>Not applicable</td><td>$h_0 \leq w$</td></tr>
<tr><td colspan="4">State</td></tr>
<tr><td>Cannot be used as broad crested measuring device</td><td>Not applicable</td><td>$H_0/L < 0.08$
$H_0/L > 0.33$</td><td>Article 6.4</td></tr>
<tr><td>Can be used as broad crested measuring device</td><td>Not applicable</td><td>$0.08 \leq H_0/L \leq 0.33$ $(0.08 < H_0/L < 0.15)$</td><td>Article 6.4</td></tr>
</table>

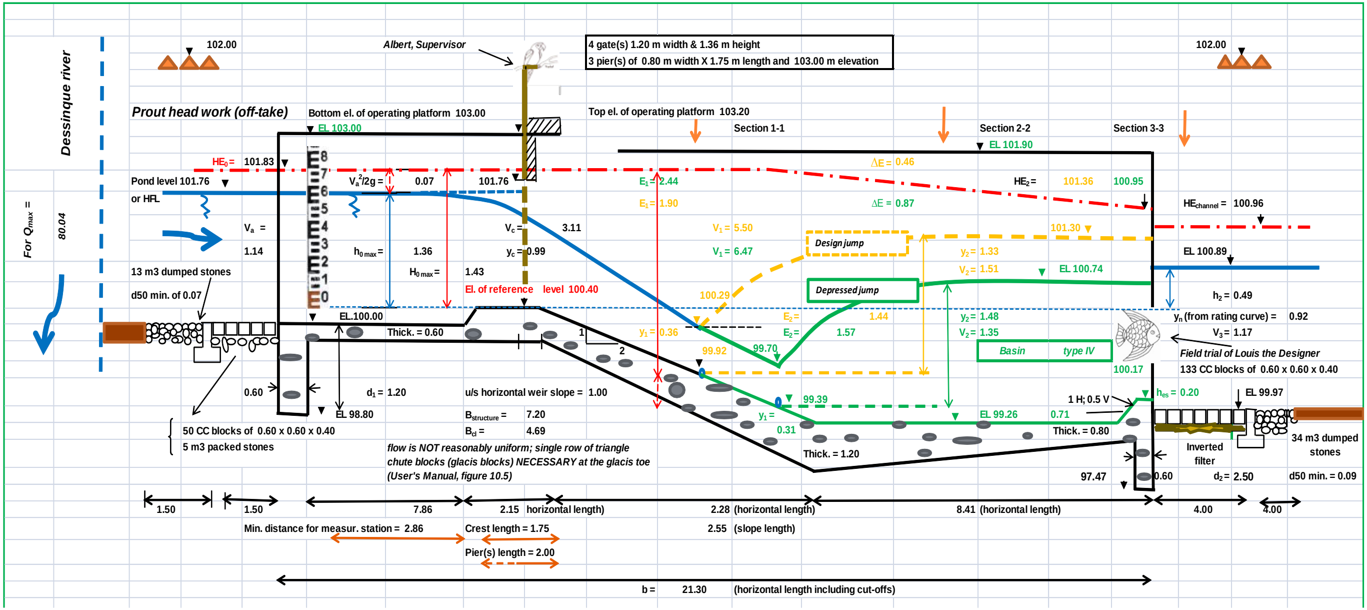


***Figure 14.6a**: spreadsheet showing the general scheme of the hydraulic and structural design of a gated head regulator*

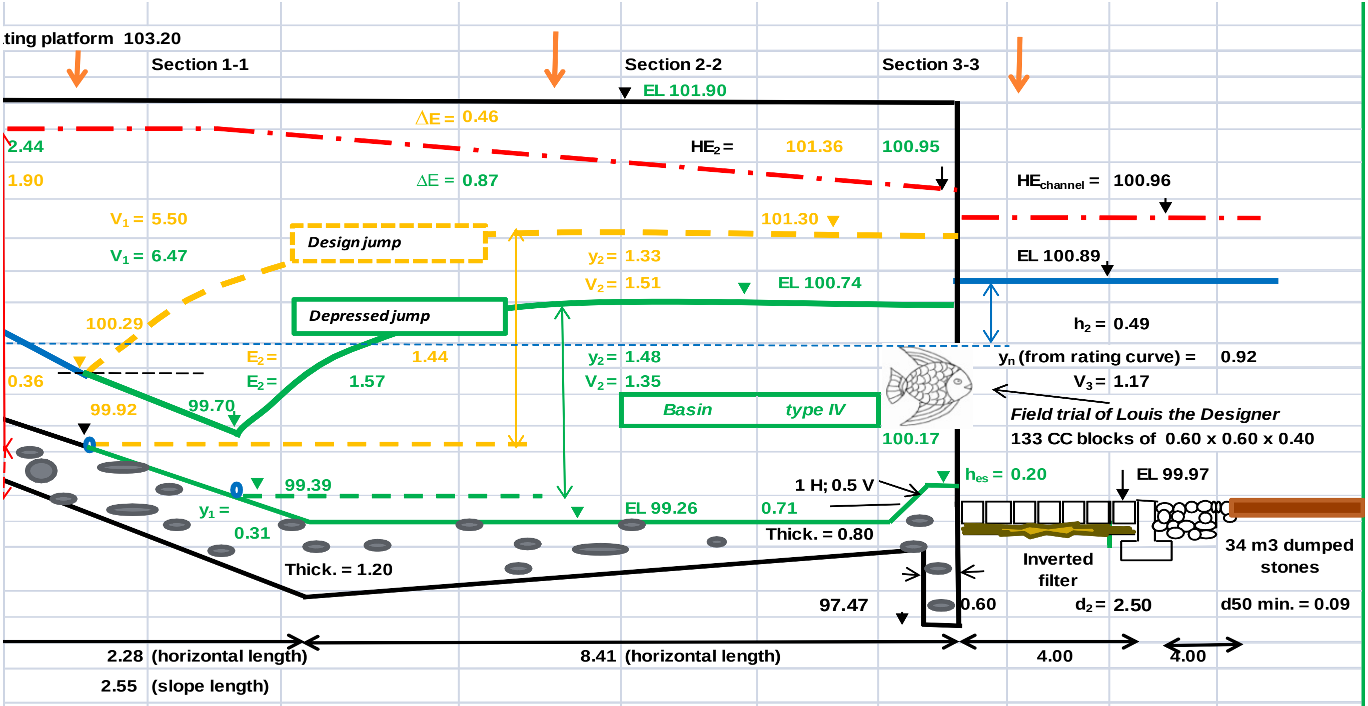


***Figure 14.6b**: spreadsheet showing a detail of the general scheme of the hydraulic and structural design of a gated head regulator*

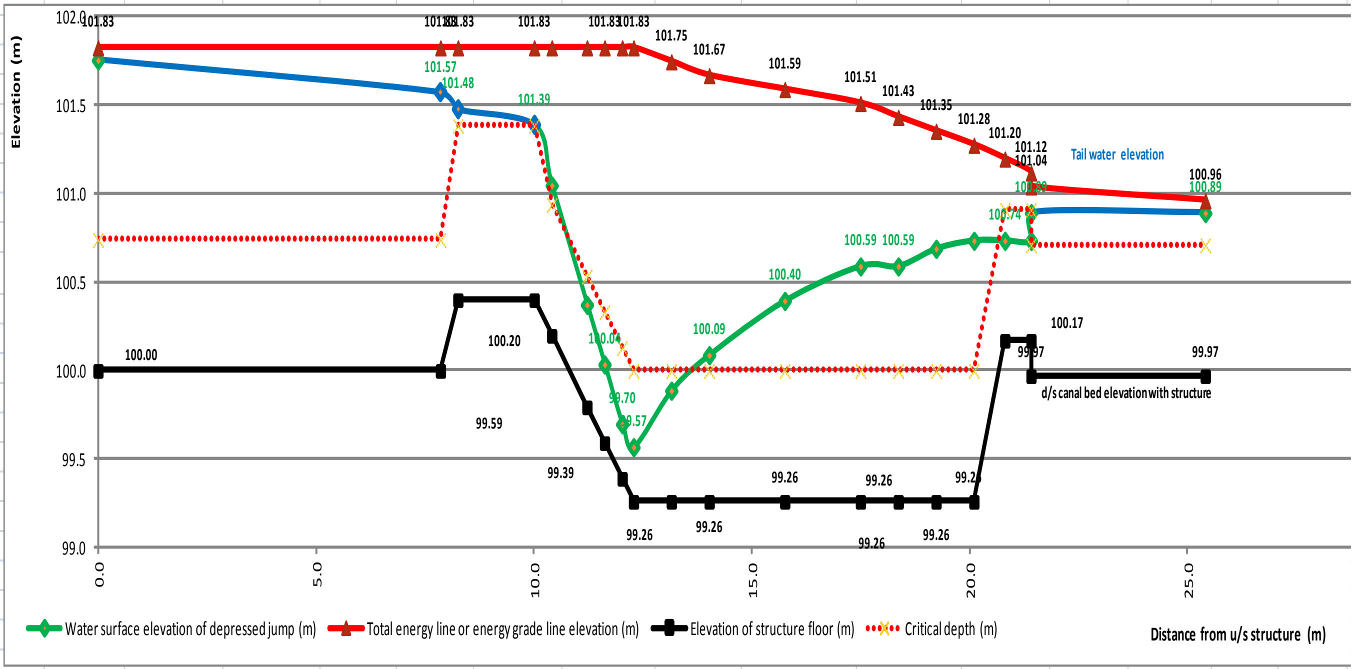


***Figure 14.6c**: spreadsheet showing structure floor (black), the water surface of jump (blue and green), the critical depth (dotted red) and the energy grade line (red) of a gated head regulator*

15.GATE(S) OPENING WITH NON ORIFICE FLOW CONDITIONS

Since the gates are used for flow distribution, the gate opening is, at some height, equivalent to a non orifice flow.

The hydraulic coefficients used in the discharge equations account for:

- Approach velocity head.
- Approach velocity distribution.
- Decrease in jet velocity caused by friction.
- Amount of jet contraction caused by the flow curving around the corner of the orifice perimeter.
- Flow states.

15.1. CASE Nº1: GATE LOWERED INTO A RECTANGULAR HORIZONTAL CANAL ABOVE FLOW

15.1.1. Outflow conditions with $h_0 < w$

When the gate is kept above the u/s water head h_0 (in relation to weir sill if existing), the gate(s) is not able to regulate the flow (refer to figure 16.8).

The flow through a one-gate chamber performs hydraulically as an open channel flow with constriction (refer to article 25.2); in case of more than one gate, the chamber performs as an open channel flow with obstruction.

15.1.2. Outflow discharge equation for non orifice conditions

The flow beneath the gate is described by the horizontal broad crested or short crested weir equations (refer to equation 6.1a and figure 6.1).

Free flow type

With fully open gate(s) (refer to figure 16.8), the free water discharge available through the gated bay(s) (with pier if more than 1 gate) is given by the discharge formula of a rectangular weir as shown below:

$$\boxed{Q = C_d * C_v * B_{cl} * h_0^{3/2}} \quad (15.1)$$

<u>Where</u>:

Q = discharge through gate(s) (m^3/s).

Refer to list of symbols.

The product of $C_d * C_v$ is called the effective discharge coefficient C_e (refer to articles 6.1). For practical reasons, the energy head H_0 is replaced by the water head h_0 (refer to equation 6.1b and article 6.1.1).

Submerged flow type

In case of submerged flow, the head discharge relationship can no more be applied and the d/s water surface elevation must also be measured.

With fully open gate(s) (refer to figure 16.8), the water discharge available through the gated bay(s) section is given by the discharge formula shown below:

$$\boxed{Q = C_d * C_v * B_{cl} [2 * g (h_o - h_2)]^{0.5}} \quad (15.2)$$

Where:
Q = discharge through gate(s) (m^3/s).
Refer to list of symbols.

The product of C_d*C_v is called the effective discharge coefficient C_e (refer to article 15.1.3 and 15.1.5).

Submerged flow in the structure will always decrease accuracy of flow measurement (refer to article 17.5.4).

In the User's Manual, discharges under drowned conditions are also obtained by applying a flow reduction factor f to the free flow discharges (equation 15.1) as shown below:

$$\boxed{Q_{submerged} = f * Q_{free}} \quad (15.3)$$

Where:
$Q_{submerged}$ = submerged discharge through gate(s) (m^3/s).
Q_{free} = free discharge through gate(s) (m^3/s).
f = flow reduction factor.

This correction factor f depends upon the u/s and d/s slope of the weir faces as pictured below and the submergence ration as shown in the following table:

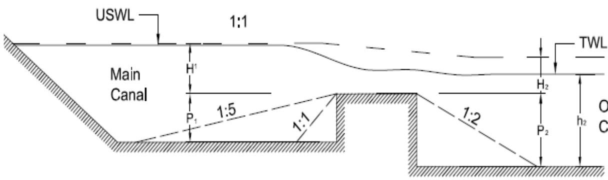


***Figure 15.1**: u/s or d/s vertical or sloping weir faces of off-taking canal*

***Table 15.1:** flow reduction factor for submerged (non modular) flow with broad crested weir*

Weir shape (H:V)	Submergence ratio	f
u/s or d/s vertical or sloping weir faces	≤ 0.75	1.0
u/s & d/s vertical faces	0.80	0.95
	0.85	0.88
	0.90	0.75
	0.95	0.57
	0.80	≈ 1

u/s face 1:1 & d/s face 2:1	0.85	0.98
	0.90	0.90
	0.95	0.73
u/s face 5:1 & d/s face 2:1	0.80	≈ 1
	0.85	0.95
	0.90	0.82
	0.95	0.62

Transitional flow type

In case of transitional type of flow, the flow is considered as semi-regulated and can be free or submerged according to the d/s conditions. The transitional flow type is calculated as orifice flow ($h_o > w$) or non orifice flow ($h_o \leq w$) in the User's manual (refer to table 14.5, conditions).

15.1.3. **Effective discharge coefficient**

The effective discharge coefficient depends upon different parameters such as the u/s water depth, the contraction coefficient and the approach velocity.

In practice, for free flow type, C_e is taken as 1.7 for rectangular broad crested weir state or 1.84 for short crested weir state (refer to article 6.1.2).

15.1.4. **Contraction coefficient**

In the case of non orifice flow, the contraction corresponds to the amount of curvature of the jet in the direction of flow due to the geometry of the piers (horizontal contraction) and is reflected in the effective waterway B_{cl} (horizontal contraction) (refer to tables 14.1 and 14.2).

15.1.5. **Velocity coefficient**

In the User's Manual, the practice to calculate the discharge rate above the weir is by measuring h_0 instead of H_0, taking into account that the u/s water surface elevation in the canal equals the u/s water surface elevation in the river or parent canal. By using h_0 instead of H_o, the correction coefficient C_v for neglecting the velocity of approach head has to be introduced with C_v is ≥ 1 (refer to article 6.1.3).

However, in irrigation systems, C_v can be assumed to be unity since most of the canals have a very flat hydraulic gradient and the flow velocities very low.

If significant u/s approach velocity occurs, C_v may be approximated from the following equation for a rectangular cross-section:

$$\boxed{\mathbf{C_v = (H_0 / h_0)^{1.5}}} \quad (15.4)\ (6.1d)$$

<u>Where</u>:

Refer to list of symbols.

16. GATE(S) OPENING WITH ORIFICE FLOW CONDITIONS

The gate(s) installed in the chamber creates orifice flow conditions when the edge of the gate(s) goes under the water surface. Between regulated and unregulated flow, the flow type is semi-regulated: the edge of the gate(s) touches or slightly goes under the water surface (refer to table 14.5).

The hydraulic coefficients used in the discharge equations account for:

- Approach velocity head.
- Approach velocity distribution.
- Decrease in jet velocity caused by friction.
- Amount of jet contraction caused by the flow curving around the corner of the orifice perimeter.
- Flow state.

16.1. CASE Nº2: GATE(S) LOWERED INTO A RECTANGULAR HORIZONTAL CANAL TO A HEIGHT BELOW CRITICAL DEPTH WITH FREE FLOW TYPE D/S OF THE GATE(S)

16.1.1. Flow description downstream of the gate(s)

When the gate(s) is progressively lowered ($h_0 \geq 1.1^* w$), the velocity of flow d/s of the gate(s) becomes larger and larger and, eventually, super-critical flow will occur immediately d/s of the gate(s) with the formation of a hydraulic jump[1] when the initial and sequent depths y_1 and y_2 and the approaching Froude number satisfy the equation 10.4 (refer to figures 14.1 and 14.3). Provided the jump has not reached the gate (submerged flow), the water depth u/s of the gate remains constant due to the super-critical flow d/s of the gate. In other words, the control is provided by the structure itself and the u/s conditions.

The tailwater depth y_n however plays a very influential role on where the jump will occur in the channel, and changes in its depth can shift the jump either u/s or d/s (refer to table 10.1).

If the gate opening w is reduced too much, it can represent a temporary choke condition since the energy may not be sufficient to pass the required amount of discharge per unit width. An increase in u/s water depth occurs (with a surface profile M_1; refer to table 14.3 and figure 14.4) with an instantaneous reduction of the unit discharge[2] due to the change in geometry.

[1] Super-critical outflow below the gate occurs as long as the roller of the hydraulic jump does not submerged the section of minimum depth of the jet that is located at a distance from 1.54 to 1.60 the height of the gate opening.

[2] After an instantaneous reduction in unit flow rate q due to the closing of the gate, q steadily increases, eventually reaching the same steady state discharge as before the movement (closing) of the gate (the time that it takes to get back to steady state discharge can be calculated). With the discharge Q remaining constant, the discharge per unit width q within the contraction must increase again to reach the initial q, so that the critical depth y_c must also increase. Therefore, the specific energy ($E = 3/2 * y_c$) increases (within the contracted section and u/s of the gate), so that the u/s water depth increases and a M_1 curve appears u/s of the gate. This behaviour follows well

16.1.2. **Free flow discharge equation**

The flow under the gate is very similar to orifice flow but not quite. First, the flow contracts on its upper surface as it goes under the gate (and lower surface with the presence of a gate sill) and, second, there is additional friction from the bed of the channel. So to find a formula for discharge for this structure, the orifice formula is a good starting point, but it needs modifying thanks to the introduction of coefficients.

The formula for discharge from an orifice modified for a gate with free outflow is given by the following equation:

$$\boxed{Q = C_d * C_v * w * B_{cl} [(2 * g (h_0 - C_c * w)]^{0.5}} \quad (16.1)$$

or

$$\boxed{q = C_d * C_v * w [(2 * g (h_0 - C_c * w)]^{0.5}} \quad (16.2)$$

<u>Where</u>:
Q = discharge through gate(s) (m^3/s).
Refer to list of symbols.

This discharge through a gate in orifice conditions with free outflow is governed by the u/s flow depth h_0 and the gate opening w.

Due to the square root of the u/s water head h_0 in this and following equations, under-shot gates are well suited for controlling the d/s flow by changing the gate(s) height. Contrary to the weir, variation in the u/s water head h_0 has little effect on the flow rate (refer to figure 17.4).

16.1.3. **Contraction coefficient**

Orifices may be partially contracted in two senses: one is the amount of curvature of the jet in the direction of flow due to the piers (horizontal contraction; refer to article 15.1.4) and the other in the amount of orifice opening perimeter which produces less or no curvature of the outflowing jet passing through the opening for greater gate openings[1] (vertical contraction).

The contraction coefficient C_c varies with the relative gate opening and the relative submergence[2]. The passage from orifice (regulated) to non orifice

with the intuitive notion that a severe constriction in the channel will cause the water to "back up". The extra specific energy, which was acquired u/s must be lost, even if there is no energy loss in the contraction itself. The required drop of energy E can occur only through the d/s development of super-critical flow (refer to figure 14.4a and article 25.2).

[1] Because effective discharge coefficients are not well defined where suppression exists, the use of a standard fully contracted (or fully suppressed) orifice is desirable wherever conditions permit. For a rectangular cross-section with <u>fully</u> contracted submerged orifice, the discharge coefficient C_d equals 0.61.

[2] At large gate opening, the value of C_c increases, meaning that its influence on Q decreases. Very little vertical contraction occurs when the gate hardly penetrates into the water stream and, usually, the contraction comes only from the bottom weir below the gate.

(unregulated) conditions increases the flow discharge. This increase in flow is due to the decreasing influence of the vertical contraction in the equations.

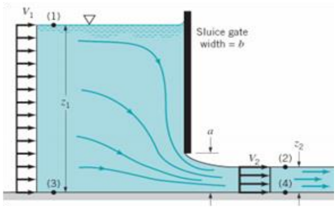


***Figure 16.1**: sluice gate with sharp 90° corner, full contraction at orifice and vena*

At small gate(s) openings, the streamlines of the flow through the gate(s) will initially not be parallel, as shown in the opposite figure. The section where parallel streamlines occur is called the *vena contracta.* The *vena contracta* effect is a result of the inability of the fluid to turn a sharp 90° corner as shown in the same of figure[1].

In the User's Manual, the determination of the contraction coefficient with super-critical outflow d/s of the gate(s) is related to the depth ratio h_0/w (refer to table 16.1).

16.1.4. Discharge coefficient

In the range $1.5 < h_0/w < 5$, orifice flow conditions prevail with free outflow for which the discharge coefficient C_d is applicable in the equation 16.1. The discharge coefficient C_d and the contraction coefficient C_c can be taken from the following table:

***Table 16.1:** values of discharge coefficient C_d and contraction coefficient C_c for the depth ratio h_0/w and free outflow below a gate*

h_0/w	C_c	C_d
1.5 to 2.5	0.633	0.597
2.5 to 3.5	0.625	0.599
3.5 to 5	0.624	0.605

When the ratio h_0/w is less than 1.5, the table does not give any value of C_c or C_d and the equation 16.1 can no longer be applied. At larger gate(s) openings where the head differences between u/s water level and the tailwater level become small (or h_0/w less than 1.5), the determination of the value of C_d may result in considerable errors and has to be derived experimentally. The flow is considered in the User's Manual as unidentifiable.

[1] The depth of flow y_{vena} is in reality smaller than the gate opening and, therefore, partially suppressed due to the vertical (gate) and horizontal (pier) contractions in the gate chamber. The ratio of the area of the jet, at vena contracta, to the area of the orifice is known as coefficient of contraction C_c. In other words, the coefficient shows how the water depth contracts after the gate opening w. When the contraction is partially suppressed such as a bottom suppressed sluice gate (no bottom weir) allowing sediments and trashes to pass the structure, the coefficient C_c is not so well defined (only the submerged fully contracted sharp edged rectangular orifice has a contraction coefficient well defined in laboratory tests and equals 0.61).

When the ratio h_0/w is above 5, the value of the discharge coefficient C_d is considered in the User's Manual to be equal to 0.61; below 1.5, $C_d = 0.60$.

16.1.5. **Velocity coefficient**

For orifice conditions, it is assumed that, on the u/s side of the gate, the depth of flow is much greater than the velocity head and so, the velocity is neglected. Furthermore, in practice, the User's Manual considers that the orifice is designed and maintained so that the approach velocity to the orifice is negligible[1], thus assuring that C_v approaches unity ($C_v \approx 1$)[2].

If significant u/s approach velocity occurs, C_v may be approximated from the following equation for a rectangular cross-section:

$$\boxed{\mathbf{C_v = (H_0 / h_0)^{1.5}}} \quad (16.3)\ (6.1c)$$

Where:

Refer to list of symbols.

16.2. CASE Nº3: GATE(S) LOWERED INTO A RECTANGULAR HORIZONTAL CANAL WITH SUBMERGED FLOW D/S OF THE GATE(S)

16.2.1. **Flow description downstream of the gate(s)**

For usually large opening of the gate(s), the sub-critical flow on both sides of the gate(s) can be established. There is some energy lost d/s the gate(s)[3] and the gate(s) is said to drown.

Independently of the gate(s) position, a d/s obstruction in the channel forces the tailwater to a depth above the conjugate depth y_2, pushing the hydraulic jump u/s and submerged flow can be established. The gate(s) inhibits the movement of the jump further u/s so that super-critical conditions d/s of the gate(s) cannot be attained. Thus, the d/s conditions control (that is the obstruction) influences the water depth u/s of the gate(s) by raising it.

16.2.2. **Modular limit**

To ensure free flow, the following ratio should not be exceeded by h_2/w:

$$\boxed{\mathbf{h_2 / w < C_c / 2\ [1 + 16\ (h_0 / (C_c * w - 1) - 1]^{0.5}}} \quad (16.4)$$

Where:

C_c = theoretical minimum contraction coefficient for modular flow (C_c = 0.611 for sharp edged gate to C_c = 0.99 for rounded edge).

[1] The value of the velocity coefficient varies slightly with the different forms of the gate lip.

[2] For $C_v = 1$, the velocity of approach is zero, as would be the case if the weir were the outlet of a deep reservoir or lake. Furthermore, in irrigation systems, Cv may be assumed to be unity since most of the canals have very flat hydraulic gradients and the flow velocities very low.

[3] A jump will emerge (but invisible) in the submerged flow.

Refer to list of symbols.

This equation is very useful since only depths are involved, which allows for any cross-section.

The maximum gate opening for free outflow is obtained when the d/s undisturbed sub-critical flow depth y_n (considered equal to y_2 on horizontal or nearly so floor) is the conjugate flow depth of y.

The following figure shows the graphical representation of the relations between h_2/w and h_0/w and the types of flow.

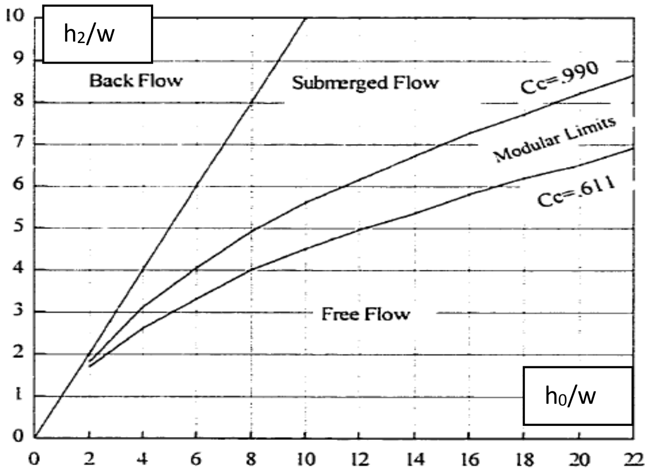


***Figure 16.2**: graphical representation of the modular limit and types of flows*

16.2.3. Submerged flow discharge equation

The upper pool elevation is controlled by both the submergence effect of the tailwater and the gate(s) opening: the u/s water head h_0 is replaced in the equation by the effective head (difference between the initial u/s water head h_0 and the d/s water head h_2).

The applicable equation is the basic head-discharge equation for a submerged fully contracted rectangular orifice flow given below[1].

$$\boxed{\mathbf{Q = C_d * C_v * w * B_{cl} * [(2 * g (h_0 - h_2)]^{0.5}}} \quad (16.5)$$

<u>Where</u>:

Q = discharge of fully suppressed or contracted orifice (m^3/s).

Refer to list of symbols.

[1] In submerged conditions, the submerged flow may also be estimated by using the modular flow with a flow reduction factor f (refer to article 15.1.2 and equation 15.3).

The discharge rate decreases with higher tailwater depth; either the u/s water depth must increase or the gate(s) opening must be adjusted to keep the same discharge.

16.2.4. Contraction coefficient

The gate is considered as a submerged orifice with regulated flow and a perimeter contraction partially suppressed with the presence of a gate sill.

The passage from free to submerged flow usually corresponds to a small gate opening towards a large gate opening. For greater gate(s) opening, the amount of orifice opening perimeter produces less or no curvature of the outflowing jet passing through the opening (vertical contraction).

The contraction coefficient may be similar in submerged flow and free flow at small openings but not at large openings. Therefore, its determination may result in considerable errors at large gate(s) openings where the head differences between u/s water level and the tailwater level become small.

16.2.5. Discharge coefficient

The discharge coefficient C_d depends upon h_2/w (refer to article 16.1.4). Where the head differences between u/s water level and the tailwater level become small (or $h_0/w < 1.5$ and close to 1), the flow is considered as unidentifiable, because the determination of the value of C_d may result in considerable errors and should be derived experimentally.

16.2.6. Velocity coefficient

Practically, its value is also taken as 1 (refer to article 16.1.5).
the orifice opening (refer to article 16.1.5).

16.2.7. Froude number

The Froude numbers depends upon the ratio h_0/w and the value of C_c. The Froude number of the jet under submerged and under free flow conditions equals is calculated with equation 16.5.

Submerged conditions occur for both Froude numbers $F_r < 1$ (fully submerged flow d/s of the gate is sub-critical), as well as for Froude numbers $F_r > 1$ (partially submerged transitional flow and super-critical d/s of the gate).

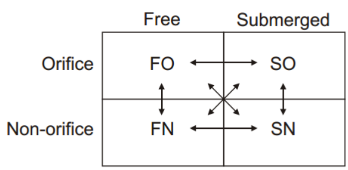


***Figure 16.3**: flow types and conditions at gated head regulator*

16.3. CASE Nº4: GATE(S) LOWERED INTO A RECTANGULAR HORIZONTAL CANAL TO A HEIGHT CREATING A TRANSITIONAL TYPE OF FLOW

The developed equations apply for defined flow conditions but are of typically low accuracy in the transitional zone where the flow can move from any behaviour to any other, from orifice (regulated) to non orifice (unregulated) conditions and from free to submerged type, as pictured in the opposite figure (refer to table 14.5).

The instantaneous unit discharge of transient flow passing through the gate(s) is difficult to calculate.

16.4. GATE SILL

The use of an under-gate sill affects the flow behavior below and d/s of the gate. The crest level or gate sill of the branching canal is generally kept 0.1 m to 0.5 m higher that the crest level in the river or parent canal.

In non orifice flow conditions, the flow of a gate without a sill behaves as broad crested weir.

The energy head H and the opening w are both related to the height of the sill. It is usually admitted that the value of C_d increases with the increase of the Froude's number and the sill height[1]. A high sill level means that the energy head H and the opening height w are smaller, so the width of the orifice B_{cl} should be increased at higher costs. The minimum costs of the structure are often obtained with a sill at canal bed level.

Some uncontrolled variations of contraction under the gate are shown in the following figures:

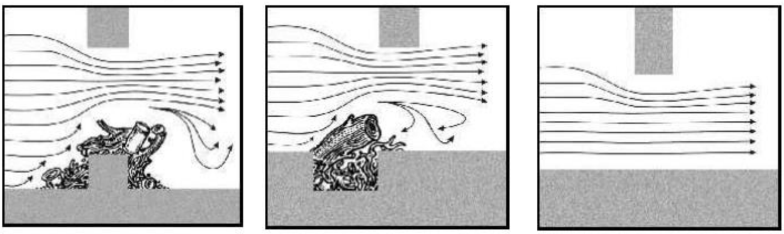


***Figure 16.4**: factors affecting the flow under a gate*

16.5. FORCES AND MOMENT ON GATE(S)

16.5.1. Forces on closed gate(s)

Gates must be made strong enough to withstand the force created by the hydrostatic pressure (horizontal thrust). The force acting on the whole immersed vertical plane of the closed gate is the resultant hydrostatic force

[1] The increases of C_d are due to increase in the velocity through the gate opening and the gradually decreasing value of the velocity after the gate due the d/s slope of the sill. The rate of increase depends also upon the configuration of both the sill and the gate.

(from a distributed load to a point load) and is calculated using the following formula[1] (neglecting friction):

$$\boxed{F_{R\ gate} = A * \gamma_{water} * \bar{y}} \quad (16.6)$$

Where:

$F_{R\ gate}$ = resultant hydrostatic force acting on the gate (N).

A = area of the gate (m^2).

γ_{water} = water specific weight (9,810 N/m^3).

$\bar{y}$ = water depth from water surface to centroid/centre of the closed gate (m).

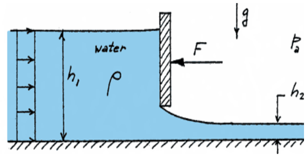


***Figure 16.5**: terminology of horizontal pressure forces on open gate*

16.5.2. Forces on closed gate(s)

The point of application of the resultant force $F_{r\ gate}$ acting perpendicular to the immersed gate is the centre of pressure located at some depth below the free surface. The depth to the application point of the resultant force is given by the following formula (rectangular gate):

$$\boxed{D_{RF\ gate} = \bar{y} + H_{gate}^2 / (12 * \bar{y})} \quad (16.7)$$

Where:

$D_{RF\ gate}$ = depth to the application point of the resultant hydrostatic force acting on the gate (m).

H_{gate} = gate height (m).

$\bar{y}$ = water depth from water surface to centroid/centre of the closed gate (m).

This water depth $D_{RF\ gate}$ (depth to force D in figure) is always greater than the water depth $\bar{y}$ from the water surface to the centroid / centre of the gate. In other words, the centre of pressure is always below the centroid.

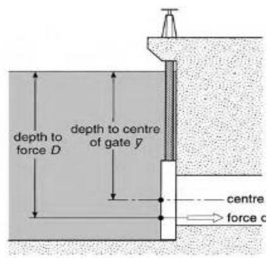


***Figure 16.6**: $F_{R\ gate}$ and depth of application*

[1] Pascal's principle states that the pressure exerted by a fluid at a depth is transmitted equally in all directions. The total pressure force on a plane area is equal to the area multiplied by the intensity of pressure at its centroid. The computation of hydrodynamic forces acting on partially opened gates is far more complicated as it is closely related to flow conditions.

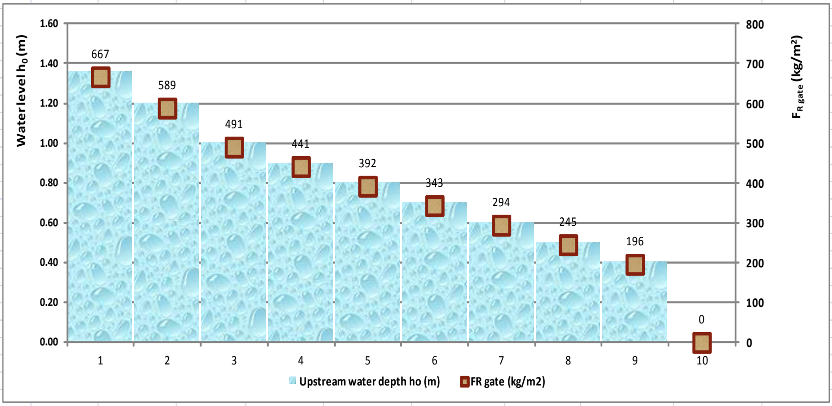


***Figure 16.7**: water depth h_o and $F_{R\ gate}$ acting on a vertical gate*

16.6. FLOW RECAPITULATION

The following table summarizes the different flows, including flow types, flow conditions and flow regulation.

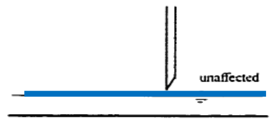
***Table 16.2:** flow types, conditions and regulations with corresponding equations*

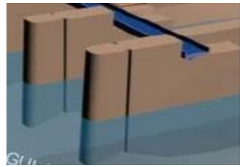
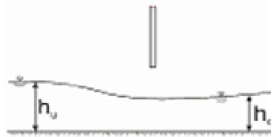
Flow types	Flow conditions	Regulation	Equations
Free	Orifice	Regulated	16.1 or 16.2
	Non orifice	Unregulated	15.1
Transitional	Between orifice & non orifice	Semi-regulated	15.1 or 15.2 16.1 or 16.5
Submerged	Orifice	Regulated	16.5
	Non orifice	Unregulated	15.2 or 15.3

For conditions outside the range covered above, a comprehensive treatment of the effects of gate(s) location and geometry on discharge for free controlled outflow has to be carried out.

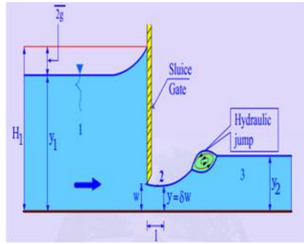
The visual difference between free and submerged flow is that the jet does not have a free surface open to the atmosphere.

Regulation & type of flow	Description	Equation	Side view
Weir or non orifice conditions			

Unregulated with free flow under the gate(s)	Fully open gate(s) (gate bottom above water surface) with horizontal constriction or obstruction due to gate(s) chamber
	Flow non affected by d/s water conditions
		$Q = C_e*B_{cl}*h_0^{3/2}$(eq. 15.1)
		• $C_e = C_d*C_v*2/3*(2/3*g)^{1/2}$ (eq. 6.1c)
		    • $C_d = 1$ (with C_e =1.70) or > 1 (with C_e =1.84) (flow state)
		    • $C_v > 1$
		• $h_2/h_0 < 75\%$
			

Unregulated with submerged flow below the gate(s)	Fully open gate(s) (gate bottom above water surface) with horizontal constriction or obstruction due to gate(s) chamber
	Flow affected (reduced) by d/s tailwater conditions
		$Q = C_e*B_{cl}[2*g(h_0 - h_2)]^{0.5}$(eq. 15.2)
		• $C_e = C_d*C_v*2/3*(2/3*g)^{1/2}$ (eq. 6.1c)
		    • $C_d = 1$ (with C_e 1.70) or > 1 (with C_e 1.84) (flow state)
		    • $C_v > 1$
		• $h_2/h_0 > 75\%$
		$Q = f*Q_{free\ flow}$ (eq. 15.3)
		• f from table 15.1
			 

Regulation & type of flow	**Description**	**Equation**	**Side view**
		Orifice conditions	

Regulated with free flow d/s of sluice gate(s)	Gate(s) partially open with super-critical outflow open to the atmosphere	$\mathbf{Q = C_e*w*B_{cl}[(2*g\,(h_0 - C_c*w)]^{0.5}}$ (eq. 16.1)
		• $C_e = C_d*C_v$
		  • $0.597 < C_d < 0.605$ (table 16.1)
		  • $C_v \approx 1$ or $(H_0/h_0)^{0.5}$ (eq. 16.3)
		• $h_2/w <$ modular limit (eq. 16.4)
		• $F_{r1} > 1$ (eq. 16.5)
			
			(with or without sill)

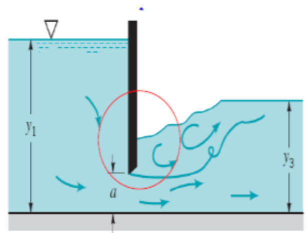
Regulated with submerged flow d/s of gate(s)	Gate(s) partially open with flow partly submerged super-critical or fully submerged sub-critical flow	$\mathbf{Q = C_e*w*B_{cl}[(2*g\,(h_0 - h_2)]^{0.5}}$ (eq. 16.5)
		• $C_e = C_d*C_v$
		  • $C_d = 0.61$ for fully contracted submerged orifice
		  • $C_d = 0.61\,[(1 + 0.15\,(P_s/P_0)]$ (eq. 16.8) or experimental
		  • $C_v \approx 1$
		• $h_2/w >$ modular limit (eq. 16.4)
			
			(with or without sill)

***Figure 16.8**: governing flow types and regulations and corresponding equations for sluice gate with orifice and non orifice conditions*

17. RATING CURVE

17.1. DISCHARGE MEASUREMENT AND CONTROL

17.1.1. Discharge measurement

Weirs, flumes and orifices can all be used for discharge measurement. But weirs and flumes are better suited to measuring discharges in rivers when there can be large variations in flow. Weirs and flumes not only require a simple head reading to measure discharge but they can also pass large flows without causing the upstream level to rise significantly and cause flooding.

Orifice structures too can be used for flow measurement but large variations in flow also mean that the gates will need constant attention for opening and closing (refer to article 17.7).

17.1.2. Discharge control

Orifices are rather cumbersome for discharge measurement but very useful for discharge control because the discharge through an orifice is not very sensitive to changes in u/s water level. If the orifice opening is assumed to be fixed, so the discharge Q changes only when the upstream depth d changes.

In contrast, if a weir (or flume) is installed at the head of a canal, it would be very easy to use for discharge measurement but it would not be so good for controlling the flow because it is very sensitive to water level changes (refer to article 17.7).

17.2. DISCHARGE RATE AND STRUCTURE DESIGN

The following table shows the type of structure to be designed according to its location and the discharge rate to be considered.

Table 17.1: *structure design according to location and discharge rate considered*

Location	Discharge rate	Type of structure
River or parent canal	Q_{max}	Cross-regulator width & stilling basin Cross-regulator cut-off walls Cross-regulator protective works
	Q_{min}	Number & gate(s) sizing of head regulator to reach Q_{crop} Head regulator weir width to reach Q_{crop}
Branching canal	Q_{max}	Head regulator stilling basin Head regulator cut-off walls Head regulator protective works
	Q_{crop}	Number & gate(s) sizing of head regulator corresponding to Q_{min} in river or parent canal

The required irrigation water Q_{crop} through the head regulator (also called discharge regulator) has to take into account the flow depth in the parent canal and the effective waterway B_{cl} corresponding to a certain opening (bay) fitted with sliding gates.

17.3. GATE SIZING AND NUMBER OF GATES

The hydraulic gate(s) width (B_{cl}) is large enough to pass the irrigation discharge Q_{crop} under low flow conditions in the river or parent canal (Q_{min}), but small enough to restrict the under flow to the maximum capacity of the canal, assuming a functioning spillway.

To check the flood water entering the canal, a breast wall is provided between designed upper pool (pond) level and high flood level. Unless the difference between the high flood elevation and the upper pool elevation is nominal, the construction of a breast wall is recommended (usually more economical than higher gates).

17.4. FLOW CONDITIONS

17.4.1. Flow conditions

The flow conditions and types have to be checked (orifice or non orifice) and the relevant equation selected for calculating the discharge rate through the head regulator (refer to figure 16.8).

The User's Manual recommends that, in order to avoid any significant backwater effect in the u/s channel, the Q_{max} or high flood flow should be totally unobstructed. Therefore, the equation to be selected for calculating Q_{max} through the head regulator is the non orifice equation with the gate(s) opening at least equal or greater than $h_{0\ max}$. With fully open gates, the Froude number should not exceed 0.5 at the flow measuring gauge (refer to article 6.4.1).

17.4.2. Gate opening and position of the hydraulic jump

The regulated flow is computed assuming that all gates are set to the same position (all gates are considered as one).

Free outflow remains as long as the ratio h_2/w with the value of the limiting contraction coefficient for modular flow C_c = 0.611 does not exceeds the modular limit (refer to article 16.2.2). With free flow, the adjustment of the gate opening is based upon information from upstream (usually water head h_0). In this case, the gate opening is given by the following equation:

$$\boxed{w = y_1 / C_c} \quad (17.1)$$

Where:

Refer to list of symbols.

The maximum gate opening w_{max} with d/s free flow is obtained when the sequent depth of y_1 is exactly matching the normal depth y_n in the d/s channel obtained by the Manning's formula (refer to table 10.1, figure 14.4a)[1].

For gate openings $w < w_{max}$, super-critical flow occurs d/s of the gate(s) with $y_2 > y_n$. The jump runs away from the gate(s) (refer to table 10.1, figure 16.8 and article 10.5.1), forcing the water depth before the jump y_1 to gradually move downwards and increase in depth[2] up to a point where the depth y_1, the sequent depth y_2 and the approaching Froude number satisfy the equation 10.4 and, therefore, triggering the jump[3].

For gate openings $w > w_{max}$, super-critical flow initially occur d/s of the gate(s) with $y_2 < y_n$. The jump runs towards the gate, eventually flooding the reach between the gate(s) and y_1 (refer to table 10.1, figure 16.8 and article 10.5.1).

17.5. RATING CURVE

17.5.1. **Introduction**

The approach of the User's Manual is to combine water distribution to water level control and water measurement is utmost important for the implementation of the Integrated Water Resources Management (IWRM) at basin level as shown in the below schematic.

17.5.2. **Rating curve for natural channel**

The rating curve presents the relationships between the discharge (m^3/s) and the river stage above the arbitrary datum. In the User's Manual, rating curves are based on the slope-area method.

The procedure for determining a rating curve in a particular section for a natural channel is the slope-area method based on the empirical Manning's equation:

1. Obtain a cross-sectional profile of the channel at the point of interest.
2. Set datum at lowest point in channel.
3. Set maximum in the channel.
4. Find difference between max. and min. and subdivide into equal increments of depth.
5. For each increment, solve for parameters of Manning's equation.
6. Plot the resulting flows versus stage to find the rating curve.

[1] With a gradually varied flow (GVF) and in certain conditions, an analog to $(F_r)^2$ is developed for applying equation 10.4 and $(F_r)^2$ may be substitute by $(y_c/y)^3$, y being the water depth at the point of interest (refer to article 10.3.4).
[2] If depth of y_1 is increasing while moving downwards, depth of y_2 is decreasing.
[3] In other word, the gate does not release enough water to permit the formation of a jump close to the gate.

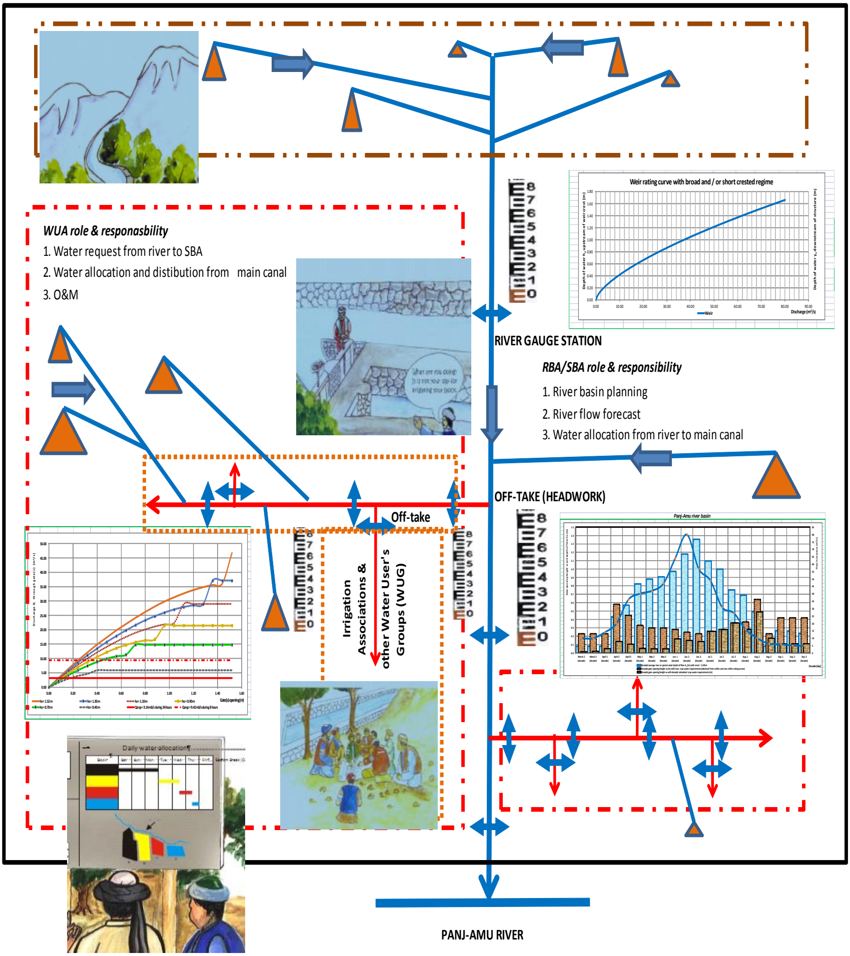
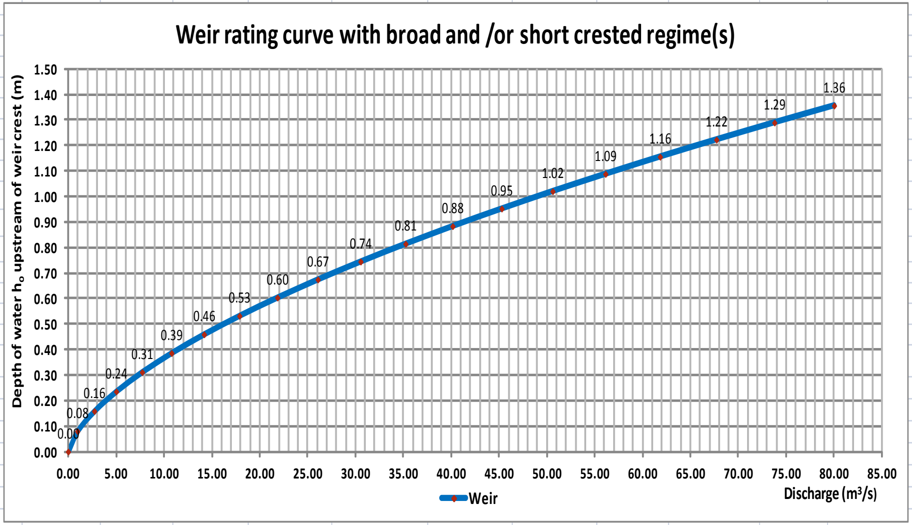
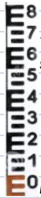


***Figure 17.1**: Integrated Water Resources Management (IWRM) schematic*

17.5.3. Rating curve of non orifice flow

The approach of the User's Manual is to combine water level control and flow measurement functions and to establish the best estimate rating curves of the cross-regulator and head regulator based on the theoretical equation.





The min. u/s distance between the fixed staff gauge and the u/s weir toe = *3.05* *m*

Ref. level of 0 corresponds to *100.40* *m*

u/s water level h_0 on staff gauge (m)	Type of flow		Q (m^3/s)
	From	To	
1.36	Free	Free	80.04

Ref. level = *100.40* *m*

u/s water level h_0 on staff gauge (m)	Regime of flow		Q (m^3/s)
	From	To	
1.36	OK	OK	80.04

Ref. level = *100.40* *m*

OK corresponds to broad crested weir regime
(see User's Manual. article 6.4.2)
WARNING corresponds to short crested weir regime
(see User's Manual. article 6.4.2)

***Figure 17.2**: rating curve for non orifice flow*

The weir (non orifice) flow has standard rating curves based on the general weir equation but with particular coefficients for each type such as the broad crested weir operating under two conditions: modular and non modular flow. The u/s water head h_0 is measured at a point where the h_0 across the width of the channel is uniform (refer to article 6.4.1).

For a weir that always operates in free flow, the calculations can offer free-flow rating curves that condense the entire range of operations into a single table. On the other hand, for a weir that experiences submerged flow, multiple tables are required to provide information covering the range of u/s and d/s water levels.

A weir rating curve is shown in the above graph taken from the attached spreadsheet.

17.5.4. **Rating curve of orifice flow**

In the case of the gated head regulator (orifice flow), for a given u/s water head h_0, the opening of the gate(s) needed to produce a given discharge rate is calculated and presented in the following graph form

For a gate that always operates in free flow, the calculations offer continuous free-flow rating curves that condense the entire range of operations (varying u/s water level and varying gate setting or discharge) into a single table. On the other hand, for a gate that experiences submerged flow, multiple tables are required to provide information covering the range of u/s and d/s water levels, as well as a varying gate settings or discharges.

The increase of discharge rate from orifice flow to weir flow is due to the decreasing influence of the vertical contraction coefficient C_c and/or the change of flow state (refer to 6.4.2).

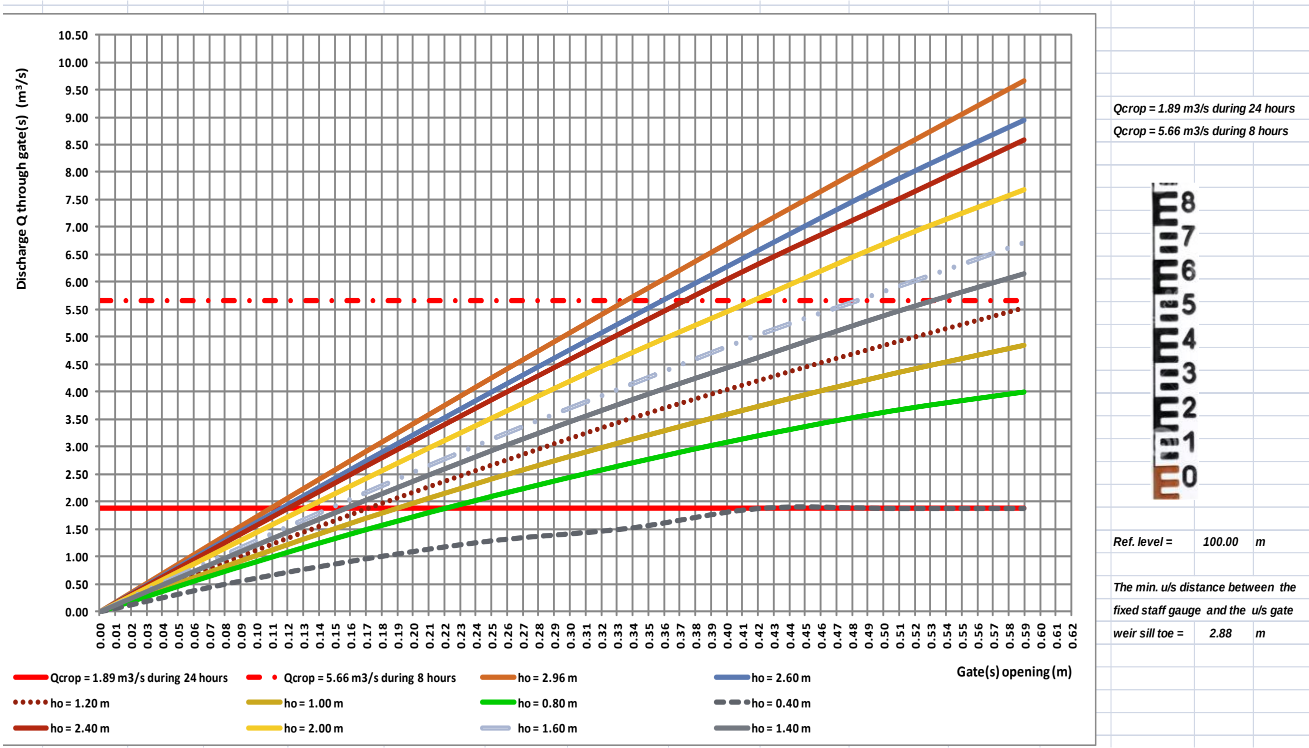


***Figure 17.3a**: gate(s) rating curves for different values of upstream water depths h_o in the supply canal*

17.5.5. **Hydrograph and gate(s) openings**

A hydrograph combines the decadal river flow (expressed in m^3/s and water depth h_o) with the corresponding decadal gate(s) opening height (w) of the canal head regulator to reach the required irrigation flow in the main canal as shown in the figure 17.3b.

The left ordinate represents the gate(s) opening height (w in m) and the depth of flow (ho in m) in the river. The right ordinate represents the river flow (m3/s) and the abscissa indicates the time in decades. The solid blue curved line stands for the discharge in the river (m^3/s) with respect to decades. The blue vertical bars show the corresponding decadal river water depth (ho in m). The brown columns state the decadal gate(s) opening height (w in m) calculated with the maximum irrigation water need (water duty) while the grey bars represent the gate opening height (w in m) calculated with the monthly irrigation water need. Finally, the red solid line represents the minimum depth of flow h_o (m) in the river or parent canal corresponding to the required discharge based on the maximum irrigation water need (July).

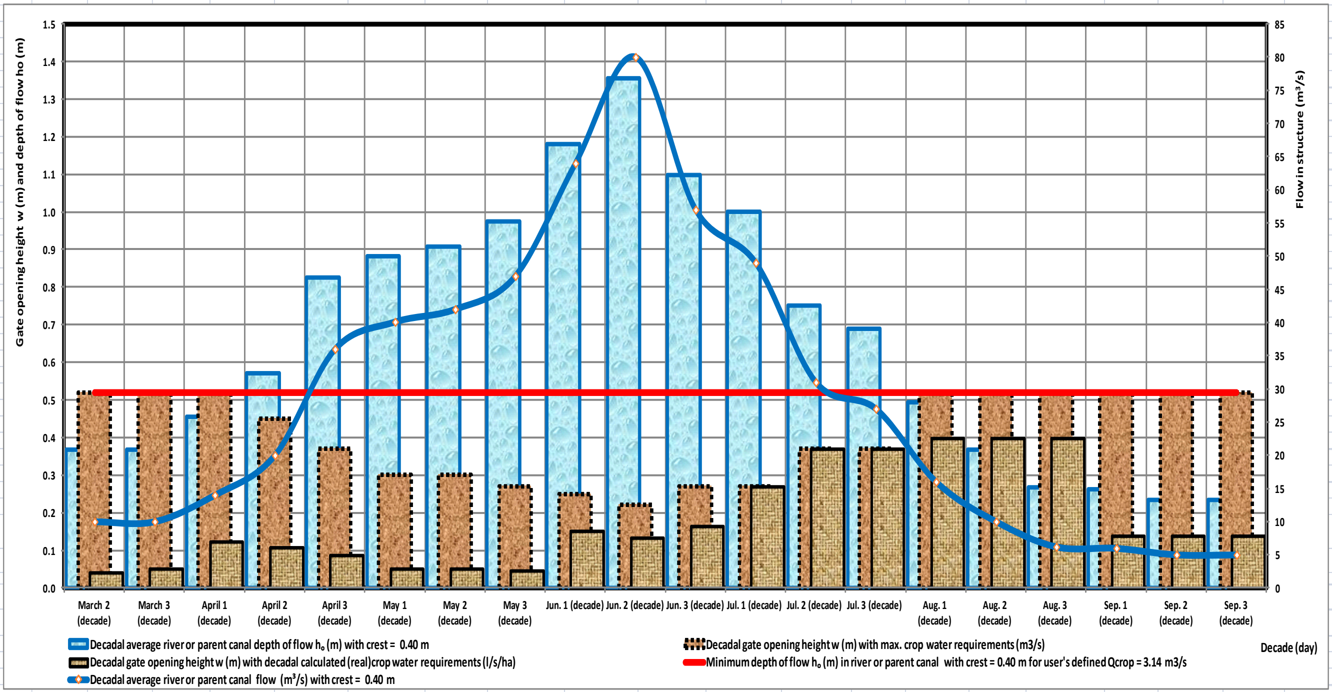


***Figure 17.3b**: river hydrograph and corresponding gate(s) openings of the canal head regulator calculated with the maximum and decadal irrigation water need*

17.6. ACCURACY OF THE RATING CURVE

17.6.1. Introduction

Stage-discharge relation is non linear and it is difficult to obtain high accurate rating curves. One of the greatest problems with standard structures is that they do not always conform to the chosen coefficients[1].

Significant errors δ are possible because of the unique approach conditions at proposed projects and of tailwater inaccuracies. An error can be random (reading errors), systematic or spurious (gate malfunction or human mistakes).

17.6.2. Source of errors

For discharge measurement structures, the main sources of error to be considered are:

1. $\delta_{coefficient}$: error in C_e[2] (C_d*C_v) (refer to article 6.1.2).
2. δ_f: error in the drowned flow reduction factor f (refer to table 15.1).
3. δ_{mn}: error in dimensional measurement of the weir, such as the width or the height of the weir.
4. δ_{water} depth: error in gauge position and or the measurement of u/s water depth in relation to weir sill elevation and/or Δh (refer to figure 6.1 and article 6.4).

The error $\delta_{coefficient}$ is considered systematic[3]. When the flow is modular, the drowned flow reduction factor f is constant (f = 1) and is not subject to error (δ_f = 0). With submerged flow, the error consists of a systematic error[4]. The error δ_{mn} depends upon the accuracy with which the constructed structure can be measured; it is also a systematic error. Finally, the error δ_{water} depth is a systematic and random error. Possible contributions to errors are an improper maintenance of the gate(s), construction faults not included in δ_{mn} etc.

Therefore, whenever a flow rate or discharge is measured in a structure, the value obtained should be considered as the best estimate of the true flow rate that can be slightly greater or less than the true discharge.

17.6.3. Submerged flow type

The submerged flow type is frequently found on many installations where the tailwater covers the jump, but is seldom recognized and corrections are rarely made. Submerged flow in the structures will always decrease accuracy of flow measurement. Failure to make corrections is surprising since a considerably greater discharge may be indicated than is actually flowing. Submerged flow type weirs can occur unintentionally by poor design,

[1] For example, C_d has a standard value for standard structures.

[2] C_e can vary with the presence of silt and debris accumulated in the region of dead water.

[3] This classification is not entirely correct because C_d and C_v are function of h_1.However, the variations of the errors in C_d and C_v as a function of h_1 usually are sufficiently small to be neglected.

[4] Δf is a systematic and constant error being in the numerical value of f (refer to table 15.1)and a systematic and random error caused by the fact that f is a function of the submergence ratio H_2 / H_0 or h_2 / h_o.

construction errors, structural settling, attempts to supply increased delivery needs by increasing d/s heads, accumulated sediment deposits, or weed growths.

The value of the discharge coefficient C_d for submerged flow is not very accurate and may result in considerable errors if the head differences between u/s water level and d/s tailwater level become small (refer to article 16.2.5). Hence, the calculation of the discharge of a vertical gate under submerged conditions will be of lower accuracy.

17.7. CONCLUSIONS

Gate rating curves, as computed by the above equations, should be considered as a preliminary rating curve. In finalizing the rating curves for major head regulators, special measurements on similar existing projects should be considered (current-meter method is more commonly used).

Although gates are rather cumbersome for discharge measurement, they are very useful for water level control. This is because the discharge through a free flow orifice is not very sensitive to changes in u/s water head h_0, as shown in the following graphs (refer to equations 16.1 and 15.1):

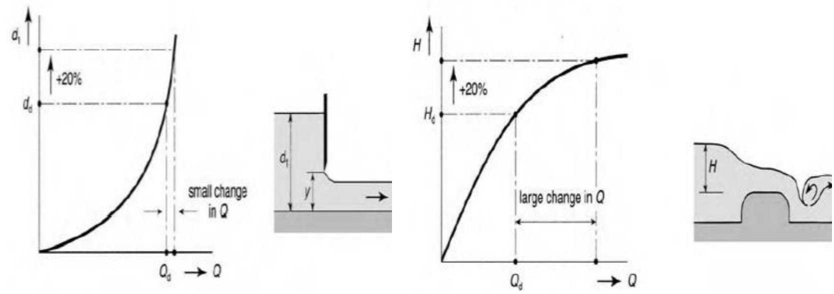


***Figure 17.4**: water level control using a gate (orifice flow) and weir*

Suppose the parent canal water head h_0 or energy head H_0 (d_1 and H_0 in the graphs) rises by say 20%. The effect on the discharge into the branch through the gate is to change by only 5%. So even though there is a significant change of h_0 in the parent canal, this is hardly noticed in the branch canal. This can be very useful for ensuring a reliable, constant flow to a farmer even though the parent canal water head h_0 may be varying considerably.

PART VI:

SCOUR DEPTH IN CHANNEL BED

18.WATERWAY AND REGIME SCOUR DEPTH

18.1. WATERWAY DETERMINATION

18.1.1. Wetted perimeter

For shallow and meandering rivers, the minimum stable width can be calculated from Lacey's modified formula:

$$\boxed{P_w = a * (Q_{max})^{0.5}} \quad (18.1)$$

Where:

P_w = *Lacey's wetted perimeter or minimum waterway width*[1] *(or stable width of waterway) (m).*

Q_{max} = *maximum flow discharge in the river (HFL) (m³/s).*

a = coefficient depending on stability of river channel.

B = overall waterway width between river banks or abutments (m).

For large river, the wetted perimeter P_w is practically equal to the river width. If the width of the river is considerably larger in comparison to the water depth, the computed perimeter is provided as width only.

With these tentative values, the adequacy of the waterway and the crest level for passing the design flood within the permissible afflux (refer to figure 5.2) need be checked and readjusted in such a way that the permissible values of afflux are not exceeded.

18.1.2. Looseness factor

The proposed above Lacey's equation applies when the looseness factor is ≤ 1. The ratio of actual width to the regime width of the river is the looseness factor, as shown in the following equation:

$$\boxed{\text{Looseness factor} = B / P_w} \quad (18.2)$$

Where:

P_w = *Lacey's wetted perimeter or minimum waterway width*[2] *(or stable width of waterway) (m).*

Q_{max} = *maximum discharge in the river (HFL) (m³/s).*

a = coefficient depending on stability of river channel.

B = overall waterway width between river banks or abutments (m).

[1] For large river, the wetted perimeter P_w is practically equal to the river width. If the width of the river is considerably larger in comparison to the depth of water, the computed perimeter is provided as width only.

[2] For large river, the wetted perimeter P_w is practically equal to the river width. If the width of the river is considerably larger in comparison to the depth of water, the computed perimeter is provided as width only.

A looseness factor ≤ 1 is obtained when Lacey's waterway is restricted due to fluming or contraction of normal/regime waterway i.e. when the actual waterway provided for the structure is less than Lacey's waterway.

Table 18.1: *coefficient a in Lacey's minimum stable width of waterway*

Channel type	a	Remark
Stable channel in scour resistant material	3.3	In most literature 4.75 is adopted
Shifting channel in sandy material	4.9	

Generally, with a high looseness factor, there is a tendency for shoal formation d/s of the structure, which may cause maintenance problems.

18.2. SCOUR DEPTH

Depending on the bed material, the river bed, during peak flood flows, may become mobile to several meters below the normally observed river bed. Unless founded on rock, the structure needs to be designed to withstand such deep scour depths (refer to opposite figure).

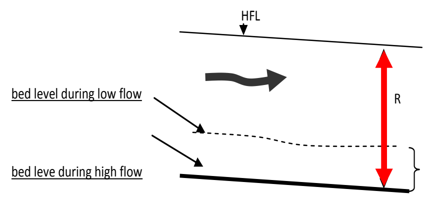


Figure 18.1: *mean scour depth R measured from the high flood level*

The Lacey's method of estimating the river scour depth R is used in this Manual. This method is basically empirical and essentially gives total scour below high flood level (HFL) in the case of meandering rivers in flood plain and is meant for non cohesive sandy material with mean sediment size of about 0.15 mm to 0.43 mm. The method is not valid outside this range[1].

18.3. SOIL CONDUCTIVITY

The grain size distribution of a soil is one of the geotechnical aspects of soil mechanic properties that affect the hydrogeological conductivity. A sorted soil with larger grains has a high hydraulic conductivity. If a sediment contains a mixture of grain sizes (multi-graded soil), the porosity will be lowered, and thus the hydraulic conductivity. This is because the void between the larger grains is filled up with smaller grains.

[1] Lacey's theory is applicable only to stable alluvial rivers and no to rocky or boulder river stages and unstable (aggrading or degrading) alluvial rivers. In the case of coarser material with larger standard deviation, as scour progresses, scouring occurs by selective removal of finer material from scour hole and hence smaller scour depth will occur. For very fine material, having cohesion, it is generally considered that there will be greater resistance to scour and hence reduced scour depth will result. Further, due to effects being site specific, larger variations in scour depth are likely to occur which cannot be related to Q and f alone. However, the method is commonly used in design of structures.

Hydraulic conductivity (or transmissivity) is correlated to the particle size and can be estimated by using methods based on grain size analysis. This grain-size analysis estimates representative value of the grain size (d_{50}[1], d_{40} etc.).

The following figure gives a simple classification of the major soil groups according to the particle size (mm).

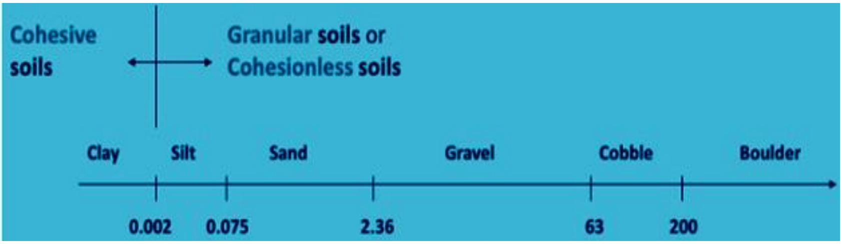


Figure 18.2: *classification of major soil groups according to particle size (mm)*

The following table gives the average particle size d_{50} for different types of material.

Table 18.2: *indicative values of average sediment grade scale (size particle d_{50} in mm)*

Type of material (soil)		Average grain size d_{50} (mm)
Clay	Very fine	0.0005 to 0.00024
	Fine	0.001 to 0.0005
	Medium	0.002 to 0.001
	Coarse	0.004 to 0.002
Silt	Very fine	0.008 to 0.04
	Fine	0.0016 to 0.008
	Medium	0,031 to 0.016
	Coarse	0.062 to 0.031
Sand	Very fine	0.125 to 0.062
	Fine	0.250 to 0.125
	Medium	0.500 to 0.250
	Coarse	1.000 to 0.500
	Very coarse	2.000 to 1.000
Gravel	Very fine	4 to 2
	Fine	8 to 4
	Medium	16 to 8
	Coarse	32 to 16
	Very coarse	64 to 32

[1] d_{50}: average particle size of the soil which 50% of the material is finer.

Boulder/cobble	Small cobble	128 to 64
	Large cobble	256 to 128
	Small boulder	512 to 256
	Medium boulder	1,024 to 512
	Large boulder	2,048 to 1,024
	Very large boulder	4,096 to 2,048

18.4. SILT FACTOR

With known average particle size d_{50} (mm) of the bed material (where the structure will be imbedded), the silt factor f may be calculated from the following relationship:

$$\boxed{f = 1.76\,(d_{50})^{1/2}} \quad (18.3)$$

Where:

d_{50} = average particle size or diameter of the channel bed material (mm)[1] (refer to table below).

The following table gives some indicative values of the Lacey's silt factor:

Table 18.3: *indicative values of the Lacey's silt factor f*

Soil type	Lacey's silt factor f
Boulders and shingle	20.0 to 15.0
Boulders and gravel	12.5
Medium boulders, shingle and sand	10.0
Gravel	4.75
Coarse sand	1.5
Medium sand	1.25
Standard silt	1.0
Medium silt	0.85
Fine silt	0.6
Clay	0.05

18.5. CALCULATION OF REGIME SCOUR DEPTH IN CHANNEL

Scour depth depends largely on the following factors:

- Particle size of the channel bed material.
- Intensity of flow per unit width.
- Velocity of flow.

During major floods, the main incised channel tends to increase in size towards the geometry corresponding to the peak flow rate. It is unlikely that a full adjustment to this geometry will be achieved during an individual flood because the appropriate changes in channel width and longitudinal hydraulic gradient take a considerable time. Nevertheless, because of the uncertainty of

[1] The sieve size through which 50% of the material passes by weight in mm)

how far any short-term changes will progress, it is assumed in the User's Manual that the geometry corresponding to the design flood would be reached.

18.5.1. Mean scour depth

When the river width does not equal the wetted perimeter[1] of 4.75 √Q, the mean scour depth for natural channels flowing in non cohesive soils[2] (where the structure will be built) measured from the high flood level (Q_{max}) is calculated from the Lacey's mean scour depth equation[3] as follows:

$$\boxed{\mathbf{R = 1.35\ (q^2\ /\ f)^{1/3}}} \quad (18.4)$$

Where:
R = mean scour depth measured from the high flood level (m).
q = discharge per unit width of stream allowing for concentration of flow where the width is the actual river width taken as the flood water width at the given site ($m^3/s.m$).
f = Lacey's silt factor related to grain size d_{50}.

The above equation applies when the looseness factor < 1.

18.5.2. Normal scour depth

When the river width > or = the regime width of 4.75 √Q, the normal scour depth for natural channels flowing in non cohesive soils (where the structure will be built) with Q_{max} (HFL) is calculated from the Lacey's normal scour depth equation[4] as follows:

$$\boxed{\mathbf{R = 0.473\ (Q_{max}\ /\ f)^{1/3}}} \quad (18.5)$$

Where:
Q_{max} = design flood discharge at the given site (m^3/s).
Refer to list of symbols.

The above equation applies when the looseness factor > 1.

18.5.3. Maximum discharge intensity per unit width of weir

The unit discharge intensity q during high flows may be kept low to avoid costly energy dissipater. The following table provides values of discharge intensities as a criterion to design the minimum required width of weirs.

Table 18.4: *foundation material and corresponding maximum unit discharge per unit width of stream*

[1] It means, the river width is still active.

[2] Alluvium.

[3] The equation is based on alluvial regime and may not be quite correct for large river or and for boulder or clayey reaches.

[4] The equation is based on alluvial regime and may not be quite correct for large river or and for boulder or clayey reaches.

Foundation material	**Unit discharge q per m width (m^3/s.m)**
Fine sand	5.0
Coarse sand	10.0
Sand & gravel	15.0
Sandy clay	20.0
Clay	25.0
Rock	50.0

In addition, the discharge intensity is related to the depth of flow and the velocity of flow which should be kept as much as possible below the value of the erosion threshold.

19. CUT-OFF WALL

19.1. CUT-OFF WALL ROLE

The purpose of providing cut-off walls is two-folded: (i) increases the flow path and (ii) reduces the uplift pressure, ensuring stability to the structure as shown in the following figure:

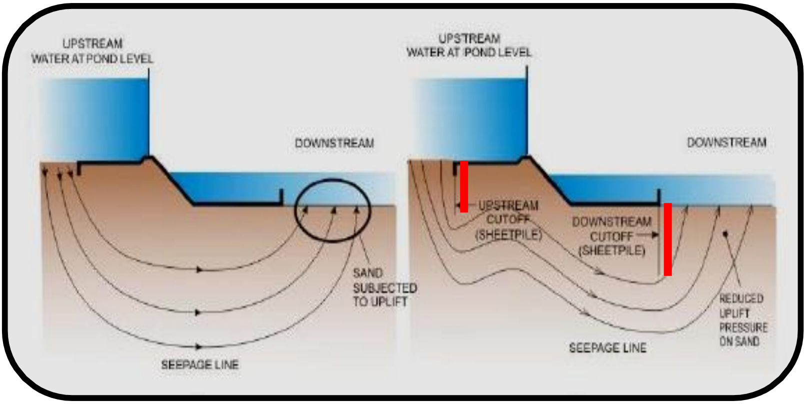


***Figure 19.1**: effect of cut-off walls on seepage lines*

The u/s cut-off wall is more efficient in reducing the uplift pressure while the d/s cut-off pile is more effective in reducing piping.

While designing a cross-regulator, d/s cut-off from the maximum scoured depth considerations is, first of all, provided and then checked for G_e. If a safe value of G_e is not obtained, then the depth of cut-off or the length of the impervious floor is increased (refer to article 22.2).

19.2. CUT-OFF WALL DEPTH

The depth of the u/s pile line will be governed by the scour depth R alone, while, on the d/s end, both the scour depth R and exit gradient G_e have to be considered.

The intermediate sheet pile lines are not required from consideration of scour or exit gradient but they act as important secondary lines of defense. They are also helpful in the matter of distribution of pressure due to uplift pressure.

The scour depth D measured from the channel bed level is given by the following equation:

$$\boxed{D = FoS * R - y_n} \quad (19.1)$$

<u>Where</u>:
D = depth of scour measured from channel bed level (m).
y_n = depth of flow from rating curve (m).
Refer to list of symbols.

To the calculated natural scour depth R of the river from the water surface level, a safety factor (FoS) is applied as given by the following table:

Table 19.1: *values of safety factor (FoS) for scour depth*

Location)	FoS
U/s cut-off depth	1.25
D/s cut-off depth	1.50 or 1.75
Pier	2.00
Flow concentration	2.00 to 3.00

In the d/s expansion of section, the velocity flow distribution is uneven in the cross-section, which could lead to asymmetry of flow and the development of erosion in places of highly concentrated velocity. Therefore, a higher value of the factor of safety (FoS) is always used in the design of the d/s protected transition.

The cut-off wall depth d_1 (u/s cut-off) or d_2 (d/s cut-off) is equal to the scour depth D measured from the channel bed level multiplied by a safety factor.

It has to be noted that the depth of the d/s cut-off wall will be governed not only by the local scour depth but also by the exit gradient value (refer to article 22.5.3). This cut-off depth is also limited by practical considerations, as very deep walls can be difficult or impracticable to build at the site.

The cut-off walls must extend from one bank of the cross-regulator to the other bank, and underneath the wing walls and return walls.

PART VII:

STABILITY OF THE STRUCURE ON PERMEABLE FONDATIONS

20.FLOW PATH IN PERMEABLE SOILS

20.1. SUB-SOIL DATA

The design should match the permeability and the bearing capacity of the sub-soil. Permeability is the measure of the soil ability to permit water to flow through its pores or voids.

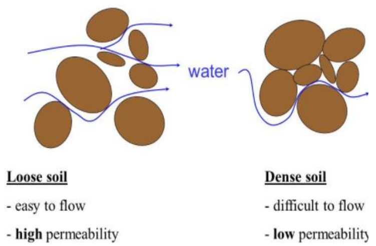


Figure 20.1*: soil permeability*

Explorations in the river bed shall be confined to periods of low river flows. Test pits through manual labour/back hoe will be excavated to expose the top stratum for physical examination, in-situ testing and sampling.

20.2. STREAM LINES

The stream lines represent the paths along which the water flows through the sub-soil. Every particle entering the sub-soil at a specific point upstream of the structure will trace out its own path and will represent a stream line. The stream line flow is a flow in which each liquid particle has a definite path and the paths of adjacent particles do not cross each other.

Every stream line possesses a difference of head. Further, at every intermediate point along the stream line, there is a residual head still to be dissipated in the remaining length to be traveled to the d/s end[1].

20.3. SUB-SOIL PRESSURE

The seepage pressure is due to the head difference between two points in a given mass of soil and it acts on the soil particles. The unit pressure is measured in kN/m^2 or in m (head).

The seepage flow, moving through the pores of the sub-soil, causes two types of sub-soil pressure affecting the stability of the structure:

1. **Direct underneath uplift pressure** throughout the sub-soil of the structure river or parent canal bed that tends to lift up the hydraulic structure floor**.**
2. **Upward raising pressure** d/s of the solid apron which causes sand particles to erupt upwards, creating the piping phenomenon (retrograde erosion).

[1] Khosla's theory shows that the loss of head along the flow net does not take place uniformly but depends upon the whole geometry of the structure, including the shape of the foundations, the u/s and d/s bed elevation etc.

21.PRINCIPAL CAUSES OF INESTABILITY

21.1. PRINCIPAL FAILURE OF A STRUCTURE

The design of a safe structure has to meet the surface flow requirements and guard against uplift pressure and seepage due to the residual force (head potential) of sub-surface water flowing from u/s end to the d/s end of a structure[1].

The main causes of failure of a structure constructed on a permeable foundation can be classified broadly into the two following categories.

21.1.1. Failure due to sub-surface flow

Failure by piping or undermining: the water from the u/s side continuously percolates through the bottom of the foundation and emerges at the d/s end of the cross-regulator floor. When the seepage water retains sufficient residual pressure at the emerging d/s end of the structure, it may lift up and remove the soil particles by scouring at the point of emergence, leading to increased porosity of the soil and formation of small cavities. A depression occurs under the structure which extends backwards towards the u/s through the bottom of the foundation (retrograde erosion). The structure may ultimately subsides in the hollow so formed, resulting in the failure of the structure[2].

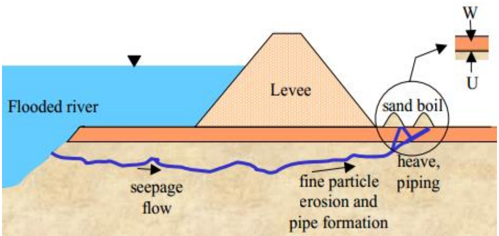


Figure 21.1: *seepage problem under levees (U: d/s uplift pressure force; W: submerged weight of soil)*

Failure by direct uplift: the percolating water exerts an upward pressure along the foundation of the cross-regulator. If this uplift pressure is not counterbalanced by the self weight of the structure, it may fail by rupture.

21.1.2. Failure due to surface flow

Unbalanced head due to standing wave: with super-critical flows, a hydraulic jump may develop. This jump causes a suction pressure or negative pressure which acts in the direction of the uplift pressure. If the thickness of the impervious floor is insufficient, then the structure fails by rupture (refer to articled 23.4.4).

[1] Calculated with the Khosla's method
[2] This failure is initiated by the sand boiling phenomenon, as shown in the above figure.

By scouring: a high discharge rate results in scouring effects on the d/s and u/s side of the structure. Due to scouring of the soil on both sides of the structure, its stability gets endangered by shearing.

21.2. ADOPTED SOLUTIONS AGAINST FAILURE

Solutions to the problems of instability due to the sub-soil pressures d/s of the structure may be obtained by lengthening the journey of water from beneath the structure (longer impervious floor, deeper cut-offs)[1]. In addition, the provision of filters (granular filter or fabric filter) underneath the u/s and d/s protective works of the structure is to check flowing out of fine subgrade material (refer to figure 24.1 and 24.3).

The scouring of the d/s channel bed is prevented by the construction of an adequate stilling basin and cut-off walls.

A failure by direct uplift due to the upward pressure of the percolating water along the foundation of the cross-regulator can be checked by increasing the self weight of the structure.

Those works can be expensive. For this reason, the trend is trying to reduce its dimensions in thickness as its length. Particularly in the case of structures of which large numbers need be constructed, the designer will be constrained by economic pressure to keep the structure as short as possible. In this case, hydraulic requirements will probably have the determining effect on the horizontal length. The length of the vertical cut-offs, especially the d/s one, will then be determined by the requirements for seepage resistance.

In practical terms, the designer must decide on an appropriate balance between the length of the horizontal elements (H) and the vertical ones (V) in order to arrive at an economic design for a particular type of soil; the V/H ratio of length 1/1.5 to 1/1.0 is widely adopted. Practices of design in a given country or region can effectively dictate this balance.

[1] More energy head will be dissipated by friction and other losses through the path of flow line ensuring the decrease of the exit gradient.

22. EXIT GRADIENT AND STRUCTURE LENGTH

22.1. HYDRAULIC GRADIENT

The hydraulic gradient is the rate of loss in unit pressure in kN/m^2 (or head in m) due to friction per unit of distance of channel at a given point and in a given direction of the flow path.

At the exit end of the structure, this gradient of pressure of water is called the exit gradient G_e.

The minimum length of the floor of the hydraulic structure is determined primarily from exit gradient considerations.

22.2. EQUATION OF THE EXIT GRADIENT

As per Khosla's method, for a standard form of a hydraulic structure as used in this User's Manual consisting of a floor length b with a vertical cut-off of depth d, the exit gradient at its d/s end is given by:

$$\boxed{G_e = \frac{Hs}{d} * \frac{1}{\pi * \sqrt{\lambda}}} \quad (22.1)$$

Where:

G_e = calculated exit gradient.

$H_{s\,max}$ = maximum hydrostatic head where G_e is acting (without flow) (m).

d = d/s vertical cut-off wall depth (m).

$1/\pi*\sqrt{\lambda}$ = calculated or estimated factor depending on the length of the floor of the structure and the depth of the d/s cut-off wall.

For the calculation of G_e, the maximum hydrostatic head $H_{s\,max}$ is the difference between the weir crest elevation (or the water surface elevation with closed gate and with or without breast wall) and the d/s practically dry channel bed elevation, where the exit gradient pressure is acting.

The above equation or its equivalent graphical form Khosla's pressure curves (not given in the Manual) gives a value of G_e equal to infinity if there is no d/s cut-off (d = 0 in the equation). It is therefore essential that a d/s sheet cut-off invariably be provided for any structure considered in the User's Manual.

22.3. SAFE EXIT GRADIENT

The exit gradient G_e is:

- Critical when the uplift (disturbing) pressure on the grain comprising the river bed material is just equal to the submerged weight of the grain at the exit point.

- Safe when a factor of safety (FoS) is used.

Values of the safe exit gradient G_{se} of the soil comprising the river bed material are given in the table below.

Table 22.1: *values of Khosla's safe exit gradient G_{se} for different types of soil*

Type of soil	Safe exit gradient G_{se}	
Fine sand	1/6 a 1/7	0.17 to 0.14
Coarse sand	1/5 a 1/6	0.20 to 0.17
Shingle	1/4 a 1/5	0.25 to 0.20
Clay	1/3 a 1/4	0.33 to 0.25

For example, a safe exit gradient G_{se} equal to 1/5 of the critical exit gradient G_e is necessary to maintain the structure safe on coarse sand.

22.4. DETERMINATION OF THE EXIT GRADIENT

To keep a structure safe against erosion (piping or other types), the value of the calculated exit gradient from equation 22.1 must be less than the value of the safe (permissible) exit gradient of the soil comprising the river bed material encountered from table 22.1.

22.4.1. **Estimated value of λ**

. Knowing the value of the maximum hydrostatic head $H_{s\,max}$ and the d/s cut-off depth, the value of $1/\pi*\lambda^{1/2}$ (and, therefore, G_e) can be obtained from the design chart shown in the following figure.

The α factor relates the floor length b of the structure with respect to the depth d of the d/s cut-off. For any value of α, the corresponding value of the factor **$1/(\pi*\sqrt{\lambda})$** factor may be deducted from curve Nº1 in the design chart.

22.4.2. **Calculated value of λ**

. Knowing the value of the maximum hydrostatic head $H_{s\,max}$ and the d/s cut-off depth, the value of G_e can be obtained from analytical form with the equation **$\lambda = [1 + (1 + \alpha^2)^{1/2}]/2$**, with known values for the structure solid floor length b.

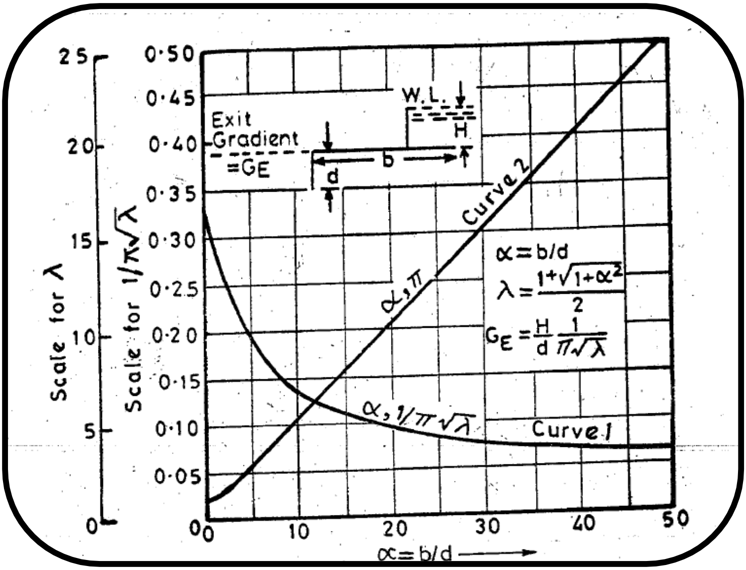


***Figure 22.1:** design chart of λ in relation to α for estimating the exit gradient* G_e

22.5. EXIT GRADIENT CONTROL

22.5.1. **Parameters of control**

The extent of scour of the bed material varies at different places along a hydraulic structure. The control of the exit gradient depends upon two parameters:

- The length of the solid floor.
- The depth of end cut-off wall.

22.5.2. **Length of the solid floor**

Naturally longer the seepage lines of flow through the soil, more is the head loss and thus lesser the uplift pressure and the value of exit gradient.

In case of G_e more pronounced, the length b of the impervious floor is extended or the depth d of the cut-off made deeper in order to reduce the value of G_e until a secure value be reached. Therefore, this approach allows the user to analyze alternatives to reach a more economic design (refer to article 21.2).

22.5.3. **Depth of cut-off wall**

The depth of end cut-off wall is also helpful in reducing the exit gradient value and thus the risk of piping (refer to article 19.2).

23.UPLIFT SUB-SOIL PRESSURE AND FLOOR THICKNESS

23.1. FLOOR THICKNESS

The structure floor should be thick enough to resist the uplift pressure and withstand wear by moving bed load and suspended material and the impact of stones and flowing water.

The determination of the hydrostatic pressure and the weight of the floor itself counteract the destabilizing uplift pressure directly under the floor. The thickness of the floor is calculated from the balance of both pressures, taking into account the corrections to be applied as shown in the following equation:

$$\boxed{t_{floor} = (H_{max} * \phi_c) / u} \quad (23.1)$$

Where:

t_{floor} = thickness of floor at specified cross-section (m).

φ_c = corrected uplift pressure at specified cross-section (in % of the total uplift).

H_{max} = maximum static or dynamic head at specified cross-section (m).

u = submerged specific gravity or density of floor material.

The maximum difference of head and hence the maximum uplift pressure imposed on the structure is when the water reaches its highest level with closed gate(s) without any discharge passing down the structure. When a discharge is passing through the structure, the maximum (seepage) head is the difference between the u/s and d/s water level.

Floors can be designed with stone masonry covered by a 0.25 m protection layer of reinforced concrete or with thinner PCC/RCC.

23.2. STANDARD PROFILES

In the User's Manual, for the calculation of sub-soil uplift unit pressure (kN/m^2) for designing hydraulic structures on pervious foundations, the Khosla's method of independent variables is used, breaking a complex profile like that of a cross-regulator structure into a number of simple standard profiles, each of which can be solved mathematically, as shown in the following figure:

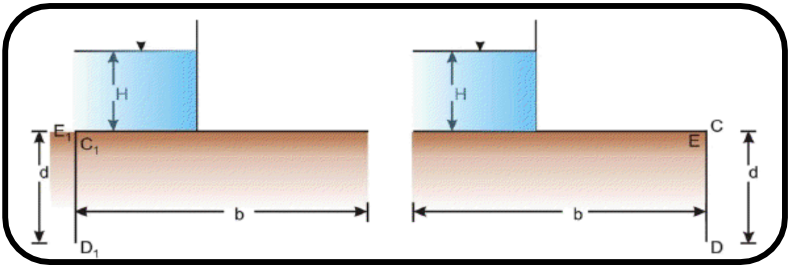


***Figure 23.1**: standard profiles with (a) u/s sheet pile and (b) d/s sheet pile and key points E, C and D*

23.3. UPLIFT PRESSURE

23.3.1. Key points

The key points of the standard profiles, where the uplift pressure is computed, are:

- U/s and d/s junction points of the sheet pile with the floor.
- Bottom point of each sheet pile.
- Bottom corners in case of depressed floor and/or at the foot of the glacis.

The maximum hydrostatic head $H_{s\ max}$ considered in the case of floor thickness calculation is the head difference between the u/s water elevation up to the highest level (weir crest elevation or pond water surface elevation[1] with closed gate) and the d/s practically dry cistern bed elevation. In other words, seepage pressure is the most dominating for gates closed conditions ($H_{s\ max}$), average during normal flow and almost none during high flow as shown in the following figure:

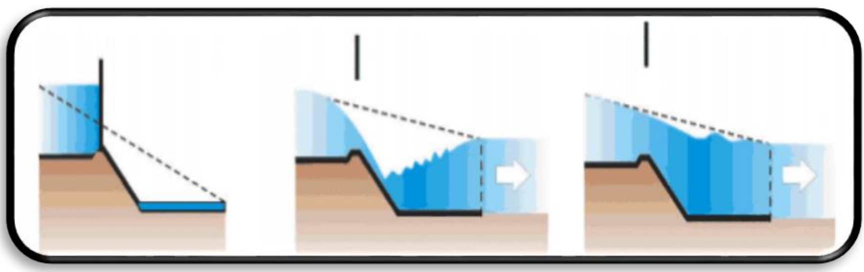


***Figure 23.2:** profiles of the sub-soil seepage grade line*

[1] With cross-regulator without shutters, crest elevation equals pond elevation.

Mathematical solutions[1] for the determination of the residual hydrostatic head percentage at various key points of the flow net in a vertical plan for the adopted simple standard profiles are presented in the form of equations given below.

23.3.2. Upstream pile

For the u/s pile, the main equations are:

φ_{E1} = 100 % of height of the maximum hydrostatic head $H_{s\,max}$.

φ_{C1} = 100% of height of the maximum hydrostatic head $H_{s\,max} - \varphi_E$.

φ_{D1} = 100 % of height of the maximum hydrostatic head $H_{s\,max} - \varphi_D$.

23.3.3. Downstream pile

For the d/s pile, the main equations are:

$\varphi_{E2} = (1/\pi)\ COS^{-1}((\lambda-2)/\lambda$

φ_{C2} = 0% of height of the maximum hydrostatic head $H_{s\,max}$

$\varphi_{D2} = (1/\pi)\ COS^{-1}((\lambda-1)/\lambda$

23.4. CORRECTIONS OF UPLIFT PRESSURE

However, the reality does not comply with the hypothesis of a standard simplified independent profile for the individual key points of the hydraulic structure.

The real profile is a complex profile specific to the whole structure. Therefore, the application of simple standard profiles is valid for any complex profile only if the percentage pressure at the key points mentioned above is corrected for:

1. Correction for mutual interference of piles or cut-off walls.
2. Correction for floor thickness.
3. Correction for slope of floor only if a cut-off is positioned at the start or end of the slope[2].

23.4.1. Correction for mutual interference of cut-off walls

The correction to be applied at C_1 or E_2 as percentage of head $H_{s\,max}$ due to the effect of the presence of the other cut-off wall is given by:

$$\boxed{\mathbf{C_{interference} = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right)}} \quad (23.2)$$

<u>Where</u>:

$C_{interference}$ = correction to be applied at C_1 or E_2 due to mutual interference of piles (%).

b = total floor length (m).

[1] In the Manual, to enhance the precision, we only use the analytical approach and not the Khosla's abacus of pressure curves.

[2] In this User's Manual, this correction is neglected owing that, usually, there is no cut-off at the toe of glacis.

d = depth of the cut-off on which the effect is considered (m).

D = depth of the cut-off, the influence of which has to be determined on the neighboring cut-off (measured below the level at which interference is desired) (m).

b' = c/c distance between the 2 cut-off walls considered (d & D) (m).

The correction due to the interference is positive in the rear of the pile (points C) and subtractive for the points in the forward direction of the flow (points E).

23.4.2. **Correction for the floor thickness**

The thickness of the (concrete) floor is designed according to the (sub) up-thrust pressure, using the weight of gravity in each reach. Therefore, the pressure percentages φ_{E1} and φ_{C1} calculated by the Khosla's equation[1] pertain to the top level of the floor.

In the standard form profiles (refer to figure 23.1), the u/s floor is assumed to have negligible thickness, since the uplift pressure is more than counterbalanced by the specific weight of the water γ above the floor. The u/s floor however needs a minimum thickness in order to withstand wear by moving bed load and suspended material and the impact of flowing water.

The corrected pressure at C_1 with an u/s cut-off wall is calculated with the following equation (the correction is positive at C_1):

$$\boxed{\mathbf{Correction_{thick} = [(\phi_{D1} - \phi_{C1}) * t_{C1}] / d_1}} \quad (23.3)$$

Where:

Correction$_{thick}$ = correction to be applied at C_1 due to u/s floor thickness as % of head $H_{s\,max}$ (%).

φ_{D1} = % of $H_{s\,max}$ at D_1 (%).

φ_{C1} = % of $H_{s\,max}$ at C_1 (%).

t_{C1} = assumed u/s floor nominal thickness (m).

d_1 = u/s cut-off depth including floor nominal thickness (m).

The corrected pressure at E_1 with a d/s cut-off wall is calculated with the following equation (the correction is negative at E_2):

$$\boxed{\mathbf{Correction_{thickness} = [(\phi_{E2} - \phi_{D2}) * t_{C2}] / d_2}} \quad (23.4)$$

Where:

Correction$_{thickness}$ = correction to be applied at E_2 due to d/s floor thickness as % of head $H_{s\,max}$ (%).

φ_{E2} = % of $H_{s\,max}$ at E_2 (%).

[1] The percentage pressures ϕ_{E1} y ϕ_{C1} can also be estimated thanks to available Khosla's abacus (not given in the User's Manual).

φ_{D2} = % of $H_{s\,max}$ at D_2 (%).
t_{C2} = assumed d/s floor thickness (m).
d_2 = d/s cut-off depth including floor thickness (m).

23.4.3. **Correction for floor slope**

A correction is applied for the slope of the floor, only to the key points of the pile line fixed at the start or the end of the slope. This correction is neglected owing that, usually, there is no cut-off at the toe of glacis.

In most of the structures considered in the User's Manual, the presence of a cut-off wall at the toe of the glacis is not necessary. Therefore, there is no correction for mutual interference and slope at this point.

23.4.4. **Correction in the jump trough**

In the User's Manual, the percentage of pressure in the jump trough is computed at the toe of the glacis considered as the theoretical base point of the jump formation and is obtained with the following equation:

$$\boxed{\phi_t = \phi_{E2c} + [(\phi_{C1c} - \phi_{E2c}) / b] * b_{basin}} \quad (23.5)$$

<u>Where</u>:
φ_t = % of $H_{s\,max}$ or $H_{d\,max}$ at toe of glacis (%).
b = total floor length (m).
b_{basin} = basin floor length (m).
φ_{E2c} = % of $H_{s\,max}$ at E_2 corrected for mutual interference and floor thickness (%).
φ_{C1c} = % of $H_{s\,max}$ at C_1 corrected for mutual interference and floor thickness (%).

In the jump trough, for the determination of the maximum head, we must examine two alternatives: the presence or absence of water flow. The thickness of the floor is calculated from the balance of both pressures: hydrostatic uplift pressure and dynamic pressure.

No water flow conditions (static conditions)

The maximum hydrostatic head $H_{s\,max}$ and, hence, the maximum uplift pressure imposed on the structure is the difference between the u/s water surface elevation due to the presence of the weir (or with the gate(s) closed in case of an off-take structure or discharge regulator) and the dry cistern elevation[1] (refer to figure 23.2).

The unbalanced hydrostatic head at the toe of glacis $H_{s\,glacis}$ due to the maximum uplift pressure $H_{s\,max}$ and therefore the level of the sub-soil hydrostatic grade line at the same point is given by the following equation:

[1] The water surface elevation may be taken because $v^2/2g = 0$.

$$\boxed{\mathbf{H_{s\ glacis} = \Phi_t * H_{s\ max}}} \quad (23.6)$$

<u>Where</u>:

$H_{s\ glacis}$ = unbalanced hydrostatic head at toe of glacis (m).

$H_{s\ max}$ = maximum hydrostatic head (m).

φ_t = uplift pressure at the jump location (in % of the total uplift pressure in key point E_1).

The unbalanced hydrostatic head at toe of glacis $H_{s\ glacis}$ is also the difference between the sub-soil hydrostatic pressure elevation at this point and the cistern elevation at the same point.

Water flow conditions (dynamic conditions)

When a certain discharge rate is passing over the weir with the formation of a hydraulic jump, the maximum dynamic head $H_{d\ max}$, neglecting the velocity head, is the difference between the u/s water surface elevation and the d/s water surface elevation[1] (refer to figure 23.2).

To avoid plotting the jump water surface elevation for knowing the dynamic head H_d at the formation of the jump[2], the unbalanced dynamic uplift pressure (head) at the toe of glacis $H_{d\ glacis}$ may approximately be calculated, using the following equation:

$$\boxed{\mathbf{H_{d\ glacis} = 0.5\ (y_2 - y_1) + \phi_t * H_{d\ max}}} \quad (23.7)$$

<u>Where</u>:

$H_{d\ glacis}$ = unbalanced dynamic head at toe of glacis (m).

φ_t = uplift pressure at the jump location (in % of the total uplift pressure in key point E_1).

$H_{d\ max}$ = maximum dynamic head (m).

Refer to list of symbols.

The dynamic uplift pressure (in m head) $H_{d\ glacis}$ due to the dynamic action of the water flow where the jump is forming is compared to the hydrostatic head $H_{s\ glacis}$ at the same point (toe of glacis). The thickness of the glacis floor is designed for the dynamic head $H_{d\ glacis}$ or the hydrostatic head $H_{s\ glacis}$, whichever is greater and divided by the submerged density of the floor material (concrete or masonry).

The inflow conditions such as the velocity may modify substantially the flow properties and affect the jump characteristics and its classification. The bottom pressure fluctuation below the hydraulic jump may create negative pressures that could lead to uplift pressure on the cistern floor. Therefore, since the base

[1] We take the tailwater level in order to avoid plotting the water surface profile after the jump. Theoretically, it is necessary to plot the post jump profile to determine $H_{s\ max}$.

[2] Here considered at the toe of glacis (refer to article 10.5.3).

point of the jump formation is likely to shift with the variation in discharge rate passing over the weir, the entire glacis has to be designed with the same floor thickness from u/s to d/s.

23.5. HEADS AND GRADIENTS

In the User's Manual spreadsheet, the green curve represents the elevation of the water surface profile and the black curve represents the elevation of the structure floor.

The sub-soil hydraulic grade line is drawn in the spreadsheet for the maximum static head for high flood conditions (dotted red curve) and the no flow conditions (solid red curve).

The (unbalanced) dynamic head is represented by the difference of elevation between the static head for high flood conditions and the water surface profile at the same vertical section (green curve). The (unbalanced) static head is represented by the difference of elevation between the sub-soil hydraulic grade line and the structure floor elevation (black curve) at the same section. This unbalanced head is compensated by the floor thickness, taking into account the buoyancy.

PART VIII:

UPSTREAM AND DOWNSTREAM PROTECTIVE AND TRANSITION WORKS

24.UPSTREAM AND DOWNSTREAM PROTECTIVE WORKS

24.1. LOCATION

Upstream and downstream of the solid floor of the hydraulic structure, the channel bed is protected by certain methods like concrete blocks protection, loose stones protection, gabion boxes protection etc.

U/s of the structure, the flow velocity is lower than the approach (or accelerated) velocity in the structure where the water depth decreases (refer to table 25.1). D/s of the structure, in the stretch of expansion, the speed is reduced and the water surface elevation rises leading to asymmetry of flow and concentrated velocity (refer to table 25.1).

24.2. TYPE OF PROTECTION WORKS

Protection should, ideally, be flexible so that it can respond to changes in the channel reach. The u/s and d/s protective works are of two types:

- Filter (granular, inverted or graded filter or fabric filter).
- Launching apron.

Many variations can be made on the basic filter construction. One or more layer can be replaced or only the revetment layer is maintained, while the underlyning layers are replaced by one single layer such as:

- Gabion boxes (matresses) on fine gravel.
- Concrete blocks on nylon filter (geotextile).

No concentration of flow is taken while designing the protective works.

24.3. INVERTED GRANULAR FILTER

24.3.1. Description

If the protective lining were to be installed on top of the fine material in which the canal is excavated, grain of this subgrade would be washed through the openings of the revetment[1] (riprap or concrete blocks). To avoid damage to the protective lining due to the washing of the subgrade, a filter must be placed between the revetment (riprap or concrete blocks) and the subgrade.

The inverted graded filter consists of 2 layers of graded materials of increasing permeability from bottom to top, as shown in the following figure. The protective construction as a whole and each separate layer of the filter must be sufficiently permeable to water entering the canal through its bed or bank. Further, fine material from an underlying filter layer or from the original material (subgrade) must not be washed into the void of the covering layer.

[1] This process is partly due to the turbulent flow of canal water in and out of the voids between the stones and partly due to the inflow of water that leaks around the structure or flows into the canal.

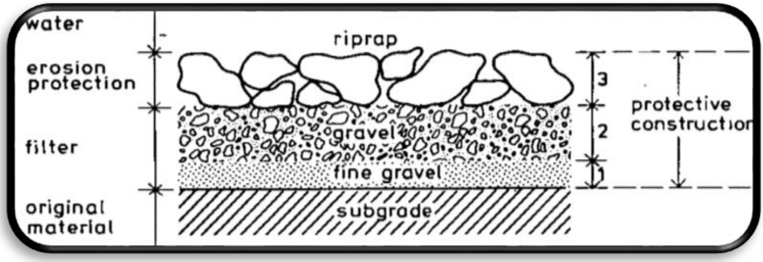


Figure 24.1: *inverted filter of graded gravel laying between the riparap protection layer and original material (subgrade) in which canal is excavated*

The protective construction as a whole and each separate layer must meet the following 3 conditions:

1. Geometrical conditions: the protective construction as a whole and each separate layer must be sufficiently permeable for water entering the canal through its bed or bank.
2. Hydraulic conditons: the fine material from the underlying filter layer must be sufficiently permeable to avoid pressure build up (such as the exit gradient).
3. Stability conditions: fine material from the filiter layers or from the subgrade must not be washed into the void of the covering layer.

To prevent the filter from dislocation under the surface flow, a revetment (concrete blocks or riprap) is laid over the filter material.

To obtain a fair grain size distribution throughout the filter layers, each layer should be sufficently thick as indicated in the following table:

Table 24.1: *minimum layer thickness for filter construction made in the dry*

Layer	Minimum thickness (m)
Fine gravel	0.05 to 0.10
Gravel	0.10 to 0.20
Revetment	Varies

To prevent from fine material of the filter being dragged into the holes in the top layer of the filter, the gradation should be such that, while it allows free flow of seepage water, the subgrade or foundation material does not penetrate to clog the filter. For the stability of each layers (refer to figure 24.1), the following requirements must be met:

1. d_{15} layer 2/d_{85} layer 1 ≤ 5.
2. d_{15} layer 1/d_{85} subgrade ≤ 5.

24.3.2. Concrete blocks revetment

Just beyond the u/s or d/s impervious floor, pervious revetment comprising of cement concrete blocks of adequate size laid over the filter (packed stones[1], inverted graded filter) or fabric filter (geotextile) is to be provided, as shown in the following figure:

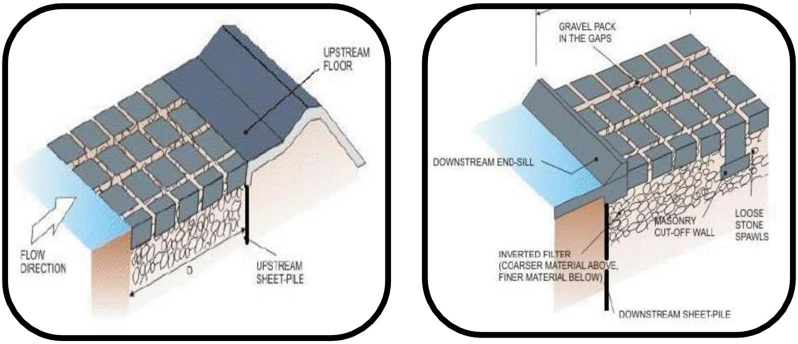


Figure 24.2: *u/s and d/s block protection works on filter*

The concrete block size and thickness according to the canal discharge are given by the following table:

Table 24.2: *concrete block size according to canal discharge*

Canal discharge (m^3/s)	**Block size (m^3)**
≤ 1	0.6 * 0.6 * 0.20
1 to 5	0.6 * 0.6 * 0.25
5 to 30	0.6 * 0.6 * 0.40
30 to 100	0.6 * 0.6 * 0.60

The gap between the blocks shall not be greater than 0.05 m and shall be packed with pebbles.

The following figure presents the action of an inverted graded filter lying under blocks in order to prevent the loss of soil through the joints.

[1] Gravel or crushed rock.

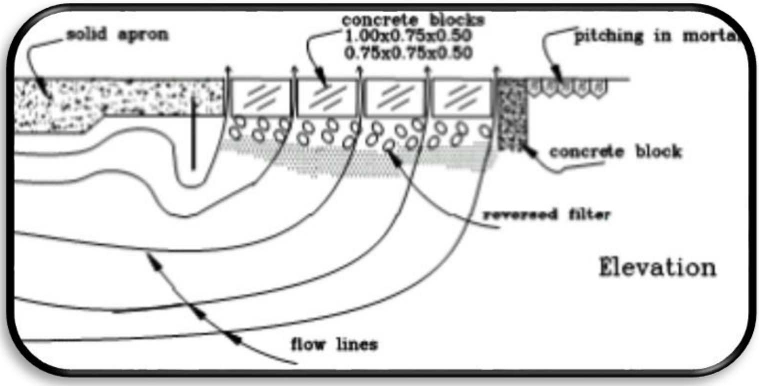


Figure 24.3: *flow lines with concrete blocks revetment on inveted graded filter*

24.3.3. Riprap revetment

A riprap is a revetment of broken rock or cobbles placed on a random fashion. It is made from a variety of rocks or stones types, as shown in the opposite figure.

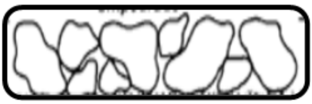


Figure 24.4: *riprap revetment*

Riprap is used where a rough surface is needed to resist wave erosion, to protect against rainfall runoff (gullying), and to dissipate high flow velocity energy as revetment in embankment protection.

The stones should be predominantly angular in shape and nearly alike in all directions so that they can easily move to lower depths without being hindered by too large stone. River bed stone tend to be rounded and are more suitable for masonry works and for filling gabion boxes.

24.3.4. Stone piching revetment

A stone pitching revetment is a regularly sized and shape stones or concrete blocks placed in an ordered fashion as protection against erosion.

Pitching will be used where a finished horizontal or sloping surface is required as a hydraulically smoother protection than riprap. Slopes of embankments should not be steeper than 1:1.

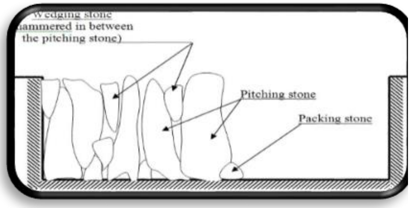


Figure 24.5: *pitching revetment*

Like for riprap, pitching is placed on a sand/gravel bedding.

The large pitching stone are put first, with its longest axis perpendicular to the surface. The smaller packing stone is driven in by hammer to support the pitching stone and fill up some voids. The much smaller spalls are used for wedging and to fill gaps to produce an even surface, without projection above the neat lines shown on the figure. Surface grounting may be considered.

The diameter and weight of the pitching stones to be selected depend upon the maximum flow velocity expected and the side slope of the embankment as shown in the graph of the following figure:

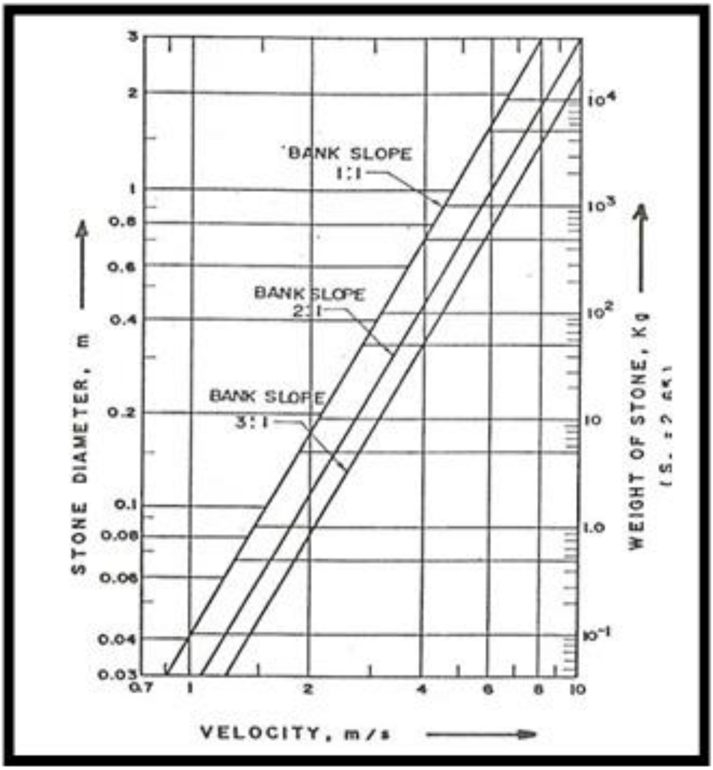


Figure 24.6: *size of pitching stone versus flow discharge rate*

24.3.5. **Boulders revetment**

Boulders are basically rocks that are too big to pick up without mechanical help. They are usualy used in embankments to break up the flow of storm-water and to stop erosion.

24.4. LAUNCHING APRON

Beyond the filter protective works, a flexible launching apron is provided to settle into the local scour hole formed around the structure in order to prevent the erosion.

For practical reasons, the thickness of the flexible apron is the same as the thickness of the filter system.

24.5. LENGTH OF PROTECTIVE WORKS

The length of protective works is determined by the scour depth D under the original river bed (Lacey equation) with a safety factor (FoS) depending on the type of structure (piers, river bank protection etc.).

Guidelines for the selection of the length of the protective works are given below.

$$\boxed{L = D * FoS} \quad (24.1)$$

Where:

L = length of protective layer (m).

FoS = safety factor (= 1,5 to 2 for d/s works; = 1.25 to 1.5 for u/s works).

D = refer to equation 19.1 (m).

For practical reasons, the thickness of the flexible apron is the same as the thickness of the filter system.

24.6. ROCK SIZING EQUATIONS FOR REVETMENT

Several factors affect the stones or rocks size of a launching apron or a riprap to resist the forces of flow which tend to move them. These factors are bottom flow velocity, flow direction, turbulence and waves. Due to the possible combination of these factors, the velocity at which the water strikes the riprap is rather unpredictable unless the structure is tested in a laboratory.

The stability of a particular riprap particle is a function of its size, expressed either in terms of its weight (specific gravity) or equivalent diameter. The rock sizing suitable for rocks placed within a zone of highy turbulent water (immediately d/s of the end sil of the energy dissipater) is given by the following formula:

$$\boxed{d_{50} = 0.081 * V^{2.26} / (s_r - 1)} \quad (24.2)$$

Where:

d_{50} = nominal rock size (diameter) of which not more than 50% of the rocks are smaller (m).

V = flow velocity (m/s).

s_r = rock specific gravity (dimensionless).

The rock specific gravity is given in the following table:

Table 24.3: *rock type and specific gravity*

Type of rocks	Specific gravity s_r
Sand stone	2.2 to 2.4
Granite & limestone	2.6
Basalt	2.9

The following curve permits to determine the stone diameter of the protective layer to be used d/sor u/s of the structure, in terms of flow velocity leaving the structure[1] based on discharge rate divided by cross-sectional area.

The average velocity above a end sill of the basin is calculated by dividing the discharge by the cross-sectional area of flow above the end sill (if no stilling basin is needed, the channel average velocity is used with Q_{max}). It is assumed that $d_{40}/d_{50} = 0.75$.

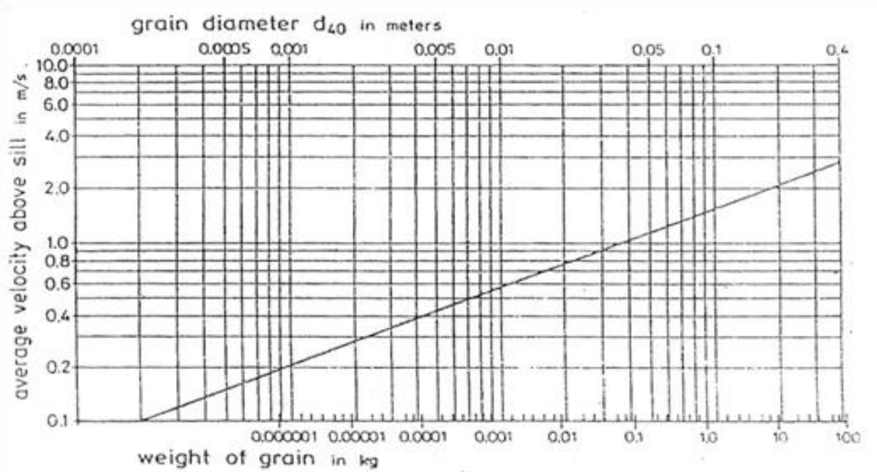


Figure 24.8*: curve to determine stone size (d_{40} in m and weight in kg) as a function of the average velocity of flow*

With d_{40} (the particule average size wich 40% of the material is finer), more than 60% on the stone mixture of the protective layer should consist of stones of sizes greater than d_{40}, should be as homogeneous as possible in length, width and thickness and should be of curve size (or curve weight or heavier).

The grading of the stone revetment should be as follows:

- Maximum stone size = $1.5*d_{50}$.
- Minimum stone size = $0.5*d_{50}$.

[1] To find the stone diameter, we should use the bottom velocity striking the protective layers. However, this bottom velocity is rather unpredictable. For practical purposes, the average velocity is used.

25.TRANSITION STRUCTURES AND CHANNEL SIDE SLOPE

25.1. INTRODUCCION

Transition structures are required to guide the flow with a change of cross-section (e. g. width, bottom etc.) designed to be accomplished in a short distance with a minimum amount of flow disturbance and losses.

A transition in its general form may have a change of channel shape (from the usual trapezoidal earth canal into the structure with usually vertical walls), a provision of a hump or a depression and contraction or expansion of channel width, in any combination. In addition, there may be various degrees of loss of energy at various components.

A channel transition is usually designed so that the losses in the transition are small. The form, friction and energy losses in the transition may be neglected and, consequently, the energy equation is appropriate for the analysis[1] and implies conservation of total head H_L.

The principal types of transition are as follows:

- Change in bed level with constant width.
- Change in channel width with constant bed level.

25.2. CHANGE OF SECTION AND SPECIFIC ENERGY

25.2.1. Change in bed level

The variation of water level is produced by a raised or a drop of the canal bed level (ΔZ), with the channel width B remaining unchanged.

Upstream sub-critical flow

A sub-critical flow (with specific energy E_1 and discharge per unit width q_1) is approaching a region where the bed is raised. No mechanical (friction) losses are considered between sections 1 (before the hump) and 2 (on the hump)[2]. The following situations occur for the water surface on the hump:

- $\Delta Z < y_c$: with a raised bed level (considered as a broad crested weir) too low for critical flow to occur, the water level y_2 at the constriction drops with unaffected u/s level y_1.or y_{init}) At this hump elevation, E_2 decreases by ΔZ[3], but still exceeds the minimum specific energy E_{min} for this discharge. The flow remains sub-critical over the hump and

[1] Energy losses if considered are consisting of friction loss and conversion loss. The friction loss is estimated by n in the Manning's formula (negligible). The conversion loss is expressed in terms of the change in the velocity head (acceleration of the flow at inlet and drop in water surface; reduced flow velocity at outlet and rise in water surface).

[2] This region is sufficiently long for parallel flow to be established (hence "broad-crested"), but insufficiently long for significant frictional losses.

[3] Specific energies at sections 1 and 2 are given by: $E_1 = y + V^2/2g$ and $E_2 = E_1 – \Delta Z$.

resumes its original depth d/s of the hump (refer to figures 25.1a & 25.1b).

- $\Delta Z = y_c$: at this point, the critical height for the given discharge is reached at the constriction. The flow reaches the critical energy $E_2 = E_{min} = E_c$ (refer to figures 25.1a & 25.1b & 5.1; no backwater curve).
- $\Delta Z > y_c$: once critical flow is achieved at the constriction, if the hump height is increased, the given flow will still go critical on the hump and remain at the critical depth y_c (it will not and cannot fall below this value) as the specific energy E cannot be less than E_{min} as shown in the below figure. To take into account ΔZ, only the u/s conditions can change with an increase of specific energy and the u/s flow must "back up" (refer also to figures 1.5 & 14.4).

What happens further d/s depends on other controls; the flow can be super-critical or sub-critical.

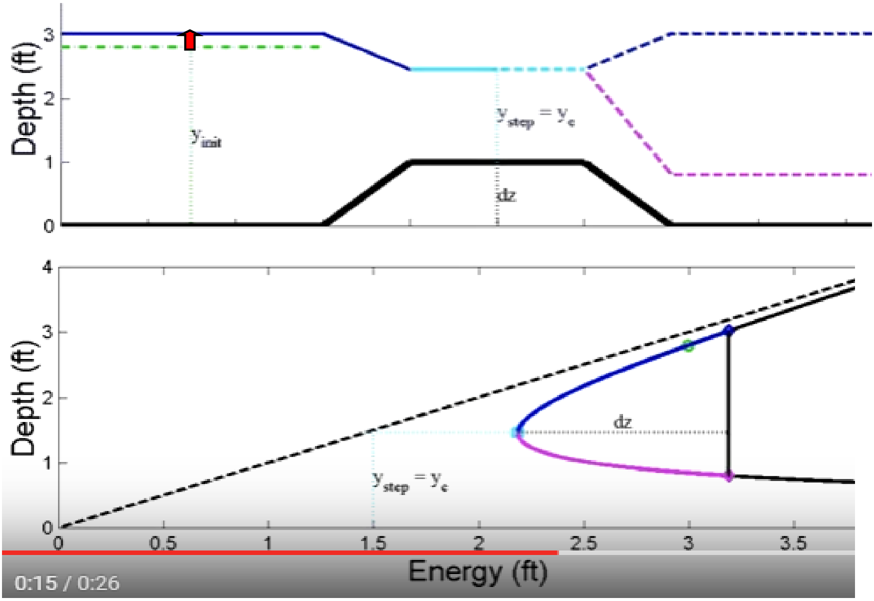


Figure 25.1a: *hump and water depth with u/s sub-critical*

Upstream super-critical flow

A super-critical flow (with specific energy E_1 and discharge per unit width q_1) is approaching a region where the bed is raised[1]. No mechanical (friction)

[1] This region is sufficiently long for parallel flow to be established (hence "broad-crested"), but insufficiently long for significant frictional losses.

losses are considered between sections 1 (before the hump) and 2 (on the hump). The following situations occur for the water surface on the hump:

- $\Delta Z < y_c$: the water surface on the hump will rise due to a decrease in the specific energy as shown in the following figure.
- $\Delta Z = y_c$: by raising more the hump, the critical height for the given discharge will be reached. The flow will reach the critical energy $E_2 = E_{min} = E_c$ (refer also to figure 5.1; no backwater curve).
- $\Delta Z > y_c$: once critical flow is achieved at the constriction, if the hump height is increased even more, sub-critical flow will occur and a hydraulic jump will take place.

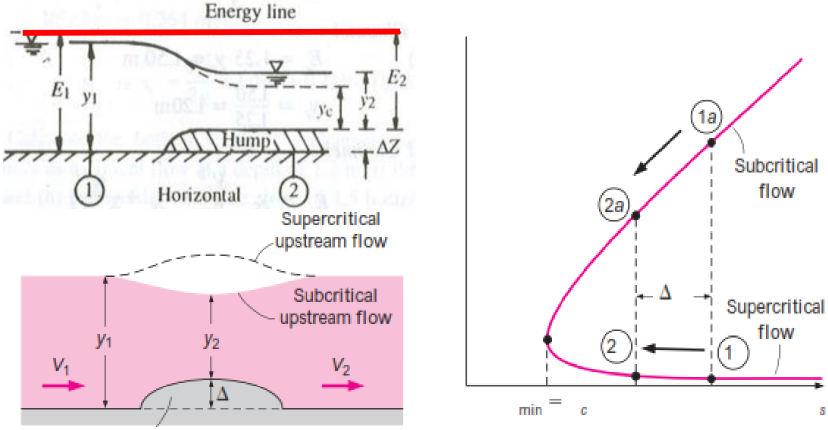


Figure 25.1b: *hump and water depth with u/s sub-critical or super-critical flow*

Excessively tall broad crested weir is not a problem in terms of water measurement or calibration; it is only troublesome with respect to unnecessarily raising the u/s water surface elevation and increasing costs.

25.2.2. **Change in channel width**

The variation of water level is produced by a reduction/increase in the channel width with the channel bottom remaining horizontal. It is convenient to analysis the flow in terms of the discharge intensity q (refer to figure 9.3 with constant specific energy).

Channel constriction with u/s sub-critical flow

With a horizontal contraction ($B_2 < B_1$), the specific energy at u/s section is equal to the specific energy at d/s (constriction) since the bed elevation at both sections is the same. Based on the constant specific energy graphs (refer to figure 9.3), the water depth y_2 decreases with an increase of q ($q_2 > q_1$) and F_r.

The limit of the contracted width is obviously reached when corresponding to the critical depth y_c where $q = q_{max}$ for a given specific energy and critical-flow condition at the constriction (F_r = 1). With a further reduction in the channel width, the flow will not be possible with the given u/s conditions; the u/s E will increase and the water level will have to raise provoking a decrease in velocity (the water depth may be different but remains critical at the constriction).

What happens further u/s or d/s depends on other controls and the flow can be super-critical or sub-critical as shown in the following figure:

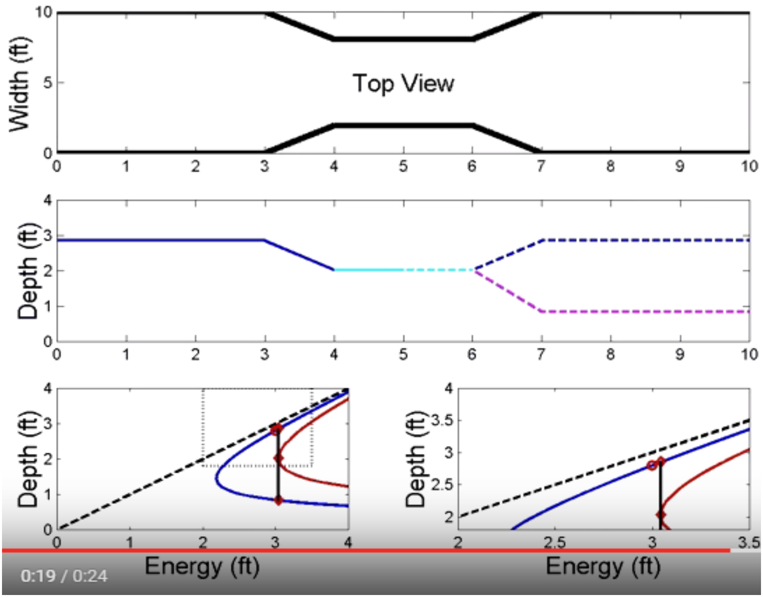


Figure 25.2: *constriction and unit discharge with critical depth (u/s sub-critical flow)*

Channel constriction with u/s super-critical flow

If the u/s flow has super-critical flow conditions, a reduction of the channel width (and hence an increase in the discharge intensity) causes a rise in water depth.

Channel expansion

When the channel width increases, lower flow velocities take place in the downstream section. The sediment transport capacity is smaller in the downstream reach and accretion may take place[1].

[1] This geometry is well suited to the design of settling basins: e.g. at the intake of an irrigation channel, a settling basin will trap sediment materials to prevent siltation of the irrigation system.

Table 25.1: *summary table of changes in canal section*

Section	Type of change	Characteristics	Water depth y_2 at new section
Constriction	$\Delta Z \neq 0$ Raised canal bed level ($E_1 = E_2 + \Delta Z$)	$B_2 = B_1$ $q_2 = q_1$ (eq. 10.2b) $E_2 < E_1$ (eq. 10.1) $y_{c2} = y_{c1}$ (eq. 10.2a) $E_{min2} = E_{min1}$ (eq. 9.1a)	$y_2 < y_1$ with u/s sub-critical flow $y_2 > y_1$ with u/s super-critical flow
	$\Delta Z = 0$ Decrease of canal width ($E_1 = E_2$)	$B_2 < B_1$ $q_2 > q_1$ (eq. 10.2b) $E_2 = E_1$ (eq. 10.1) $y_{c2} > y_{c1}$ (eq. 10.2a) $E_{min2} > E_{min1}$ (eq. 9.1a) (1)	$y_2 < y_1$ with u/s sub-critical flow $y_2 > y_1$ with u/s super-critical flow
Expansion	$\Delta Z \neq 0$ Drop in canal bed level ($E_1 = E_2 - \Delta Z$)	$B_2 = B_1$ $q_2 = q_1$ (eq. 10.2b) $E_2 > E_1$ (eq. 10.1a) $y_{c2} = y_{c1}$ (eq. 10.2a) $E_{min2} = E_{min1}$ (eq. 9.1a)	$y_2 > y_1$ with u/s sub-critical flow $y_2 < y_1$ with u/s super-critical flow
	$\Delta Z = 0$ Increase of canal width ($E_1 = E_2$)	$B_2 > B_1$ $q_2 < q_1$ (eq. 10.2b) $E_2 = E_1$ (eq. 10.1) $y_{c2} < y_{c1}$ (eq. 10.2a) $E_{min2} < E_{min1}$(eq. 9.1a) (2)	$y_2 > y_1$ with u/s sub-critical flow $y_2 < y_1$ with u/s super-critical flow

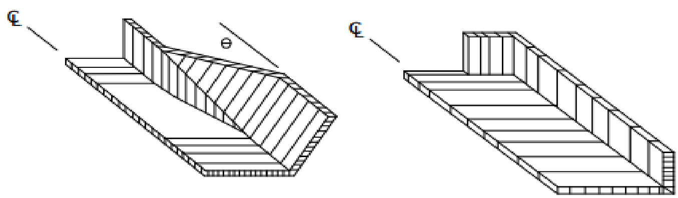
(1) Specific energy graph curve shifts to the right ($q_2 > q_1$) and up ($E_{min2} > E_{min1}$)
(2) Specific energy graph curve shifts to the left ($q_2 <$ q1) and down ($E_{min2} < E_{min1}$)

25.3. TYPES OF TRANSITIONS

25.3.1. Transitions and wing walls

The variation of water level is produced by a reduction/increase in the channel width with the channel bottom remaining horizontal. It is convenient to use specific energy graph (refer to figure 9.2a).

The main designs of transitions are shown in the following figure:



Wedge Abrupt

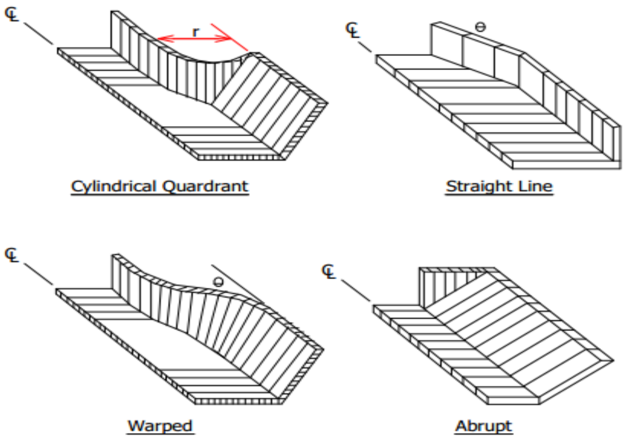


***Figure 25.3**: isometric view of transitions*

The most common transition structures used in the User's Manual are the abrupt transition wall (especially in the d/s diverging transition where energy recovery is usually not necessary). This transition should be vertical walled with walls at 45º to the canal center line. It may be used directly without elaborated design. An apron, which is an integral part of the transition, protects from erosion the channel bottom at the outlet[1].

Special transition structures such as wrapped wall are useful when the conservation of energy is essential because of allowable head water considerations such as an irrigation structure in sub-critical flow.

It is left to preference of the engineer which type he/she prefers but complex transition structures such as the warped type are costly and only warranted if the least amount of energy loss must be afforded.

25.3.2. Head loss

If the losses with transition are accounting ($E_1 = E_2 + H_L$), the head loss is given by the following formula:

$$\boxed{H_L = K * V^2 / 2g} \quad (25.1)$$

Where:

H_L = energy head loss in transition (m).

[1] Sudden expansion (or contraction) ratios like 1:1 or 2:1 are not very effective for energy conversion because the high velocity jet leaving the throat cannot suddenly change direction to follow the boundaries of the transition. In the flow separation zones that result, eddies are formed that convert kinetic energy into heat and noise.

K = coefficient depending of the type of transition[1].

The transition head losses of inlet and outlets are summarized in the following figure:

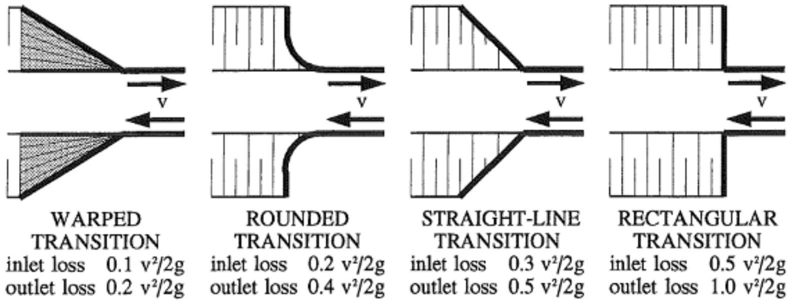


Figure 25.4*: transition head losses of inlets and outlets*

25.4. CHANNEL SIDE SLOPE

Erosion most commonly occurs when the velocity of flow exceeds the velocity at which the soil of the channel will erode (refer to figure 8.1). Erosion can be prevented by lowering the velocity below the soil-erosion velocity (change of channel geometry), by lining the natural channel material with a more erosion-resistant material or by changing the canal side slopes. The following table shows the recommended canal side slopes.

Table 25.2: *recommended canal side slopes*

Type of channel	Side slope (H:V)
Firm rock	Vertical to ¼:1
Fissured rock	½:1
Stiff clay	½:1
Firm earth with stone lining	1:1
Firm earth and large channel	1:1
Firm earth and small channel	1½:1
Loose sandy earth	2:1
Sandy porous loam	3:1

25.5. STRUCTURE FREEBOARD

The freeboard of a channel is the vertical distance from the top of the channel to the water surface at the design condition. The distance should be sufficient to absorb sudden changes in water surface elevation due to errors in water management or rainfall runoff entering the canal or prevent waves or fluctuations in the water surface from overflowing the sides

[1] For contraction: 0.1 < K < 0.6; for expansion: 0.3 < K < 0.8.

There are no universally accepted rules for the determination of a freeboard, since wave action or water surface fluctuations in a channel may be created by many uncontrollable causes.

The calculation of the freeboard is based on Q_{max} and the interpretation of the following table containing freeboard in relation to design discharge rate.

Table 25.3: *minimum freeboard for canal and structure*

Discharge rate (m^3/s)	Total freeboard (m)	Lining freeboard (m)
0.01	0.30	0.15
< 0.5	0.40	0.20
0.5 – 1.5	0.50	0.20
1.5 – 5.0	0.60	0.25
5.0 – 10.0	0.75	0.30
10.0 – 15.0	0.85	0.40
> 15.0	1.00	0.50
> 30.0	1.00	0.60

Particularly for higher weirs in rivers with low embankments, it is important to determine how high the embankment must be raised d/s to prevent flooding that would normally not have occurred (backwater curve).

26.DESIGN PRINCIPLES OF GRAVITY WALLS

26.1. DESIGN PRINCIPLES

The four primary concerns for the design of nearly any retaining wall are:

1. That it has an acceptable FoS with respect to overturning.
2. That it has an acceptable FoS with respect to sliding.
3. That the allowable foundation soil bearing capacity is not exceeded (bearing failure).
4. That the stresses within the components (stem and footing) are within code allowable limits to adequately resist imposed vertical and lateral loads.

The retaining wall design calculates the location of the resultant force, from the soil backfill and the weight of the wall[1]. This resultant force needs to act within the middle third of the retaining wall for stability, avoiding tension in the masonry. The active load against the retaining wall combined with the dead weight of the retaining wall and the resultant force is then checked against the FoS. Generally, free-standing gravity retaining walls are economical for retaining low walls, possibly up to 3 m.

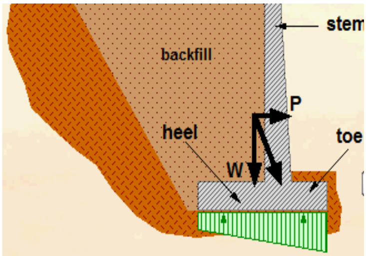


***Figure 26.1**: retaining wall cross-section and acting forces*

26.2. PRESSURE

Retaining wall design considers all the forces acting on the wall, including the effect of the water table. Pressures are shown in the following figure:

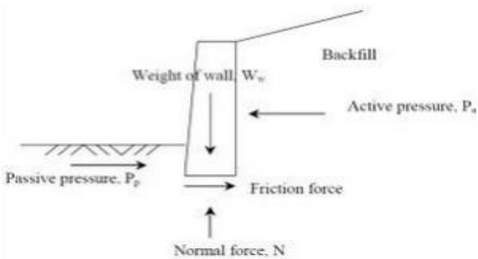


***Figure 26.2**: pressures on gravity wall*

[1] In the User's Manual, the backfill material is typically coarse grained and non cohesive material.

26.3. VERTICAL PRESSURE FORCES

The acting vertical pressure forces are the soil and the wall components weight. The resisting vertical pressure forces are developed in the underlying soil. It varies uniformly between the toe and the heel of the footing and should not exceed the allowable bearing capacity of the soil on which the footing is built.

26.4. LATERAL PRESSURE FORCES

Knowing the properties of the soil behind the wall enables the engineer to determine the lateral pressure distribution that has to be designed for. Lateral earth pressure varies linearly with depth.

The relationship between the vertical earth pressure force (weight of soil) and the lateral earth pressure force is through the appropriate earth pressure coefficient K_a or K_p. The coefficients depend upon the shearing resistance of the soil itself Ø as shown in the following simplified equations with a level back slope of the embankment behind the wall.

$$\boxed{K_a = (1-\sin\phi)/(1+\sin\phi) = \tan^2(45° - \phi/2)} \quad (26.1a)$$

and

$$\boxed{K_p = 1 / K_a} \quad (26.1b)$$

<u>Where</u>:

K_a = coefficient of active pressure force.

K_p = coefficient of passive pressure force.

Ø = failure plane angle of internal soil friction or (critical) angle of repose (degree).

The resultant pressure is located one-third of the height above the base of the wall for triangular pressure diagram.

Because no adhesion or frictional forces (angle δ) are assumed to exist between the soil and the wall, the lateral pressure is taken to act horizontally and parallel to the surface of the backfill[1] (Rankine's theory with wall friction δ = 0 and slope of back fill β = 0).

26.4.1. **Active pressure forces**

The total lateral active earth pressure (thrust) forces P_a on a vertical plane (wall) above the point under consideration (here H_t at bottom from top) develop when the wall is free to move outward and the soil mass stretches sufficiently to mobilize its shear strength. The pressure force acts horizontally

[1] In practise, considerable friction may develop and, as a consequence, the earth pressure is inclined at a certain angle to the normal to the wall. Rankine's assumption results is an overestimation of active earth pressure and underestimation of passive earth pressure. This error is anyway on the safe side.

at a distant $H_t/3$ from the base of the wall[1] and is given by the following equation (friction angle δ does not appear in equations):

$$\boxed{\mathbf{P_a = \sum C * K_a * \gamma * H_t^2}} \quad (26.2)$$

Where:

C = coefficient (=1 for rectangular pressure diagram; = 0.5 for triangular pressure diagram)

P_a = total lateral active (earth and water) pressure force on the wall above the point under consideration (kN/m run of wall).

H_t = height (here total height) of backfill (m).

γ = unit or specific weight of soil or water (kN/m³).

26.4.2. **Passive pressure forces**

The total lateral passive earth pressure forces on the wall P_p occurs if the wall moves towards the soil, then the soil mass is compressed which mobilizes its shear strength and the passive pressure develops[2]. The pressure force acts horizontally at a distant usually $H_t/3$ from the base of the wall under consideration[3] and is given by the following equation (friction angle δ does not appear in equations):

$$\boxed{\mathbf{P_p = \sum C * K_p * \gamma * H_t^2}} \quad (26.3)$$

Where:

C = coefficient (=1 for rectangular pressure diagram; = 0.5 for triangular pressure diagram)

P_p = total lateral active earth pressure above the point under consideration (here at bottom from top) (kN/m run of wall).

γ = unit or specific weight of soil (kN/m²).

H_t = total height of passive earth layer under consideration (m).

26.5. BASIC INSTABILITY MODES

There are three basic instability modes to be checked: sliding, overturning and soil bearing, as shown schematically in the pictures below.

26.6. SLIDING

The backfill and the other applied loads exert a lateral pressure against the wall. This load actually pushes the wall out, so it tends to slide. This sliding force is resisted by the friction between the soil and the footing and by the passive

[1] When the lateral stress on the wall is triangular.

[2] This situation occurs along the wall section below grade on the opposite side of the retained section of fill. In the User's Manual, it is ignored as additional restraint to lateral movement.

[3] When the lateral stress on the wall is triangular.

pressure developed against the soil at the front of the wall. A portion of this passive force is usually ignored to account for the fact that the front soil may have been disturbed during or after the construction. When more sliding resistance is required, a shear key may be provided. This component is very efficient since it works in bearing against the foundation soil. The factor of safety with respect to sliding equals the resisting force divided by the driving force and the minimum value should be 1.25 (refer to table 26.2).

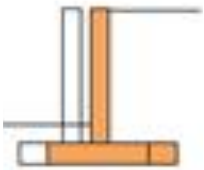


***Figure 26.3a**: sliding*

To carry out this test, the horizontal and the vertical forces must be calculated and the FoS verified (refer to table 26.2). In general, the structure of the User's Manual is very stable against sliding because of its wide base combined with cut-off walls (or key walls), in one monolithic structure.

26.7. OVERTURNING

As a result of the lateral pressure forces on the back of the wall, a retaining structure has the tendency to rotate outward about the toe. The overturning moment from the applied forces must be resisted by an opposite moment produced by the vertical loads, including the wall self weight and the weight of the backfill over the heel, plus any surcharge. The factor of safety with respect to overturning is then defined as the resisting moment divided by the overturning moment, and the minimum value should be 1.50 (refer to table 26.2).

***Figure 26.3b**: overturning*

To prevent the structure from overturning, the sum of the moments of forces tending to resist overturning about 0 (P_p neglected in the User's Manual) must exceed the sum of the moment forces tending to overturn the structure, with a safety factor overturning (refer to table 26.2).

26.7.1. **Overturning moments**

Overturning pressure forces include:

- Earth pressure (horizontal thrusts by soil).
- Hydraulic pressure (horizontal thrusts by water).

26.7.2. **Stabilizing or resistance moments**

Stabilizing or restoring pressure forces include:

- Structure weight.
- Soil weight above the heel.

The water weight is neglected in the resistance moment. Multiplying the forces for their respective arms of leverage, the values of the overturning and stabilizing moments are derived and the FoS checked (refer to table 26.2).

Providing a toe increases both the resistance to overturning and sliding. The design is mainly concerned with keeping the resultant force within the middle third of the base to prevent the base from losing contact with the soil at the heel and tension in the foundation slab.

In general, the structure of the User's Manual is very stable against overturning and sliding because of its wide base (combined with cut-off walls) in one monolithic structure.

26.8. SOIL BEARING

The bearing capacity of the soil must be adequate to support the structure.

***Figure 26.3c**: soil bearing*

Usually the bearing pressure under the footing is trapezoidal with the maximum bearing pressure at the end of the toe. It is important to keep the resultant of the bearing pressure within the kern, so that the minimum pressure at the heel side is not negative, in which case the footing should be re-proportioned.

The load per unit area of the foundation at which shear failure in soil occurs (or the ultimate soil bearing capacity) is given by the following table:

***Table 26.1:** bearing capacity for different types of rocks and soils*

Category	Types of rocks and soils	Bearing value (kN/m^2)
Non cohesive soils	Dense gravel or dense sand and gravel	> 600
	Medium dense gravel, or medium dense sand & gravel	350 to 600
	Coarse river boulders and gravel, loose gravel or loose sand and gravel	350
	River gravel, boulder, sand	320
	Alternate gravel and sand layers with silt content	210
	Fine to medium sand, saturated: loose to medium dense compact	160
Cohesive soils	Very stiff bolder clays & hard clays	300 to 600
	Stiff clays	150 to 300
	Firm clay	75 to 150
	Soft clays and silts	< 75
	Very soft clay	Not applicable

26.9. DRAINAGE

Facilities for drainage from the retained soils are always provided. If water pressure is allowed to accumulate behind a retaining wall, then the resulting force along the back of the wall is increased considerably, as shown in the following table:

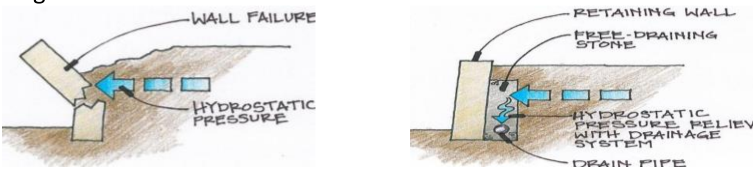


***Figure 26.4**: hydrostatic pressure and drainage*

Poor or inadequate drainage is the cause of nearly all retaining wall failures when the soil is fully saturated and the resulting hydrostatic pressure pushes the wall over. The drainage system is critical to the long term structural stability of any retaining wall.

26.10. SAFETY FACTORS

The calculated and recommended safety factors used in the design of a retaining wall in the User's Manual are given by the following table:

***Table 26.2:** safety factors FoS*

Movement	FoS recommended	FoS calculated
Overturning about the toe	> 1.50 (1.50 to 2.50)	$M_{resistance}$ / $M_{overturning}$
Sliding along base	> 1.25 (1.50)	Friction coefficient*vertical load/active earth pressure force
Bearing capacity failure	2.00 to 3.00	Bearing capacity / max. soil foundation pressure

PART IX:

DID YOU OBSERVE THAT…..?

HYDRAULIC & ENERGY GRADE LINE

1. *In uniform flow, the hydraulic grade line (or surface of water) is parallel to the (total) energy grade line and to the channel bed.*
2. *In uniform flow, total mechanical energy falls gradually as energy is lost through friction. But specific energy remains constant along the channel because there are no changes in depth and velocity.*
3. *The total energy grade line lies over the hydraulic grade line (water surface) by an amount equal to the velocity head. A turbine in the flow decreases the energy grade line (total energy line) and a pump in the line increases the energy grade line (total energy line).*
4. *The total mechanical gradient is the graphical representation of the total head at any section of a conduct.*
5. *In an open channel, the specific energy is the total energy measured with respect to the datum passing through the bottom of the channel or the weir crest of a structure and not with any horizontal datum.*
6. *The water depth in a channel corresponding to the minimum specific energy is known as critical depth.*
7. *The discharge in an open channel corresponding to critical depth is the highest for a given specific energy.*
8. *It is important to draw a clear distinction between total mechanical energy and specific energy. They are linked but they are quite different. The total energy is measured from some fixed datum and its value can only reduce as energy is lost through friction. Specific energy, in contrast, is measured from the bed of a channel and so when the bed level changes the specific energy also changes.*
9. *When there is a change in the bed level of a channel (e.g. when water flows over a weir), there are also changes in the energy components but the total mechanical energy remains the same. It means that specific energy can rise as well as fall depending on what is happening to the channel bed.*
10. *In a fluid flow, a particle may possess elevation energy, kinetic energy, pressure energy and initial energy.*
11. *For conservation of energy, the losses are due to conversion of turbulence to heat, sound and potential energy; it must account for losses if applied over long distances. For conservation of momentum, the losses are due to shear at the boundaries.*
12. *In problems involving the use of conservation of energy, the path taken by the object can be ignored. The only important quantities are the*

object's velocity (which gives its kinetic energy) and height above the reference point (which gives its gravitational potential energy).

BERNOULLI'S EQUATION

13. *For a perfect incompressible liquid flowing in a continuous stream, the total energy of a particle remains the same, while the particle moves from one point to another. This statement is called the Bernoulli's equation. In other words, the main assumptions of Bernoulli's equation are:*
    - *Velocity of energy of liquid particle, across any cross-section is uniform.*
    - *No external force except the gravity acts on the liquid.*
    - *There is no loss of energy of the liquid while flowing.*
14. *In the Bernoulli's energy equation, two types of energy are considered: gravitational potential energy and kinetic energy. The sum of the energies is called the mechanical energy. If energy actually leaks from the system via frictional head loss, the Bernoulli equation will overstate the energy available to the flow and the related predictions of velocity and depth will proportionately be in error.*

FLOW

15. *The flow in a channel is said to be non uniform when the liquid particles at different cross-sections have different velocities.*
16. *The force present in a moving liquid is the inertia force, the viscous force and the gravity force.*
17. *Reducing the hydraulic radius will decrease the flow velocity. This decrease in hydraulic radius can be accomplished by increasing the wetted perimeter in relation to the area. This can be done by widening the channel, flattening the side slopes or widening the bottom. The increases the wetted perimeter without materially increasing the area.*
18. *Our intuition often claims that the larger the flow depth, the larger the specific energy. It is incorrect because the specific energy decreases as the flow depth increases with super-critical channel flow.*
19. *Is the flow super or sub-critical? The answer comes from calculating the normal depth of flow using a formula such as Manning's and then comparing with the critical depth.*

20. *The distinguishing difference between free and submerged flow in a channel constriction is the presence of critical velocity in the vicinity of the constriction (usually a little u/s of the narrowest section of the constriction).*

SLOPE

21. *Two channels of the same slope can be classified differently (one mild and the other steep) if they have different roughness and thus different values of n.*
22. *In general, the slope of the free surface is not equal to the slope of the bottom surface of the flow. However, in some situations, the conditions are met to have both surfaces parallel and the flow is called uniform flow.*
23. *The bottom slope alone is not sufficient to classify a downhill channel as being mild, critical or steep.*

DISCHARGE MEASURING

24. *For steady, fully developed channel flow, the pressure distribution within the fluid is merely hydrostatic, which means that the streamlines are parallel. A hydrostatic pressure means that each water particle pushes on the underlying particle with the same force, resulting in a linear pressure distribution because the lowest particle carries the accumulated weight of the water particles above.*
25. *For curved streamlines, the water pressure is no more hydrostatic.*
26. *The flow measuring devices used in the User's Manual (broad crested weir and gate) are based on the principles associated with rapidly varied flow.*

FROUDE NUMBER

27. *The ratio of the inertia force to the gravity force is called the Froude number.*
28. *If the forces are due to inertia and gravity and frictional resistance plays only a minor role, the design of the channels is made by comparing the Froude number.*
29. *The effect of gravity on the flow is introduced thanks to the Froude number.*

30. *The Froude number is not a function of any temperature dependent properties.*
31. *The higher the Froude number at the entrance of a stilling basin (F_{r1}):*
    a. *The better the hydraulic jump is in energy dissipation.*
    b. *The shorter the stilling basin.*
32. *To increase F_{r1} by converting potential energy into kinetic energy:*
    a. *Decreasing y_1 with an expansion of the cross-section of the glacis.*
    b. *Increasing the incoming velocity V_1 with the appropriate slope of the glacis.*
33. *Isabelle drops a large boulder in a channel, causing gravity water waves to spread out from the source of the disturbance. With sub-critical flow in the channel, the velocity of the stream V_{stream} is less than the celerity of the wave or wave propagation c_{wave} ($V_{stream} < c_{wave}$) so the gravity water waves can move u/s with a velocity of $c_{wave} – V_{stream}$ and affect the water level there as well as the d/s conditions.*
    In super-critical flow, V_{stream} is greater than the wave speed c_{wave} ($V_{stream} > c_{wave}$) so the wave or the ripples cannot travel u/s but will be swept downwards with a velocity of $V_{stream} – c_{wave}$.
    With critical flow, $V_{stream} = C_{wave}$. In other words, the wave celerity sets the boundary between sub-critical and super-critical flow for a given flow.

HYDRAULIC JUMP AND SEQUENT DEPTH

34. *The type of hydraulic jump that develops usually in the design of a structure thanks to the User's Manual is an oscillating jump.*
35. *The formation of a hydraulic jump on a sloping glacis as compared to that on a horizontal floor is more definite and less efficient.*
36. *The sequent depth y_2 depends upon F_{r1} and y_1*
37. *The ideal condition for energy dissipation in the design of a stilling basin is the one when the tailwater rating curve coincides with the jump rating curve at all discharges.*
38. *With increasing jump submergence, the efficiency of energy dissipation is reduced.*
39. *The exit flow d/s of the stilling basin is sub-critical and, hence, it is controlled by the d/s flow conditions i.e. by the tailwater flow conditions. The location of the jump is determined by the u/s and d/s flow conditions and can be at slope break point (toe of glacis), u/s of break point or d/s of break point.*

40. *Deepening of the cistern level excessively is not hydraulically efficient, though it might be required if there is considerable uncertainty as to the stability of the tailwater channel.*
41. *As an alternative to the deepening of the cistern level, a wider basin might be considered, providing a shallower basin.*
42. *Hydraulic jump action in a trapezoidal basin is less complete and less stable than it is in a rectangular basin.*
43. *By changing the slope of the d/s channel and, therefore, changing y_n, the location of the hydraulic jump formation (and it submergence) will change.*
44. *When a hydraulic jump occurs in a channel, conjugate depths have the same pressure-momentum force (the momentum of the flow u/s and that d/s of the jump are equal) but the energy is not equal. Alternate depths however have the same specific energy. Two conjugate depths can never be alternate depths or vice versa.*

PERMEABLE FOUNDATION

45. *The Khosla's theory is applicable to structures founded on permeable material only.*
46. *The safety of a hydraulic structure founded on pervious foundation can be ensured by:*
    a. *Providing sufficient length of its concrete floor.*
    b. *Providing a d/s cut-off of reasonable depth.*
47. *The Khosla's critical G_e is less for more porous soils.*
48. *The exit gradient G_e is independent of the depth of the u/s cut-off wall.*
49. *Seepage endangers the stability of a structure built on permeable foundation because of piping which depends upon the value of the exit gradient G_e.*
50. *The design of the d/s cut-off wall of a structure on permeable foundation depends upon:*
    a. *Scour depth.*
    b. *Exit gradient G_e.*
51. *While designing a structure on permeable foundation, the correction for mutual interference of cut-off walls is usually not applicable on an intermediate cut-off wall if the outer pile goes only just as deep as the intermediate pile and is within a distance of one and half times its own length.*
52. *U/s and d/s cut-off walls are meant to control the uplift pressure and the exit gradient (piping) respectively.*

53. *By increasing the impervious floor of the u/s reach of the structure, the uplift pressure force will decrease below the d/s impervious floor.*
54. *The thickness of the u/s impervious floor of the structure is nominal because the net uplift pressure governs it.*
55. *The term of piping used in connection with the type of hydraulic structures presented in the User's Manual is associated with failure initiated by the sand boiling phenomenon.*

PROTECTIVE WORKS & SCOUR DEPTH

56. *The inverted filter should be such as:*
    a. *To let out the residual seepage.*
    b. *Not to allow the soil particles to escape.*
    c. *Not to get clogged in itself.*
57. *An aggrading river is a silting river.*
58. *The river reach u/s of a newly build cross-structure usually behaves as an aggrading river.*
59. *The scour depth as measured below the highest flood level in a river or channel will be less in a boulder river than in an alluvial river and will be more in fine silt than in core silt.*
60. *Ruptures of the d/s impervious floor of the structure can be due to:*
    a. *Insufficient length of the u/s impervious floor.*
    b. *Insufficient length of the d/s impervious floor.*
    c. *Insufficient depth of the d/s pile.*
    d. *Choking of the d/s inverted filter.*

ORIFICE FLOW AND NUMBER OF GATES OF THE HEAD REGULATOR

61. *For a same water requirement, the number of gates of the head regulator can be increased if necessary by raising the gate sill elevation and vice versa.*
62. *The value of the coefficient of discharge is less than the value of the coefficient of velocity.*
63. *The coefficient of discharge is not a simple constant number like the coefficient of contraction.*
64. *The discharge through a totally submerged orifice is directly proportional to the square root of the difference in elevation of water surface.*

RETAINING WALL

65. *As the wall moves away from the soil backfill, the active condition develops and the lateral pressure against the wall decreases with wall movement until the minimum active earth pressure force is reached.*
66. *As the wall moves towards (into) the soil backfill, the passive condition develops and the lateral pressure against the wall increases with wall movement until the maximum passive earth pressure is reached.*
67. *If y is the depth of water retained by a vertical wall, the height of centre of pressure above the bottom is y/2.*

PART X:

PERSONAL NOTES

LIST OF PRINCIPAL SYMBOLS

A	m^2	Wetted cross-sectional area
b	m	Total solid floor length
B	m	Overall waterway (open water) width between river banks or canal banks (across the flow)
B_{cl}	m	Clear or effective waterway (open water) width at gate section or chamber (across the flow)
$B_{structure}$	m	Overall waterway (open water) width of structure between abutments (across the flow)
C_c	X	Contraction coefficient
C_d	X	Discharge coefficient
C_e	X	Effective discharge coefficient
C_v	X	Velocity coefficient
d	m	Depth of cut-off wall or pile
d/s	X	Downstream
d_1	m	U/s vertical cut-off wall depth
d_2	m	D/s vertical cut-off wall depth
d_{50}	mm	Average particle size or diameter of the channel bed material of which 50 % of the mixture is finer by weight. Sieve size through which 50% of the material passes by weight
D	m	Depth of scour measured from channel bed level
D	m	Depth of cut-off wall or pile, the influence of which has to be determined on the neighboring pile of depth d.
D_{FR}	m	Depth of the application point of the resultant hydrostatic force acting on a gate
E	m	(specific) energy
E_{f1}	m	Estimated specific energy head in super-critical flow range of hydraulic jump as expressed in relation to the canal or channel bottom
E_{f2}	m	Estimated specific energy head in sub-critical flow range of hydraulic jump as expressed in relation to the canal or channel bottom
E_{min}	m	Calculated minimum specific energy for a set discharge rate
E_1	m	Calculated specific energy head per weight of the fluid in super-critical flow range of hydraulic

		jump as expressed in relation to the canal or channel bottom
E_2	m	Calculated specific energy head per weight of the fluid in sub-critical flow range of hydraulic jump as expressed in relation to the canal or channel bottom
f	X	Drowned flow reduction factor
f	X	Lacey silt factor
FoS	X	Factor of safety
F_{r1}	X	Froude number of super-critical reach of hydraulic jump (ratio of inertial forces to gravitational force in a system)
F_{r2}	X	Froude num of sub-critical reach of hydraulic jump (ratio of inertial forces to gravitational force in a system)
g	m/s^2	Gravitational acceleration
G_e	X	Exit gradient
h_{es}	m	End sill height of the hydraulic structure in relation to d/s channel or canal bed elevation
h_j	m	Height of the hydraulic jump
h_0	m	U/s water head in relation to weir sill elevation
h_2	m	D/s water head in relation to weir sill elevation
$H_{d\,max}$	m	Maximum dynamic head (with flow)
$H_{s\,max}$	m	Maximum hydrostatic head (without flow)
H_0	m	Total u/s energy head in relation to the weir sill elevation
H_1	m	Total energy head u/s of jump in relation to the weir sill elevation
H_2		Total d/s energy head in relation to the weir crest
H_L	m	Total energy head loss
HE_0	m	Total u/s energy elevation including kinetic energy (energy grade line)
HE_1	m	Total energy elevation including kinetic energy (energy grade line) at u/s jump formation
HE_2	m	Total energy elevation including kinetic energy (energy grade line) at d/s jump formation
L	m	Flow wise length of crest or protective layer
n	X	Manning's roughness or friction coefficient
N_{sc}	X	Number of side contraction of a weir
q	$m^3/s.m$	Unit discharge rate per m width of structure, stream or channel
Q	m^3/s	Discharge rate in channel
Q_{crop}	m^3/s	Discharge rate corresponding to irrigation water need or water duty

Q_{max}	m^3/s	Flood discharge rate in channel
Q_{min}	m^3/s	Low water flow rate in channel
R	m	Lacey's normal or mean scour depth measured from high flood level
s_r	X	Specific gravity of rock
S	m/m	Channel slope
t	m	Thickness
u/s	X	Upstream
V	m/s	Fluid velocity
V_a	m/s	Flow velocity of approach
$V^2/2g$	m	Kinetic energy head due to flow velocity
V_1	m/s	Velocity of super-critical flow
V_2	m/s	Velocity of sub-critical flow
V_3	m/s	Flow velocity d/s hydraulic structure
w	m	Under-flow gate opening height
w_{crop}	m	Under-flow gate opening height producing, after contraction, a flow satisfying the field water requirements
y	m	Depth of flow
y_{gate}	m	Depth of water from water surface to centre of a closed gate
y_n	m	Normal depth of flow
y_s	m	Water depth with the presence of a structure
y_1	m	Super-critical initial depth of flow
y_2	m	Sub-critical sequent depth of flow
Z	m	Elevation of datum
Z_c	m	Crest elevation to cistern elevation (drop height)
γ	N/m^3	Specific or unit weight
ΔE	m	Mechanical head loss over the structure
Δh	m	Head loss (water depth change) over structure in relation to the weir sill
Φ	%	Hydrostatic pressure force
ϕ_t	%	Residual hydrostatic or dynamic pressure force at toe of glacis

GLOSSARY

Abutments: walls that flank the edge of a weir or other hydraulic structure, and which support the river banks on each side of the weir.
Accretion: process by which particles carried by the flow of water are deposited and accumulate (opposite of erosion).
Afflux: rise in water level.
Aggradation: general or progressive rise of the bed level of a channel by the accumulation of sediments (silting) (opposite of degradation).
Angle of internal soil friction: steepest angle of descent of a granular material relative to the horizontal plane to which a material can be piled without slumping.
Apron: a layer of stone, concrete or other scour protection placed on the channel bed in the vicinity of a hydraulic structure.
Backfill: soil placed behind a wall.
Backwater effects: effects in sub-critical flow that flow conditions in one location have on flow conditions farther upstream (in particular, the water surface elevation u/s of a weir).
Baffle pier: block placed in intermediate position across the stilling basin.
Bank: the edge of a river or stream. Note that left and right refer to the river viewed looking downstream.
Bank protection: works to protect a bank from erosion or undermining by scour.
Boil: concentrated outflow of seepage water, for example through a crack channel or a hole in the sub-soil (sand boil is a type of boil which carries out sand out of the substrate).
Braided river: alluvial river having two or more channels that form a braided pattern and whose size, length and transverse pattern tend to vary considerably in successive floods.
Broad crested weir: weir with a crest section of significant length or thickness measured in the direction of flow.
Centroid: the centre of mass (or gravity) of a geometric object of uniform density.
Channel: natural open watercourse that contains and conveys water.
Chute blocks: blocks placed at the entrance of the stilling basin to form a serrated device.
Control section: control section of a measuring structure is located where critical flow occurs and sub-critical, tranquil, or streaming flow passes into super-critical, rapid, or shooting flow.
Crest (of weir): top part of weir. The level of the crest, its length and its cross-sectional shape determine the discharge (flow) characteristics of the weir.

Critical angle of repose: steepest angle of descent of a granular material relative to the horizontal plane to which a material can be piled without slumping.

Critical depth: depth representing the minimum specific energy (occurs when the Froude number equals 1.0).

Cross-regulator or weir: an artificial obstruction or constriction in any watercourse that results in increased water surface level upstream or used to measure the discharge.

Cut-off wall: barriers constructed vertically either of reinforced concrete, masonry or of steel sheet pile, and provided at the bottom of the structure to protect it by extending the line of seepage against scours and possible piping due to excessive exit gradients of the seepage flow below the foundations.

Degradation: the processes of progressive lowering or drop in the bed of a channel by erosion (scouring) (opposite of aggradation).

Depth of flow: vertical distance from the bed of a channel to the water surface.

Design discharge rate: peak flow at a specific location defined by an appropriate return period to be used for design purposes.

Discharge rate: flow rate expressed in volume per unit time (typically m^3/s). In this Manual, the word "flow" is used to mean flow rate or discharge.

Discharge intensity: discharge per unit length of weir or stream (see also unit discharge).

Drowned: in the context of weir hydraulics, a weir is said to be drowned (or drowned out) when the downstream water surface elevation rises to the point where it begins to affect flow over the weir.

End sill: vertical, stepped, sloped, solid or dentated wall constructed at the downstream end of the stilling basin.

Exit gradient: hydraulic gradient in the sub-soil at the location of the exit point.

Energy grade line or total energy line: profile of the free flow surface flowing along the structure or channel specifying the total energy head available at any point of the structure or channel. The graphical representation of the elevation of the line at any point is the sum of the datum head, the flow depth and the velocity head. In uniform flow (Manning's equation), the energy grade line (total energy line) is parallel to the hydraulic grade line (water surface).

Erosion: process by which material forming the bed or banks of channel is removed by the action of flowing water or waves (opposite of accretion).

Filter: granular or fabric material placed between the erosion protection revetment (riprap, blocks etc.) and the underlying soil surface to prevent soil movement into and through the revetment. Riprap and concrete block layer should have a filter placed under it in all cases.

Flood bank: embankment, usually earthen, built to prevent or control the extent of flooding.
Flow: flow rate or discharge.
Freeboard: height of the top of a bank, flood bank, or structure above the level of the water surface.
Froude number: dimensionless parameter representing the ratio of the velocity wave relative to inertia and gravity forces in a fluid, taking the value of unity for critical flow. It is an indication of the energy in a channel at a certain point.
Geotextile: permeable synthetic fabric filter that prevents migration of the fine soil particles through voids in the structure, that distributes the weight of the armor units to provide more uniform settlement and that permits relief of hydrostatic pressures with the soils.
Glacis: downstream sloping face of a weir, between the weir crest and the stilling basin.
Gradation: size distribution of a particular layer (riprap etc.) or aggregate. Material that is well graded has a uniform distribution of sizes, within a given minimum and maximum range.
Grade: inclination or slope of a stream channel, energy etc. usually expressed in terms of the ratio or percentage of number of units of vertical rise or fall per unit of horizontal distance.
Head (of water)[1]: the height of water surface elevation above a datum (such as the weir crest).
Head loss: the drop in water surface elevation across a weir or other hydraulic structure.
Heel: portion of the footing extending behind the wall (under the retained soil).
Hydraulic depth: average depth for a river cross-section.
Hydraulic grade line: surface or profile of water flowing in an open channel specifying the sum of the static pressure and the elevation head Z at any point of the structure or channel. The difference between the energy grade line and the hydraulic grade line is equal to the dynamic (velocity) head $v^2/2g$. In uniform flow, the hydraulic grade line is parallel to the water surface and is also represented by the bed of channel.
Hydraulic gradient: slope of the hydraulic grade line (quotient of the difference in head between the two points and the distance between those points).
Hydraulic head: difference in head between two points.
Hydraulic jump: abrupt rise in water surface elevation when flow changes from a super-critical to a sub-critical state, with associated dissipation of energy.

[1] There are more technically precise definitions of head, making the distinction between static head, velocity head and total head.

Invert level: level of the lowest point of a cross-section of a natural or artificial channel or a structure.

Jet: water exiting an orifice.

Kinetic energy: energy that a system possesses as a result of its motion relative to a reference frame.

Launching apron: apron of riprap or other material that subsides as scour occurs to prevent scour undermining a structure.

Modular flow: condition in which flow is able to discharge freely over a weir, resulting in a unique relationship between flow rate and referenced upstream water surface elevation (Upstream head).

Modular limit: the maximum submergence ratio for which a weir will operate with critical depth flow in the throat reach.

Nappe: over falling sheet of water passing over a weir crest and plunging into the stilling basin. Term normally only applied where the jet is not in contact with the weir structure (i.e. there is an air gap between the underside of the nappe and the downstream face of the weir).

Normal depth: water depth corresponding to the minimum energy level in conditions of normal flow.

Non modular flow: condition in which flow is not able to discharge freely over a weir, with the downstream water surface elevation influencing the upstream level (i.e. drowned flow).

Pitching: regularly sized and shape stones or concrete blocks placed in an ordered fashion as protection against erosion.

Profile: flow profile represents the surface curve of the flow

Rating curve: a plot of water head against flow rate for a channel, weir or other hydraulic structure (also called a stage-discharge curve).

Regulator: hydraulic structure for controlling water heads or division of flow.

Regime channel: state in which a channel or river formed in erodible material which has reached a stage of virtual equilibrium (has adjusted its gradient and cross-section to an equilibrium condition) with no long-term aggradation or degradation (accretion or erosion), but which may be subject to meandering.

Retaining wall: any constructed wall that restrains soil or other material at locations having an abrupt change in elevation.

Riprap: broken rock or cobbles placed on a random fashion as protection against the action of water.

Roller: the large-scale turbulence region of a stilling basin.

Scour (local): scour that results directly from the impact of individual structural elements (pier, abutment etc.) on the flow and occurs immediately in the vicinity of those elements.

Scour (natural): erosion resulting from the shear forces associated with flowing water or wave action.

Sediment: erodible material forming bed or banks of channel, which may be eroded or deposited depending on the prevailing flow conditions.

Short crested weir: streamline curvature above the weir crest has a significant influence on the head-discharge relationship of the structure.

Side weir: weir installed in a channel to divert part of the approach flow into a separate spill channel.

Sill: top of an embedded structural member on which a gate rests when in closed position.

Siltation: the deposition of sediment.

Specific energy: for a given cross-section, total mechanic energy per unit weight of water as expressed in relation to the channel bottom; it is the sum of the water depth and the velocity head, provided the streamlines are straight and parallel.

Seepage: water which flows through the sub-soil, as a consequence of the upstream existing hydraulic head.

Stilling basin: an energy dissipater comprising a basin in which a hydraulic jump occurs.

Steady flow: flow with streamlines parallel and hydraulic characteristics remaining constant for the time interval under consideration.

Streamline: path followed by a molecule of water.

Sub-critical flow: the flow in a channel at less than critical velocity.

Super-critical flow: the flow in a channel at greater than critical velocity.

Tailwater level: the water surface elevation downstream of a hydraulic structure.

Toe: portion of footing which extends in front of the front face of the stem (away from the retained earth).

Uniform flow: flow of water in a channel in which the depth and the velocity remain constant along the channel.

Water level: the elevation of the water surface.

Wing wall: a wall on a weir or other hydraulic structure that ties the structure into the river bank.

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