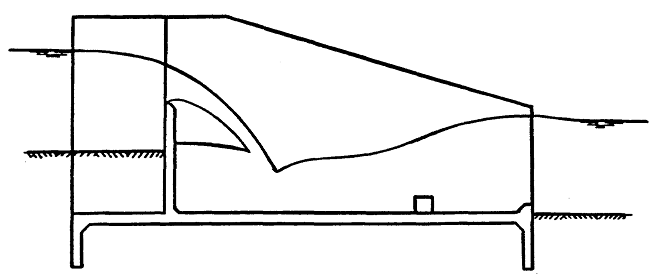
HYDRAULIC STRUCTURES



BY

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Hydraulic Structures

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ABOUT THE AUTHOR

Professor Smith obtained his B.Sc. in Civil Engineering from University of Alberta in 1948, and later obtained his M.Sc. in Hydraulic Engineering from the University of Saskatchewan. In 1949 he joined PFRA in Regina, becoming senior hydraulic engineer in 1953. In this capacity he was responsible for the design and model testing of many structures for the Gardiner Dam and the South Saskatchewan River Project. In 1959 he accepted an appointment as Associate Professor with the University of Saskatchewan. He was promoted to Professor in 1968 and was head of the Department of Civil Engineering from 1974 to 1984. Over a period of 35 years he has served as a consultant or specialist advisor on some 80 projects for various government and private agencies. Professor Smith has authored 85 technical reports and has 60 technical papers published on hydraulic engineering topics in various national and international journals and conference proceedings. He is a double winner of the CSCE Keefer Medal in 1966 for his paper "Head Losses at Open Channel Inlets", and in 1992, with Kells, for their paper "Reduction of Cavitation on Spillways by Induced Air Entrainment". In 1986, Professor Smith was awarded the CSCE Camille Dagenais Award for outstanding contributions to the development and practice of hydrotechnical engineering in Canada. In 1991 he received the APES Engineering Achievement Award and Silver Medal for excellence in hydrotechnical engineering, and in 1993, the Saskatoon Engineering Society Engineer of the Year Award.

Professor Smith has been an executive member of the CSCE Hydrotechnical Division for 15 years. During this time he was editor of the Hydrotechnical Division Newsletter from 1974-78, chairman of Hydrotechnical Division Student Awards Committee from 1980-82, and chairman of the Hydrotechnical Division CSCE 1982-83. In 1983 he was elected Senior Vice-President of the CSCE, becoming President 1984-85. Professor Smith was a member of the Professional Engineers Board of Examiners for 10 years and a Councillor of the Association of Professional Engineers of Saskatchewan for 3 years. He is a Fellow of the Engineering Institute of Canada, Fellow of the Canadian Society for Civil Engineering, Fellow of the American Society of Civil Engineers, an Honourary Fellow of the Pakistan Institution of Engineers, Member of the International Association for Hydraulic Research, Member of the Canadian Water Resources Association, Member of the Western Canada Water and Wastewater Association, Member of the Canadian Dam Safety Association, Delegate to the Canadian National Committee on Large Dams and Delegate to the Universities Council on Water Resources.

As Professor Emeritus of the University of Saskatchewan, Professor Smith has continued his work in research, writing, consulting and speaking engagements.

PREFACE

A hydraulic structure may be defined as any structure which is designed to handle water in any way. This includes the retention, conveyance, control, regulation and dissipation of the energy of water. Such water handling structures are required in many fields of civil engineering, the principal ones being water supply and conservation, hydroelectric power, irrigation and drainage, navigation, flood control, fish and wildlife services and certain aspects of highway engineering.

In order to insure that the function intended for a structure will actually be achieved, a hydraulic design must be carried out. Various equations, based on continuity, energy, and momentum principles, may be used to calculate the most suitable length, width, shape, elevation and orientation of the structure. The application of these basic principles to the practical problem of the design of hydraulic structures is called hydraulic design.

Although Chapter 1 and Chapter 2 deal specifically with dams and spillways, the reader is advised that these two chapters are in a sense, prerequisite to the remaining chapters. The methods of load determination and stability analysis used in Chapter 1 apply to many other structures or structure components, such as pipeline anchor blocks, retaining walls, spillway sidewalls, diversion dams, drop structures and others. The gravity dam was selected simply as a vehicle to demonstrate the method. Similarly, the methods of discharge calculation, velocity determination and stilling basin design, discussed for spillways in Chapter 2, apply as well to many of the hydraulic structures discussed in latter chapters.

Design details for all possible hydraulic structures are beyond the intended scope for the book. The object has been to present general information on the form and function of many types of structures and to give detailed theory and design procedures on some of the more common ones. The application of principles is intended to be sufficiently rigorous that a student of the subject will gain a working knowledge of the basic procedures used in hydraulic design. While, of necessity, certain design procedures are empirical, this book is not concerned with the "how" aspect of hydraulic design alone. Equal emphasis is placed on the "why" aspect, since it is only through this approach that the engineer can properly assess the merits of a proposed design for a specific application.

Many of the design procedures are readliy adaptable to solution by computer programming. Such procedures may be advantageous where a large number of variations in loading conditions or structure dimensions are to be investigated. For example, there are many possible combinations of basin width and floor evaluation which will work equally well for a hydraulic jump stilling basin. In the case of irrigation structures or drainage structures, programs may be written to output structure dimensions for various combinations of head and discharge. The Saskatchewan Department of Highways and Transportation has such a program for sizing drainage culverts, based on the design criteria covered in Chapter 10. However, the author is a strong advocate of the need to have a thorough understanding of the philosophy and basic principles of hydraulic structure design before programming is attempted. This text is intended to satisfy that need.

The author has chosen to use SI units exclusively, rather than dual units, in order to avoid the frustrations of constant conversions which add little to the learning process. Of course the engineer will be confronted by other systems of units everytime he or she consults a reference or text printed prior to about 1975. These references are and shall remain an invaluable source of engineering information, and therefore the engineer must maintain some working knowledge of other systems of units for some time into the future. This is not expected to be a very difficult task, as engineers traditionally have been required to understand and use different systems. In a few decades, however, most other systems should become of historic interest only.

A section on the practice, usage and conversion factors for SI units has been added as Appendix 1. Material for Appendix 1 has been extracted largely from the Universities Council on Water Resources pamphlet on use of SI units in water resources engineering. This pamphlet was originally prepared by P.C. Kingeman, Director, Water Resources Research Institute, Oregon State University. The University of Saskatchewan is an Affiliate of the Universities Council on Water Resources, and the author is the University delegate.

The current edition is an updated and expanded version of the 1992 edition. The author gratefully acknowledges the constructive criticism by Professor J.A. Kells, Department of Civil Engineering, University of Saskatchewan, who carefully reviewed the 1992 edition. His many suggestions for improving the clarity of the text were incorporated in the present edition.

C.D. Smith

Saskatoon, Canada
July, 1995

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CHAPTER I

STORAGE DAMS

A. INTRODUCTION

1. Purposes of Storage Dams

Storage dams are dams built to retain a significant volume of water in storage in a reservoir. As such, these are relatively high dams. They are to be contrasted with diversion dams which are generally low in height and may have little or no usable storage.

Storage dams may be built for flood control, power, irrigation, navigation, municipal and industrial water supply, conservation of fish and wildlife, or recreational benefit. Generally, however, a single dam will serve several of these purposes at once. These are called multi-purpose dams. The storage feature of the dam and reservoir results in a reduction in maximum flow, for flood control benefit, and an increase in minimum flow, for the benefit of power, irrigation, and navigation in the downstream channel.

In addition to providing storage, a storage dam will also create head. This is particularly important in a power dam because the power potential varies directly as the head available. Head is also important in irrigation dams because it is frequently necessary to pump water from the reservoir to a higher level in order to serve the irrigable areas. Head is not important in flood control dams, navigation dams, or dams for recreational benefit, except as it affects the storage volume or surface areas of the reservoir.

2. Types of Dam

Dams may be classified according to the purpose, as in Section 1, or they may be classified according to the materials of construction, as discussed below.

i) Concrete dams - Concrete dams may be either solid concrete or hollow concrete. The hollow concrete dams are constructed of thinner sections and require steel reinforcement. Solid concrete dams may be further subdivided into gravity dams or arch dams. The solid dams do not require steel reinforcing. Stability for the gravity dam is obtained by virtue of its own weight. Stability for the arch dam is obtained by arch action, which transmits the load in compression into the abutments. A concrete gravity and concrete arch dam are shown in Figures 1 and 2.

ii) Earth fill dams - These are dams constructed of natural earth materials with little processing. The materials used vary according to availability, and may include rock, gravel, sand, clay, or combinations of these. Earth dams may be further subdivided according to the method of construction rolled fill or hydraulic fill. The rolled earth fill is the most common.

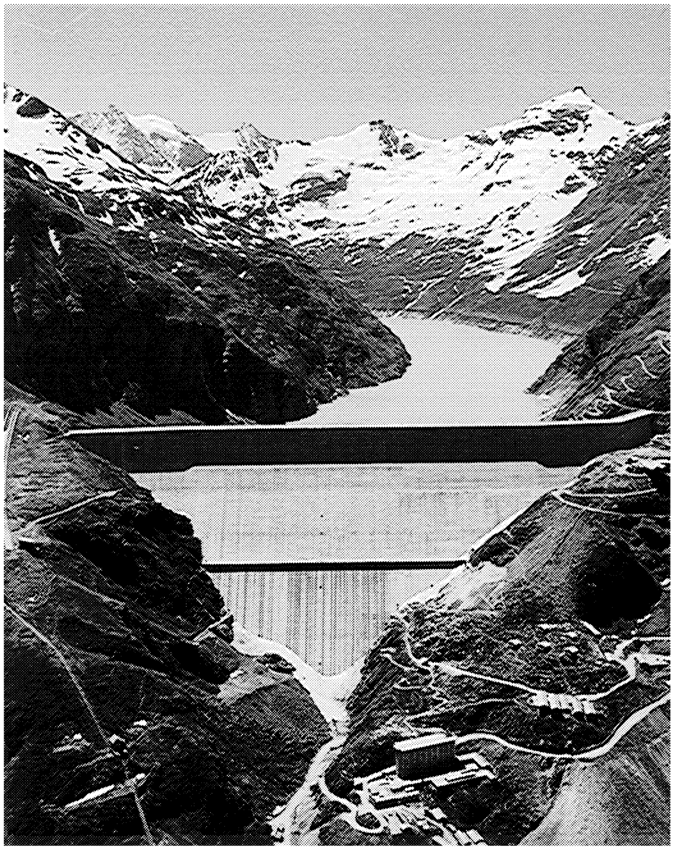

Figure 1. ***Concrete Gravity Dam Grand Dixence Switzerland***

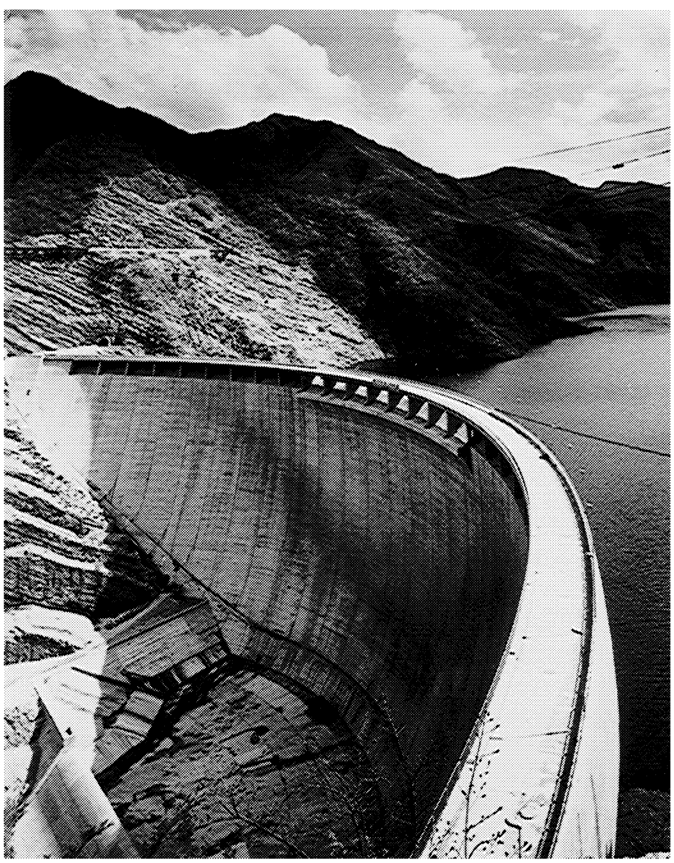

Figure 2. ***Arch Dam Ridrocoli, Italy***

iii) Rock fill dams - Rock fill dams are classified separately from earth fill because of certain basic differences in the design, construction, and site conditions.

iv) Timber and steel dams - Timber or steel may be used for low height dams or temporary dams. Generally they are less durable than the other types.

The various types of dam are discussed in greater detail in later sections of this chapter. However, since the loads to be resisted are the same for all types, and since the basic principles of stability analysis are also similar, a detailed design procedure is given for only one type - the solid concrete gravity dam.

It should be pointed out that there is no particular relationship between the type of dam and the purpose to be served. Concrete gravity, concrete arch, earth fill, and rock fill dams have all been built to heights approaching 300 m. An earth dam can be built for power, or a concrete dam for irrigation, or vice versa. The type of dam used in a given situation depends on factors other than the purpose to be served, as explained in Section 3.

3. Considerations in Choice of Site

There are five principal factors which determine the choice of site for a dam, as follows:

i) Water supply - There must be ample stream flow at the proposed site to fulfill the function intended, otherwise the dam would serve no useful purpose. The stream flow may be assessed from data in the hydrometric records and from a hydrological analysis.

ii) Topography of the site - Topography is one of the most important factors affecting the choice of damsite. It determines the available head, the available storage, and the volume of materials required for the dam, each of which is related to the economics of the project. In addition, river alignment and bank topography largely determine the layout for appurtenant structures (e.g., powerhouse, spillway, outlet works). Topographical maps are required for this assessment.

iii) Foundations - Foundation conditions are assessed from a subsurface exploration and geological survey. The type of dam which must be built is closely related to the foundation conditions. The bearing strength, shear strength, and impermeability of the foundation determines its suitability for a given type of dam.

iv) Availability of materials - The quantity, quality, and distribution of construction materials is an important economic factor in locating the dam site and determining the type of dam to be constructed.

v) Land costs - Large areas may be inundated by the creation of the reservoir, resulting in considerable loss of property previously used for industry or agriculture. In addition, it may be necessary to relocate towns, railroads, or highways. These costs must be charged to the project and should be considered when making a benefit - cost study on the site location of the dam.

B. FORCES ACTING ON GRAVITY DAMS

4. Nature of Forces

The principal forces which act on a gravity dam are those due to water, ice, silt, earthquake, weight, and foundation reaction. The forces due to water include those caused by waves and uplift pressures, as well as the obvious forces due to hydrostatic pressure. Weight is the principal stabilizing force, and the foundation reaction is simply the force which is equal and opposite the resultant of all the applied loads.

Since each metre of gravity dam must be stable in its own right, it is customary to calculate forces and stability on a unit width basis rather than for the gross section. Such an analysis is normally carried out for the maximum section first. Additional analysis may be required at other sections or in the vicinity of the abutments where the height is less than the maximum section.

5. External Water

The forces due to headwater and tailwater acting on the upstream and downstream face of the dam are shown in Figure 3. The water force F_w for a hydrostatic pressure distribution on a unit width of dam is given by

[1] $$F_w = \gamma h_1{}^2/2$$

in which h_1 is the reservoir depth in metres above the base of the dam, and γ is the specific weight of water at 9.81 kN/m^3. This force acts at $h_1/3$ from the base of the dam. The force due to tailwater acts normal to the inclined downstream face of the dam. However, it is more convenient to evaluate this force in terms of its horizontal and vertical components because all other forces used in the analysis are also horizontal or vertical. The horizontal component of the tailwater force, which is independent of the slope of the downstream face, is simply $\gamma h_2{}^2/2$ The vertical component is equal to the weight of the volume of water directly above the inclined surface, and acts through the centre of gravity of that volume.

6. Silt

Sediment deposits invariably occur in any reservoir constructed on an alluvial stream. These deposits, consisting of sands, silts, and clays, may eventually build up against the face of the dam and exert an additional horizontal load, called silt force, F_s. The magnitude of the silt force is given by

[2] $$F_s = C(\gamma' - \gamma)\, h_s{}^2/2$$

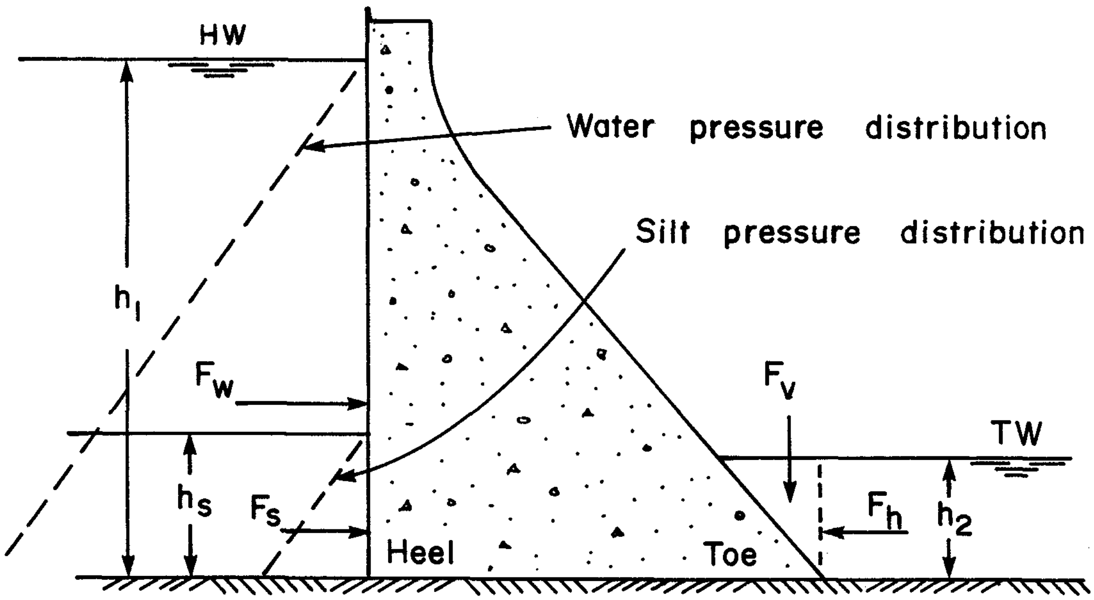

Figure 3. External Water and Silt Forces

in which h_s is the depth of the silt deposit, γ' is the saturated weight of the silt, and C is the coefficient of lateral pressure.

The term $(\gamma' - \gamma)$ represents the submerged weight of the silt, and is responsible for the vertical pressure in the silt deposit. The coefficient C represents the ratio of the lateral pressure to the vertical pressure and is normally about 1/3. The product $C(\gamma' - \gamma)$ is sometimes referred to as the equivalent fluid pressure, and has a magnitude from 2.5 to 3.5 kN/m^2/m of height. It is assumed that the silt pressure increases linearly with depth. This is not strictly correct because γ' usually increases slightly with depth due to consolidation of the deposit as the silt depth increases, but this refinement is unnecessary. The position of the force F_s may be assumed at 2/3 of the silt depth below the top of the silt.

It is important to note that the water force on the upstream face of the dam is not reduced because of the presence of the silt deposit, so the silt force from [2] must be superimposed on the full water force. The reason for this is that the permeability coefficient for the silt deposit normally will be several hundred times greater than the permeability coefficient for the rock foundation, and hence the pressure loss due to seepage through the silt will be negligible.

The most uncertain aspect of the calculation for silt force is the design silt depth h_s. The bed load transported by the stream, consisting of the coarser sediments, will normally deposit at the head end of the reservoir in the form of a delta or multiple deltas. The suspended load is transported further into the reservoir before it settles out, and in fact it may proceed to the face of the dam as a density current. On a low height dam, such as an overflow diversion dam,the bed load deposit may very well reach the crest of the overflow section within the life of the structure. On a high storage dam with a large reservoir, the bed load may never progress as far as the dam. The depth of silt deposit at the face of the dam will depend on the elapsed time since completion of the structure, the concentration of sediment in the stream, the river hydrograph, and size and shape of the reservoir. It is also possible to control the upper limit of the silt level by means of low level sluice outlets. Fortunately, the silt force is small compared to the water force so an exact determination of the ultimate silt level is not critical in design.

7. Ice

In northern latitudes ice load on a vertical face dam can be a very important consideration. It is often more important than the silt load because the ice load, acting near the top of the dam, has a much longer moment arm from the base.

Contrary to the layman's conception of ice pressure, the greatest ice thrust on a dam occurs during expansion of the ice sheet upon thawing, not freezing. Although the volume change is large during the fusion process as water is transformed into ice at 0°C, this process does not exert a significant thrust on the dam because the liquid is not confined. Expansion during ice formation may occur freely in the vertical direction. Once the ice has formed, it behaves like any other solid, contracting as the temperature drops and expanding as the temperature rises. In fact, the coefficient of linear expansion for ice is about five times greater than for steel. During a severe cold spell the ice sheet will shrink and cracks will develop. These cracks may become filled with snow or water, which subsequently freezes, thus forming a solid ice cover at low temperature. Upon temperature rise, the ice sheet expands and thrust is exerted around the confining perimeter.

Studies by Rose in 1946 related the ice thrust to the thickness of the ice sheet, the rate of temperature rise, and the degree of edge restraint around the perimeter of the sheet. The effect of these variables on ice is shown on Figure 4. Naturally a thicker ice sheet is capable of exerting more thrust. The thrust depends upon the rate of temperature rise and not the amount, because ice will flow under pressure, and if the sheet expands slowly, the ice thrust at the edges is relieved. The ice temperature of the underside of the sheet is always 0°C, but on the surface the ice temperature closely follows the air temperature unless there is a snow cover. A snow cover will minimize ice thrust as snow has a remarkable insulating effect. On the other hand, solar radiation on an exposed ice sheet will increase the rate of expansion of the ice sheet. The degree of lateral restraint refers to the condition of the sides of the reservoir at the F.S.L. (full supply level). In mountainous areas the sides may be steep banks of solid rock which effectively prevents any lateral expansion of the ice sheet. This results in an increase in ice thrust on the dam. In the plains areas the sides of the reservoir may be gently sloping earth banks. Upon expansion the ice sheet can ride up the banks of the reservoir, greatly relieving the thrust exerted on the dam. If the dam itself has a sloping face, the ice thrust may be eliminated. In fact, it is customary to neglect the ice load on earth dams for this reason. Figure 4 applies only to vertical face dams.

It is important to note that the ice thrust given by Figure 4 is independent of the size of the ice sheet. Intuitively this does not seem reasonable at first, but it may be explained by drawing an analogy to temperature stresses in a steel rod. If linear expansion of the rod is prevented, the compressive stress in the rod due to temperature rise is independent of its length. In the same way the stress due to temperature rise in the ice sheet is independent of the size of the reservoir.

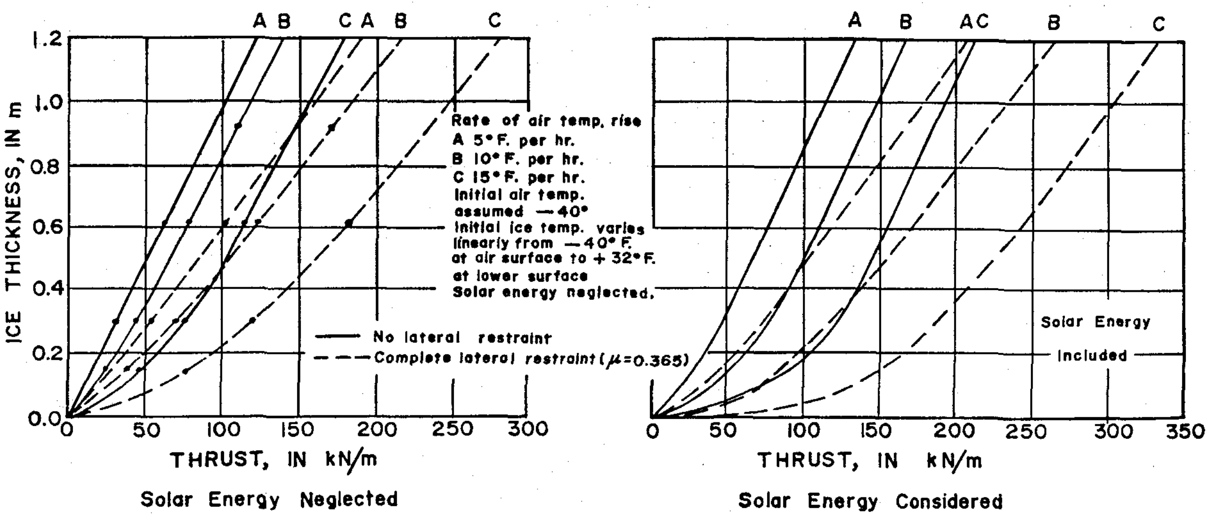

Figure 4. Chart for Ice Thrust on Vertical Face Dams

The compressive stress in an expanding ice sheet is a maximum at the surface, where the temperature change is greatest. At the dam, or any restraining boundary, there is usually a local redistribution of stress so that the ice pressure becomes more uniformly distributed. However, for the design purposes, it is customary to apply the ice thrust P_i at the surface of the ice sheet.

It is evident that ice thickness is an important variable in determining ice thrust. Ice thickness varies from year to year, depending on the freezing index. It is satisfactory for design to base the thickness on the 'normal freezing index'. The normal freezing index for Canada, in Fahrenheit degree-days, is given in Figure 5. The corresponding ice thickness may be calculated from

[3] $$t = 0.026\,\alpha\,\sqrt{S}$$

in which t is the ice thickness in m, S is the freezing index from Figure 5, and α is a location coefficient. Values for α are given in Table 1.

TABLE 1
LOCATION COEFFICIENT

Value	Description
0.8	Open lake or reservoir, windy, no snow cover
0.5. - 0.7	Small lake or reservoir with snow cover
0.4 - 0.5	Average river with snow cover
0.2 - 0.4	Small sheltered river with snow cover

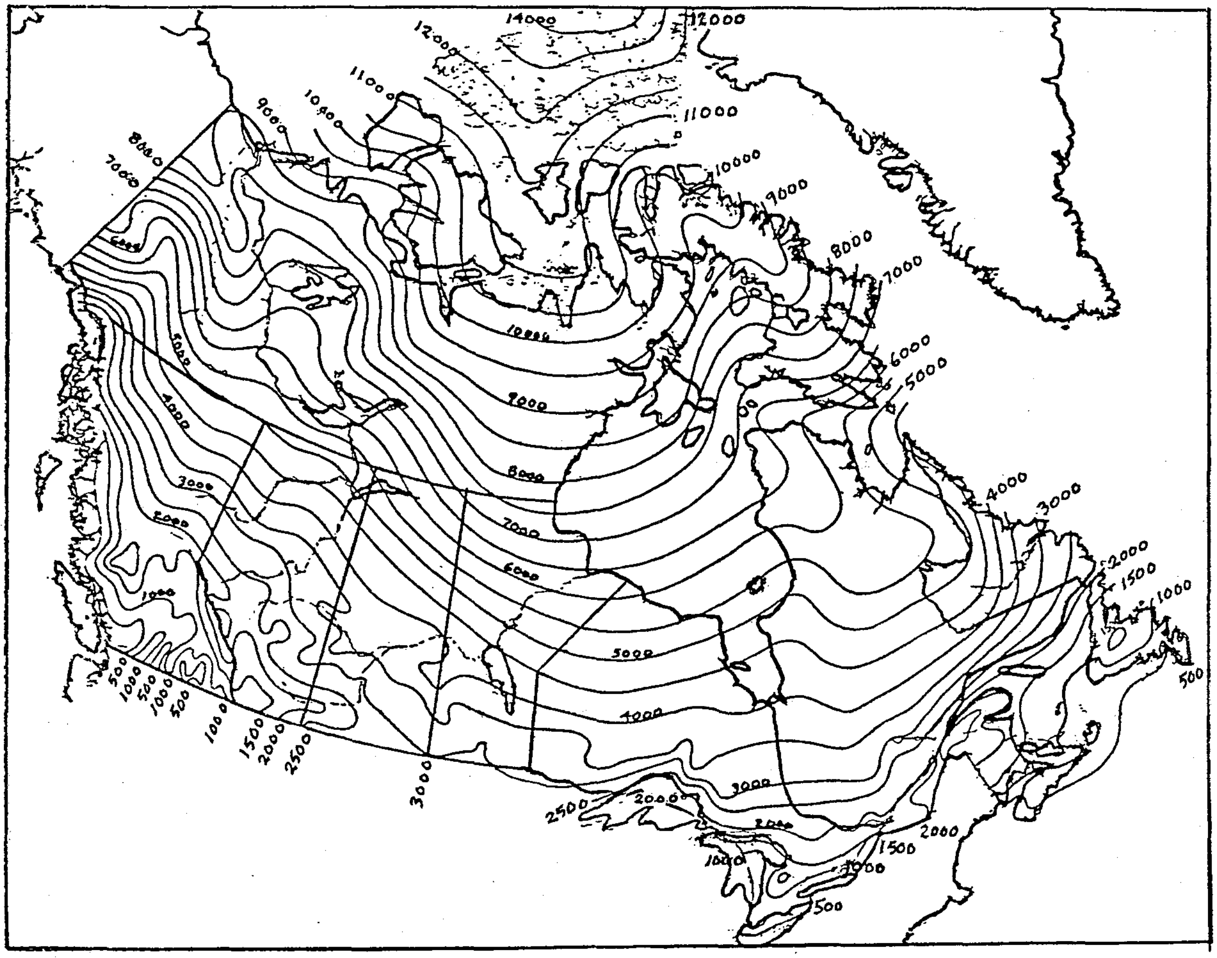

Figure 5. Normal Freezing Index Fahrenheit Degree-Days From Boyd, 1976, based on period 1931-60.

8. Uplift

Uplift is the force produced by water under pressure in the pores of the dam and foundation. This is sometimes referred to as the internal water pressure in order to distinguish between it and the external hydrostatic pressure acting on the faces of the dam. It is one of the most misunderstood but important forces acting on the structure. Despite the low permeability of the concrete and rock, it must be recognized that, given time, the dam and foundation may become saturated and a hydraulic gradient will be established. Considering a section of the dam as a free body, as shown on Figure 6, the base a-a indicated may be any horizontal section through the dam, or indeed at the base of the dam itself. It can be assumed that the pressure drop due to seepage resistance from headwater to tailwater is linearly distributed, and therefore a trapezoidal pressure distribution diagram results, decreasing from γh_1 at the heel to γh_2 at the toe.

It is further assumed that the pressure can act on the full area of the base. Actually this is not physically possible because a small percentage of the base area will be occupied by the points of contact between the solid particles above and below a-a.

However, Leliavsky has shown that the contact area is only 9% for laboratory specimens of concrete, and is apt to be less at horizontal joints in the structure. For soil with 40% voids the contact area is only about 2%, which is why the full water load is superimposed on the silt load when calculating the combined load on the face of the dam below the silt level. Neglect of the contact area will give a value for the uplift force which is slightly high. For the conditions shown in Figure 6 the resultant uplift force U, in kN/m, is

[4] $$U = \gamma(h_1 + h_2)\, b/2$$

in which b is the base width in metres. The position of this uplift force, measured from the heel, may be found by combining the resultant for the rectangular portion and triangular portion of the uplift diagram, from which

[5] $$x = b(2h_2 + h_1)/[3(h_2 + h_1)]$$

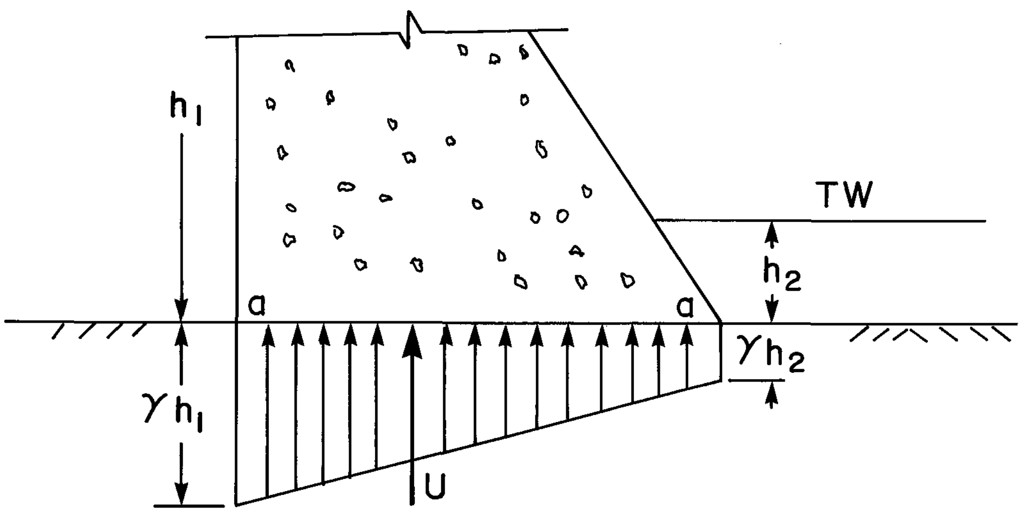

Figure 6. Uplift Pressure

The uplift force adds to the overturning moment on the dam, and at the same time reduces the effective weight, which lowers the sliding resistance. The uplift force according to [4] will be so large that it will significantly affect the economics of the design. Accordingly, measures are usually taken to reduce the uplift pressures on the base. This is accomplished by a cutoff and drainage system, as shown in Figure 7.

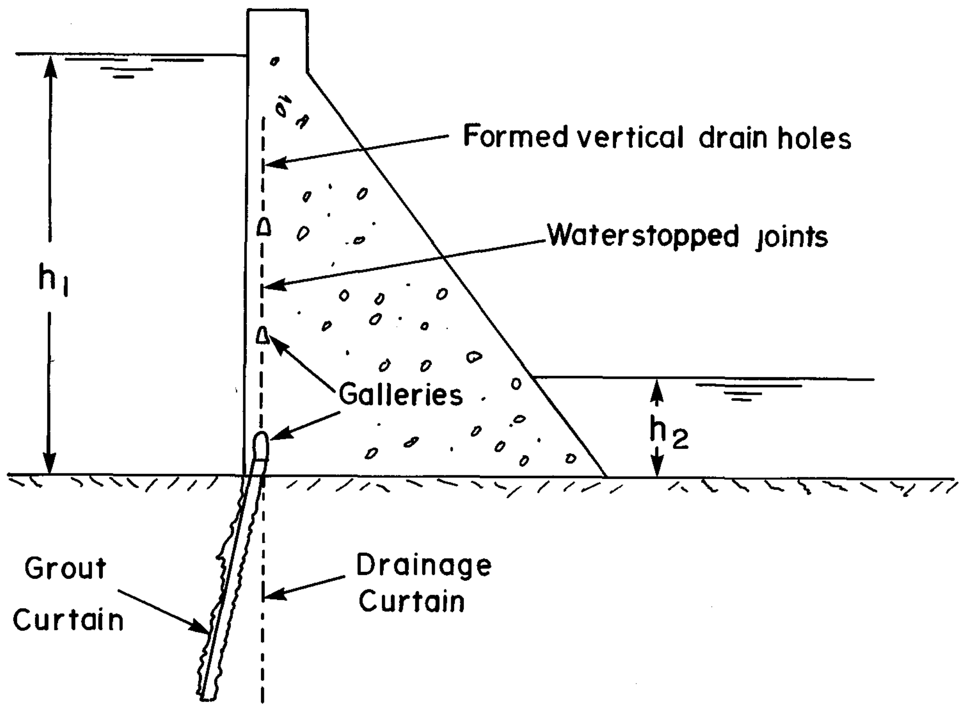

Figure 7. Grout and Drainage Curtains

A grout curtain is most common for the cutoff for a concrete gravity dam on rock. This is constructed by drilling a line of closely spaced holes about 4 to 6 cm in diameter into the foundation near the upstream toe of the dam. Cement grout is pumped under pressure into the holes in order to seal the cracks and fissures in the rock. Grouting pressures vary from 350 to 1700 kPa. The hole spacing and depth depends upon the height of the dam and the condition of the rock. A fairly wide hole spacing may be used for initial grouting. If necessary, more holes are drilled and grouted between the original holes. A minimum spacing of 1 metre may be needed to get a continuous seal in badly fractured rock. Usually a thin grout, one part cement to five parts water, is used at the start of pumping to get good penetration. The grout is thickened to a 1:1 mix as grouting proceeds. The grout curtain may be constructed before any concrete for the dam is placed, or it may be constructed later by drilling holes from a gallery located near the base of the dam at the heel. Should experience show after the reservoir is filled that the cutoff is not sufficiently tight, additional holes can be drilled and grouted. Usually the grout holes are drilled at an angle inclined upstream so that the resultant water force due to differential head across the grout curtain will have a downward component.

A drainage curtain is usually used in conjunction with a grout curtain. The drainage curtain consists of a row of holes, 5 cm or more in diameter and perhaps 3 m on centres, which are drilled from the gallery into the rock just downstream from the grout curtain. The purpose of the drainage curtain is to intercept any seepage which may escape past the grout curtain. This seepage is collected in a gutter in the gallery and flows away by gravity or is pumped away.

Concrete dams are normally constructed in successive lifts, rather than a continuous pour. This simplifies the problem of dissipating the heat associated with hydration of the cement, and allows for initial shrinkage of the concrete. In any case it would not be physically possible to schedule a continuous pour over the entire construction face. Accordingly, there are horizontal and vertical joints at the bottom and sides of each pour. As seepage is more likely to occur through joints, these joints are usually waterstopped with a metal (copper) waterstop near the upstream face of the dam. In a sense this is an extension of the cutoff over the upstream face. Similarly, formed vertical drain holes located downstream from each waterstop may be considered as an extention of the drainage curtain.

The combined effect of the grout and drainage curtains is to greatly reduce the uplift pressure at the heel and thereby reduce the uplift force under the dam. This reduction is accounted for in design by use of the uplift intensity factor δ. Without a cutoff the uplift head at the heel would be h_1 or $h_2 + (h_1 - h_2)$, in which $(h_1 - h_2)$ is the differential uplift head, or headwater minus tailwater. With a cutoff, assumed for example to be 50% efficient (i.e., $\delta = 0.5$), the differential uplift head would be reduced to $0.5\ (h_1 - h_2)$, and the resultant uplift head at the heel would be $h_2 + 0.5\ (h_1 - h_2)$. It is important to note that the intensity factor applies only to the differential head at the heel and not the total head h_1. The reason for this is that the upstream cutoff can in no way reduce the uplift pressure which originates with the tailwater. In other words, even if the cutoff was 100% effective, a pressure head of h_2 would still exist under the entire base. Figure 8 illustrates these effects.

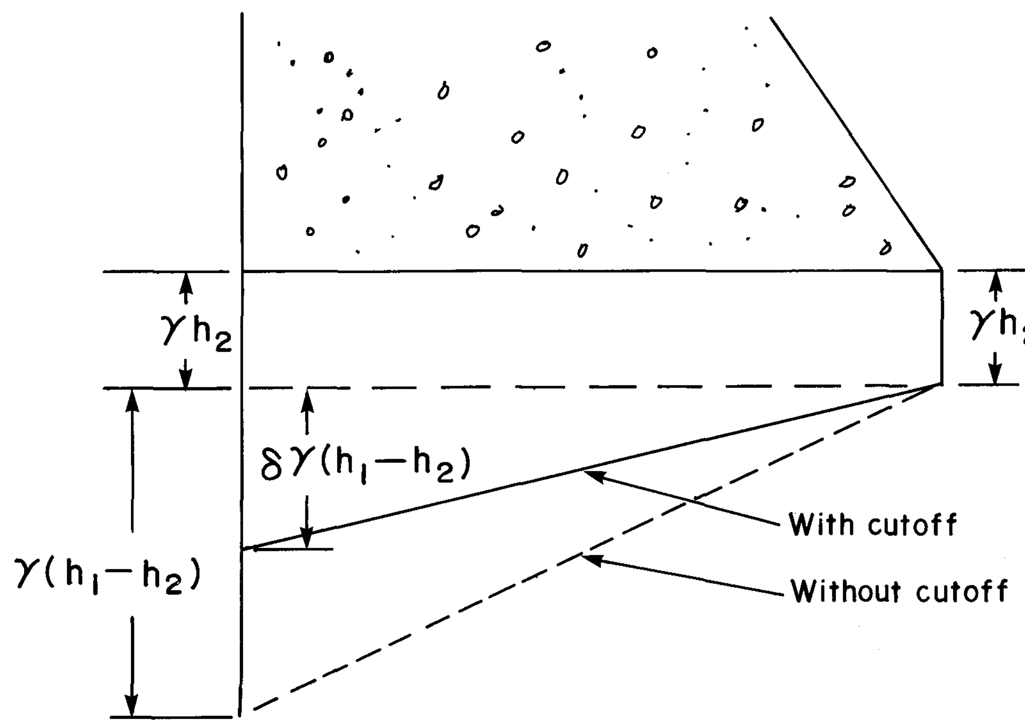

Figure 8. Effect of Cutoff on Uplift Pressure Distribution

Using an uplift pressure of $\gamma h_2 + \delta\gamma\ (h_1 - h_2)$ at the heel and γh_2 at the toe, the uplift force U then is

[6] $$U = \gamma[h_2 + \delta(h_1 - h_2)/2]b$$

Recommended values of δ range from 0.33 to 1.0. Naturally a value of unity should be used for dams without cutoffs. Pore pressures observed on some recent well constructed dams on good foundations have indicated intensity factors far lower than 0.33, but until these dams withstand the test of time it is not prudent to recommend lower design values. Usually a value of 0.5 is selected for design.

Equation [5] can still be used to locate the position of the resultant uplift force if h_1 is replaced by $(h_2 + \delta (h_1 - h_2))$. This is necessary to account for the different shape of the trapezoidal uplift pressure distribution diagram if δ is not unity.

9. Wind Effects

(a) **Wind Tide.** One obvious effect of wind blowing over water is the generation of waves. A less obvious effect, but one which can be equally important in terms of force on the dam, is due to wind tide. When wind blows over water, a horizontal force develops on the water surface in the direction of the wind. This results in the water piling up at the downwind boundary of the reservoir. Naturally there will be a drop in water surface elevation at the upwind boundary as well, but this is not important in design.

This phenomenon is referred to as wind tide or setup, S, where S represents the rise in water level above the normal horizontal reservoir surface at the downwind boundary. If setup can occur at the same time as the reservoir is at maximum level, then the increase in reservoir depth must be taken into account when calculating the hydrostatic force on the upstream face of the dam. It may be quite significant, particularly for low dams on shallow reservoirs.

The variables affecting setup are wind velocity, reservoir topography, and fetch length. The fetch length for wind tide is the straight line distance between the dam and a point upstream in the reservoir. The point is located to give the maximum distance for a wind blowing over the reservoir area toward the dam, as shown in Figure 9. It is not necessary that the line selected be continuously over water because setup can accumulate around bends.

The design wind velocity in the direction of the fetch may be determined from a study of the wind rose and frequency data for the area in question. The design wind is often selected as having a recurrence interval of 50 to 100 years. Most wind velocity data taken at meteorological stations give the wind velocities over land, as taken at 7.5 m height above ground. Due to the absence of topographical relief, the over water wind velocity is invariably greater than this, and may be accounted for by multiplying the over land wind speed by a factor depending on the length of the fetch. If the fetch length exceeds 10 km the multiplying factor is 1.3, which is the maximum. For shorter fetch lengths the factor is reduced. The effect is shown in Table 2.

TABLE 2
FETCH WIND SPEED CORRECTION FACTOR

Fetch length, km	0	1	2	3	5	10
Correction factor	1.0	1.09	1.15	1.20	1.25	1.30

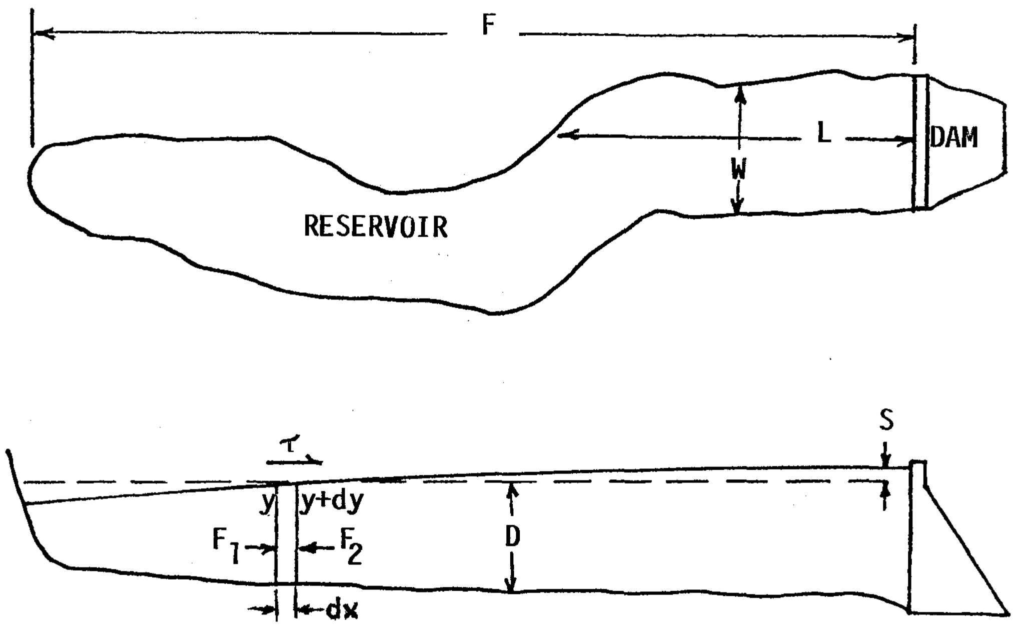

Figure 9. Definition Sketch for Setup

The wind speed used in calculations is the average sustained wind speed, and not the maximum velocities recorded during short duration gusts. It is possible to have wind gust velocities up to twice the average velocity. Typical design wind velocities in North America range from 80 to 160 km/h, except in hurricane areas where velocities of 240 km/h may be reached. An indication of the variation in wind velocity in different parts of Canada is shown in Figure 10. This figure shows the isovels for the maximum average wind speed of one hour duration, as determined by the Atmospheric Environmental Service over a 30 year period of record. Average wind speeds can be expected to be higher for durations shorter than one hour, and lower for longer durations. Also a 1:50 wind would have a somewhat larger wind speed than the 1:30 wind.

Wind durations required to develop maximum setup are primarily a function of fetch length. Suggested values from the Guide to Bridge Hydraulics (edited by Neill) are given in Table 3. These values are a guide only, since actual setup times are affected by the reservoir profile and planform.

TABLE 3
DURATION OF SUSTAINED WIND TO PRODUCE MAXIMUM SETUP

Fetch (km)	≤ 15	30	50	65
Wind Duration (h)	2	3	4	5

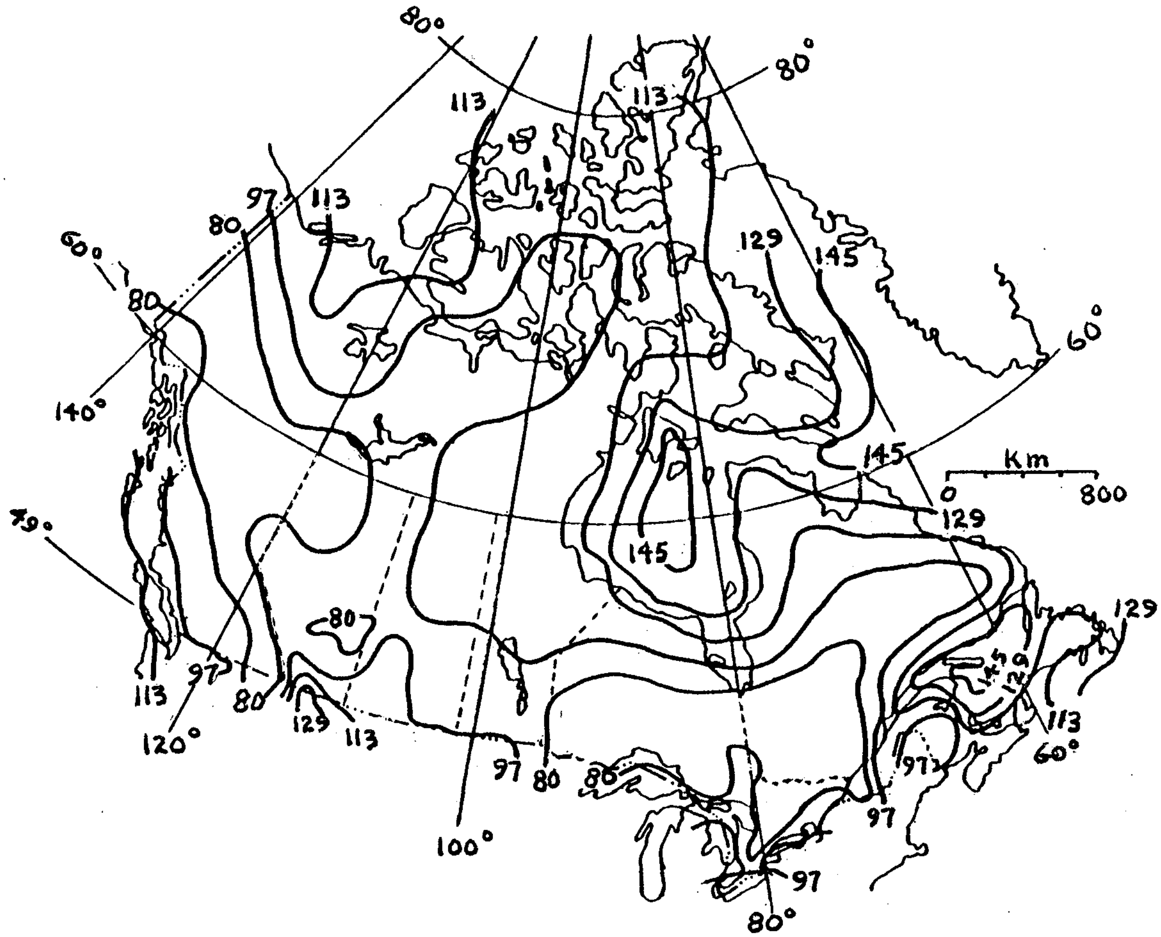

Figure 10. Maximum Hourly Wind Speed in km/h for Annual Probability 1:30 (from Atmospheric Environment Service 1986)

An equation for setup may be derived as follows: Consider an elemental length dx and unit width, as shown in Figure 9. Due to the water surface slope dy/dx, the hydrostatic force F_2 is greater than F_1. The equilibrium equation is

$$\tau dx = F_2 - F_1$$

in which τ is the shearing stress on the water surface due to the wind. This shearing stress may be expressed in terms of the surface drag equation, and hence

$$C_f \rho \, dx \, V^2/2 = \gamma[(y + dy)^2 - y^2]/2$$

or

$$dy/dx = C_f \rho \, V^2/(2\gamma y)$$

from which

$$dy/dx \propto V^2/y$$

This equation shows that with a variable reservoir depth, the water surface slope is not constant over the whole reservoir even for a constant wind velocity. However, if y is taken as the average reservoir depth D over the fetch length, the calculated water surface slope will also represent an approximate average value, from which

$$S \propto F\, dy/dx \propto FV^2/D$$

or

[7] $$S = kFV^2/D$$

where k is a constant of proportionality.

The factor k includes all the constants and has been established from observations on prototypes. With k having the units of 1/g, [7] is dimensionally homogeneous; however, for convenience it is customary to express F in km and V in km/h. With S and D expressed in metres, the equation reduces to

[8] $$S = FV^2/(63{,}000D)$$

This is known as the Zuider Zee formula, having been originally developed from studies in Holland in the 1930's.

Normally both the design wind velocity and the fetch length will change if a different direction is selected for the analysis. It is evident from [8] that the maximum setup will occur for the direction for which FV^2 is a maximum. Thus, a shorter fetch could give a greater setup if the wind velocity in that direction is substantially greater.

Equation [8] is accurate for a reservoir of constant depth and rectangular in planform. Normally the reservoir is both deeper and wider at the dam (downwind boundary of the fetch) and [8] must be modified to

[9] $$S = k_1\, k_2\, FV^2/(63{,}000D)$$

in which k_1 is the setup reduction factor for an increase in depth at the dam and k_2 is the planform factor to account for a change in reservoir width. Table 4 gives values of k_1 for various ratios of the upwind depth to downwind depth, D_u/D_d, based on a more or less uniform rate of depth variation along the fetch, with $D_u < D < D_d$.

TABLE 4
SETUP REDUCTION FACTOR FOR INCREASE IN RESERVOIR DEPTH

D_u/D_d	1.0	0.8	0.6	0.4	0.2
k_1	1.0	0.972	0.941	0.922	0.911

The value of k_2 is given by

[10] $$k_2 = 0.667\,(2b_u + b_d)\,/\,(b_u+b_d)$$

in which b_u is the reservoir width at the upwind boundary of the fetch and b_d is the width at the downwind boundary. For a water body that is triangular in plan, with the apex of the triangle at the upwind boundary, then from [10], with $b_u = 0$, $k_2 = 0.667$. If the reservoir is rectangular in planform, with $b_u = b_d$, then $k_2 = 1$. It is also possible for k_2 to be greater than unity if the water body converges in the downwind direction ($b_u > b_d$). These corrections arise from consideration of the fact that the increase in water volume under the setup above the original still water level must equal the loss of volume due to setdown below the original still water level.

(b) Waves. Wind generated waves are not regular. In a wave group there will be waves of different heights, and since the wave velocity depends upon the height, various combinations of reinforcement and annulment occur. In coastal hydraulics, wave studies are based on a wave height called the significant wave height, which is the average height of the highest one-third of the waves. Frequency studies on actual storm waves have shown that the significant wave height is equalled or exceeded by 13% of the waves. The significant wave height has also been adopted for design on inland lakes and reservoirs. Based on studies of 45 storms over Denison and Fort Peck reservoirs, the Corps of Engineers recommended the dimensionally homogeneous equation

[11] $$h_w = 0.0026V^{1.06} F_e^{0.47}/g^{0.53}$$

in which h_w is the significant wave height in m, V is the design wind velocity in m/s, F_e is the effective fetch in m, and g is acceleration due to gravity. With the wind velocity in km/h and the fetch in km, the equation can be reduced for design purposes to

[12] $$h_w = 0.00513\, V^{1.06} F_e^{0.47}$$

Also, the equation for wave length L_o is given by

[13] $$L_o = 0.187\, V^{0.88} F_e^{0.56}$$

Equations [12] and [13] are for deep water waves and apply to the case where the reservoir depth over the generation area is great enough so that wave heights will not be reduced by bottom effects. This is usually a reasonable assumption for winds blowing toward the dam because the reservoir depth is greatest at the dam. Bottom effects can be disregarded if the depth at the dam exceeds one-third of the wave length L_o, although the effect is still small even at one-quarter of the wave length.

The fetch F_e used in wave calculations is not the same as the fetch used in wind tide calculations. Wind tide effects can be transferred around bends or through openings in causeways, whereas wave effects cannot. Further, wave heights may be reduced by frictional effects on the sides of a narrow reservoir. Although some sophisticated methods have been developed to compute the effective fetch for wave height calculations, for most designs the effect of the average reservoir width W can be accounted for by multiplying the maximum straight unobstructed water length L facing the dam by a coefficient K such that

[14] $$F_e = KL$$

Values for K according to Saville are given in Table 5.

TABLE 5
FETCH CORRECTION FACTOR

W/L	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.8	1.0	1.2	1.5	2.0
K	0.00	0.26	0.40	0.51	0.60	0.67	0.73	0.83	0.90	0.94	0.98	1.00

Wind durations required to develop the maximum wave height are given in Table 6. Although fetch values in excess of 1000 km are possible in coastal hydraulics, the irregularities of shape and alignment on inland reservoirs make it unlikely that a 50 km effective fetch would be exceeded, even for the deepest reservoirs. Accordingly, the required wind durations are quite short, particularly for the higher velocity winds, so it is usually necessary in practice to design for the maximum wave height. However, the very short durations indicated in Table 6 for the short fetch values would suggest that higher design winds should be used for the smaller reservoirs. For example, in an area where an 80 km/h design wind is used for a large reservoir with a 30 km fetch, a 90 or 100 km/h wind should probably be used for a small reservoir with a 1 km fetch. The rationale here is that in an extensive sustained 80 km/h wind storm, it is possible to get a local shorter duration wind of higher velocity.

TABLE 6
WIND DURATION TO DEVELOP MAXIMUM WAVE HEIGHT
(Tabled values are wind duration in hours)

Wind Velocity km/h Fetch km	50	75	100	125
2	0.45	0.37	0.32	0.29
5	0.82	0.69	0.60	0.52
10	1.38	1.15	1.02	0.92
15	1.88	1.56	1.38	1.27
20	2.38	1.96	1.73	1.58
30	3.20	2.63	2.32	2.08
40	3.93	3.24	2.86	2.56
50	4.60	3.82	3.37	3.03

A further limitation in the use of [12] and [13] is given by

[15] $$F_e(\text{max.}) = 0.031\ V^2$$

with F_e in km and V in km/h. The reason for this limitation is that wave heights will not increase indefinitely with fetch length. There is an upper limit to the wave height independent of fetch length, just as there is an upper limit independent of wind duration. Equation [15] is important in coastal hydraulics, or perhaps on the Great Lakes, where a large unobstructed expanse of water exists. Examination of the equation will show, however, that this fetch limit will not be reached for artificial inland reservoirs.

When a deep water wave approaches a smooth vertical barrier the wave does not break, but is reflected as a wave of double the amplitude. The vertical pressure distribution in the wave is somewhat less than hydrostatic, but conservatively a hydrostatic pressure may be assumed, for which the wave force P_w in kN/m is

[16] $$P_w = \gamma(2h_w)^2/2$$

This force may be applied at the mean water surface elevation, and should be superimposed on the full hydrostatic force due to headwater.

The horizontal water forces due to setup and waves are often neglected for reservoirs over 60 m deep. The setup for deep reservoirs is negligible, and even for a 3 m wave the wave force would be less than 1% of the hydrostatic force on a 60 m dam. Some designers neglect wave forces even for lower height dams, reasoning that due to wave irregularities it is unlikely that the wave will peak along the entire face of the dam at the same time.

It should be pointed out that for waves approaching a shoreline or barrier over a gently sloping bottom, the wave length will be progressively reduced and the height amplified as the wave moves into the shallows. Once the wave steepness h_w/L_o exceeds one-seventh, the wave will break. The impact forces associated with a breaking wave can be many times greater than indicated by [16]. Generally, it is advisable to conduct model studies for shoreline structures because the wave performance can be affected by the local site geometry.

10. Earthquake

Earthquake, which is a violent shaking of the earth's crust, may be treated as a reversing horizontal acceleration. Due to the inertia of the dam, it tends to resist the motion, and the stresses in the dam and foundation may increase momentarily. In the static loading method of analysis, the motion is replaced by the equivalent inertia force P_e, as given by

[17] $$P_e = Ma$$

in which M is the mass of the dam and a is the earthquake acceleration. The force P_e is applied at the centre of gravity of the section.

Earthquake intensity is expressed in terms of the factor α, which is the ratio of earthquake acceleration to acceleration due to gravity. Since M = W/g, where W is the weight of the dam, [17] may be written as

[18] $$P_e = \alpha W$$

Design values for α range from zero to 0.40, the highest values for seismically active regions in Japan, parts of China, the Middle East and in North America in Alaska, California and the Yellowstone Park area. Earthquake magnitude may be expressed in terms of the Richter number n, in the expression

[19] $$n = \log_{10}(CA)$$

in which C is a constant and A is the horizontal amplitude of the ground motion 100 km from the centre of the disturbance. Put another way, the amplitude, or earthquake intensity, can be expressed as some constant times 10^n, which shows that each increase in n of one unit corresponds to a tenfold increase in intensity.

A quake with Richter number 3 is barely felt indoors, 5 gives minor damage, 6 significant damage, 7 severe damage, 8 widespread enormous damage, 9 catastrophic damage on a continental scale, and 10 a world scale quake. No quake exceeding 8.6 has ever been recorded. It is believed a Richter 10 could only be caused by earth collision with a large meteor several km in diameter. The 1906 San Francisco earthquake , in which 700 people died, was measured at Richter 7.9. The 1978 Iran earthquake, in which 20,000 were killed, measured 7.7.

The horizontal g forces on a structure produced by horizontal acceleration are obviously related to the Richter number. Unfortunately a direct correlation is not possible because acceleration at a structure depends on distance from the epicentre, soil conditions, and type of structure. However, a seismic risk map has been prepared by the California Applied Technology Council for the United States, as shown in Figure 11. This map gives earthquake acceleration contours in terms of g forces. The risk of exceeding the indicated value in a 50 year period is stated as 1 in 10.

Although a similar risk map has not been prepared for Canada, much of Canada is considered seismically inactive and earthquake load is often neglected. Values of 0.1 may be assigned for the West Coast regions or for southern Ontario.

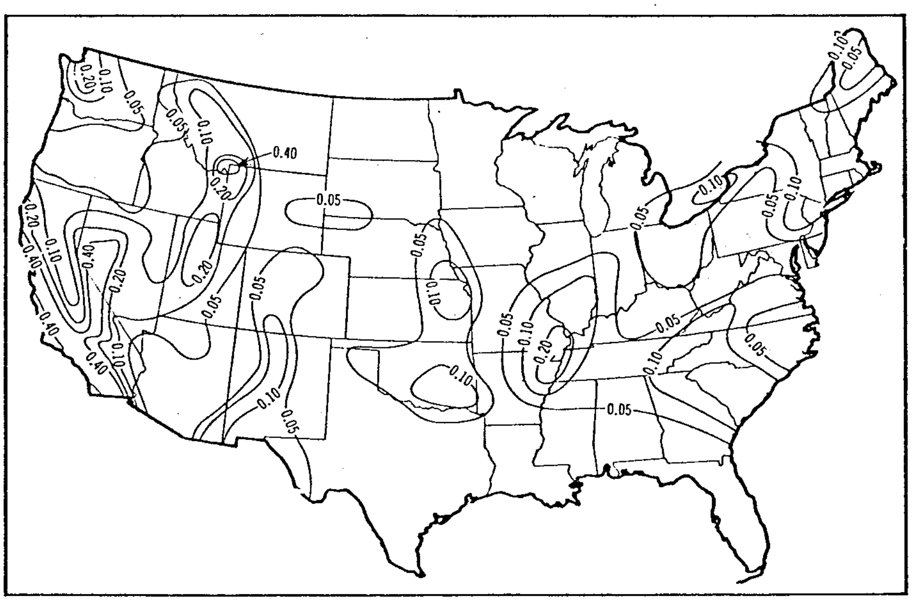

Figure 11. ***Seismic Risk Map - USA (Value on contour represents the ratio of earthquake acceleration to the acceleration due to gravity)***

It has been observed that the period of motion for an earthquake is many times greater than the natural frequency of a concrete dam, so resonance will not occur and the risk of a large amplitude response is slight. The static loading method is believed to be conservative because the actual acceleration response of the dam decreases with height above the base, and therefore the true earthquake force is somewhat smaller than assumed. If necessary, earthquake effect on important dams may be model studied using a shaking table.

Evaluation of the increase in load on the dam due to changed pressure distribution in the reservoir during an earthquake is quite complicated. As an approximation of the effect the load determined from [18] may be increased by 50% if α is 0.1 or less. More detailed analysis is recommended for larger values of α. The critical direction for earthquake load is downstream when the reservoir is full, upstream when the reservoir is empty.

11. Weight

The weight W is the principal stabilizing force for the gravity dam. It is applied vertically at the centre of gravity of the section. For design purposes it may be assumed that the concrete weight is 23.6 kN/m^3.

12. Foundation Reaction

All the loads applied to the dam or by the dam must ultimately be transmitted to the foundation on which it rests. The foundation reaction is the force which is equal in magnitude and opposite in direction to the resultant of all the other forces. The problem of stability analysis, therefore, involves a relatively simple application of principles of static equilibrium.

C. STABILITY CRITERIA FOR GRAVITY DAMS

13. Overturning

Consider conditions existing along the horizontal plane m - n in Figure 12. This plane may be at the base of the dam, or at any horizontal section through the dam above the base where a stability check is to be made. The forces acting on the section above m - n may be combined into a single resultant force R. Since R may be applied at any point along its line of action, it can be applied at point o where the line of action cuts m - n. The components of the resultant are R_H and R_V. Evidently R_H will produce only sliding or shear stresses on plane m - n, whereas R_V will produce only compression or tension stresses on plane m - n.

Obviously overturning per se would occur if the resultant R fell outside of the base downstream from n. However, a failure would occur long before the resultant reached this position. As R moves closer to the toe, the pressure would decrease at m and increase at n. Eventually tension cracks would form at the upstream face of the dam, resulting in a further increase in uplift pressure, and excessive compressive stresses would occur at the toe, resulting in crushing. This development would be accompanied by movement, tipping, and rupture in the concrete and/or foundation, and probably loss of the reservoir. Although the dam may not actually overturn, the result is still a failure.

In a sense the word "overturning" is a misnomer, because the objective in design is to keep the compressive stresses within the allowable limit, and prevent the development of any tension stresses in the concrete.

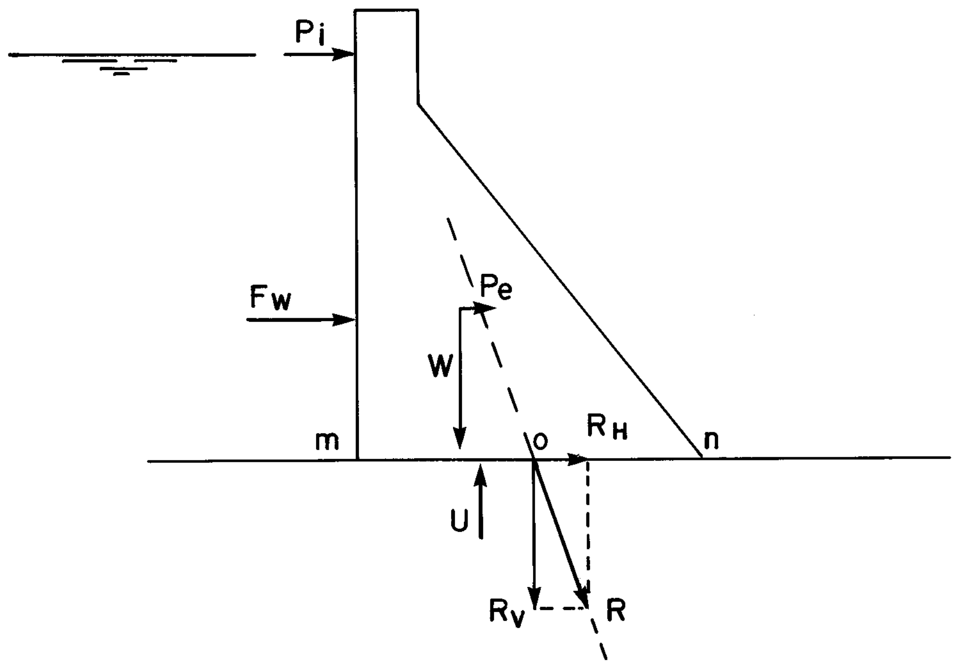

Figure 12. Resultant Forces

Concrete is weak in tension, the ultimate tension strength being only one-tenth of the ultimate compressive strength. Since the gravity dam is not reinforced with steel, it is considered unwise to permit any tension stresses whatsoever.

It may be assumed that the distribution of vertical stress between m and n is linear, as shown in Figure 13. In this case the actual stresses at these extreme points may be solved using the combined stress formula, as in the case for an eccentrically loaded column. Hence, for a unit width,

[20] $$f = R_V/b \pm M(b/2)/I$$

in which f is the vertical stress in kN/m^2, R_V the resultant vertical force in kN, b the base width in m, M the bending moment in kN.m, and I the moment of inertia of the section. The bending moment may be written in terms of R_V as

[21] $$M = R_Ve$$

in which e is the eccentricity of the force R_V (i.e. the horizontal distance from the centre of the base to the point where R_V acts). The moment of inertia of the base section, which is b metres long by 1 m wide, is,

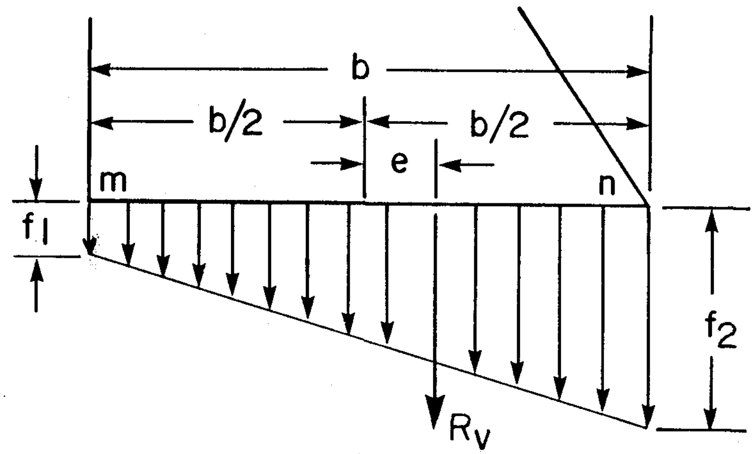

Figure 13. Foundation Stress Distribution Diagram

[22] $I = b^3/12$

If [21] and [22]are substituted into [20], the stress equation becomes

[23] $f = R_v\ (\ 1 \pm 6e/b)/b$

The + sign is used to solve for the toe stress and the - sign to solve for the heel stress. Of course e can also be negative for certain conditions of loading.

Equation [23] may be used to solve for the permissible eccentricity of the resultant. Since no tension is allowed, the minimum allowable value for f is zero, and the limits for e are then

[24] $e = \pm\ b/6$

The length ± b/6 comprises the middle third of the base. This is the basis for the Middle Third Rule, which states that the resultant force must fall within the middle third of the base for all conditions of loading, including uplift. This rule must be satisfied with the reservoir full or empty, and at any horizontal section through the dam above the base as well as the base itself. This insures that there will be no tension anywhere in the cross section, and overturning criterion will be satisfied.

In design, it is customary to select the base width so that under the worst conditions of loading the resultant of all the applied forces will cut the base right at the downstream third point. This gives the shortest permissible base length and therefore the most economical cross section for the dam.

A simple overturning analysis can be used to determine the most economical shape for a gravity section. A number of possible basic shapes are shown in Figure 14. The most economical basic shape for the dam will be the one with the smallest cross-sectional area. This may be found by first solving for the required values of b to satisfy overturning criterion, that is by setting the summation of the moments of the forces equal to zero with the resultant of the foundation reaction acting at the downstream third point. It is most convenient to take moments about the downstream third point so the resultant will be eliminated from the equation. For purposes of illustration only the principal forces, water, weight and foundation reaction, are considered in the analysis.

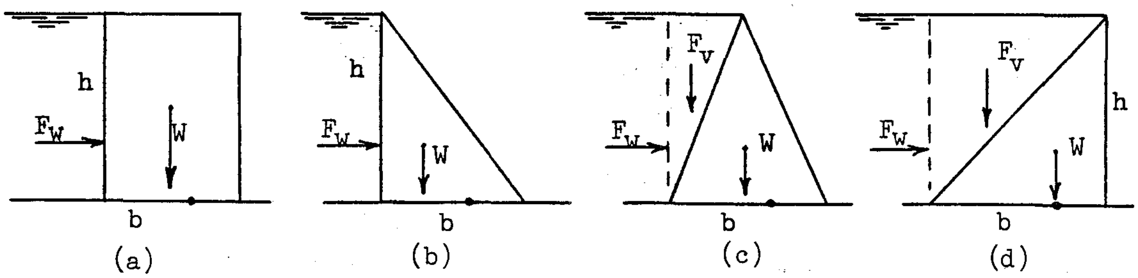

Figure 14. Possible Dam Shapes

Referring to Figure 14, and using γ_c as the specific weight of concrete at 23.6 kN/m^3, and for water $\gamma = 9.81\ kN/m^3$ then for $\Sigma\ M_o = 0$ the moment equations are as follows:

Case (a) $\gamma h^2/2 \times h/3 - \gamma_c bh \times b/6 = 0$

$b/h = \sqrt{\gamma/\gamma_c} = 0.645$

Case (b) $\gamma h^2/2 \times h/3 - \gamma_c bh/2 \times b/3 = 0$

$b/h = \sqrt{\gamma/\gamma_c} = 0.645$

Case (c) $\gamma h^2/2 \times h/3 - \gamma hb/4 \times b/2 - \gamma_c bh/2 \times b/6 = 0$

$b/h = 0.716$

Case (d) $\gamma h^2/2 \times h/3 - \gamma hb/2 \times b/3 - \gamma_c hb/2 \times 0 = 0$

$b/h = 1.00$

Evidently section (b), the triangular section with vertical face upstream, is the most economical section. Section (a), the rectangular section, is sometimes used for low height dams such as cellular steel cofferdams or rock filled timber cribs. This is required because of the type of construction involved. An interesting point about section (d) is that it would satisfy the overturning stability criterion even if it was weightless because the weight force has no moment arm about the downstream third point. As a consequence, this shape, triangular with the inclined face upstream, is the basic shape for the hollow concrete dam. Stability is obtained from the vertical component of water force on the upstream face.

The b/h values from the preceding analysis are not to be used in actual designs. When all loads are considered, the actual b/h values for concrete gravity dams usually fall in the range of 0.7 to 0.8, except for case (d) where b/h = 1.

14. Sliding

Sliding could occur at the foundation joint, at any horizontal joint above the base, or even in the foundation below the base should a weak strata be present. If the foundation is homogeneous, sliding at the foundation joint is usually critical. To ensure adequate safety against a sliding failure, either the friction formula or the shear - friction formula must be satisfied.

For dams constructed on rock, the friction formula for sliding stability is

[25] $$R_H < fR_V$$

in which R_H is the summation of all the horizontal forces above the potential sliding surface, R_V is the summation of all the vertical forces, including uplift, and f is the coefficient of static friction. The value of f is determined from laboratory tests of the concrete and rock using well dressed specimens with smooth plane surfaces. Values of f for concrete on concrete and concrete on rock are about 0.7. This automatically provides for a factor of safety of at least two, because the actual sliding resistance for non-planar roughened or keyed surfaces, as actually used in construction, is at least twice as great as for well dressed specimens of the same material. Furthermore, the value of 0.7 does not include any internal shear strength of the material due to cohesion or bonding at the joints.

For dams constructed on sand or gravel, [25] must be modified to include a specified factor of safety, as given by

[26] $$R_H < fR_V/S_f$$

in which S_f is the safety factor. The value of S_f is usually taken as 2 or more. The reason a specified safety factor must be used is because sliding can occur in the foundation just below the base, and the experimentally determined value for f for the gravel or sand corresponds to the actual value which will exist in situ. There is no increase in shearing resistance due to cohesion since sands and gravels are cohesionless. The value for f is tan ϕ, where ϕ is the angle of internal friction, and ranges from 20° to 35° for sands and gravels.

For dams constructed on rock an alternative to the friction formula is the shear-friction formula. In this case the entire sliding resistance is considered, including shear strength due to cohesion, and a specified safety factor is applied. By using special construction procedures, it is possible to develop a significant shearing strength in the concrete and foundation rock. The surface to be concreted must be cleaned to remove all dirt and loose material. This may be done by sandblasting and sluicing. If the surface is a previous pour of concrete, the laitence film must be completely removed. The surface is brushed with a cement mortar paste just prior to concreting in order to insure a good bond. All horizontal and vertical construction joints are keyed, and the blocks comprising each lift may even be staggered. In this way a sliding failure cannot occur without overcoming the shearing resistance of at least part of the material.

The shear-friction formula is

[27] $$R_H \leq (fR_V + rvA)/S_{s\text{-}f}$$

in which R_H, R_V, and f are the same as for the friction formula, v is the ultimate shearing strength of the material, A is the base area, r is the ratio of the average shear stress to the maximum shear stress, and $S_{s\text{-}f}$ is the shear-friction safety factor. Values of v range from 4000 to 8000 kPa, depending on the strength of the concrete and rock, r is taken as 0.5, and a value of 4 or 5 is used for the shear-friction safety factor. For a unit width analysis the area under consideration is b x 1 m^2. The factor r is needed to account for the distribution of horizontal shearing stress along the base. Because shearing stresses are zero on planes of principal stress, the horizontal shearing stress is zero at the vertical upstream face of the dam, increasing to a maximum at the inclined downstream face. A conservative value must be used for the factor of safety because [27] includes the total sliding resistance without any allowance for strength variations due to thermal effects, shrinkage, or inadequate bonding at some joints.

If the base width determined from the overturning analysis satisfies the friction formula, a further check on sliding is unnecessary. If the friction formula is not satisfied, then the expedients to develop shearing resistance are resorted to, and the shear-friction formula is applied.

15. Compressive Stresses

The maximum principal stress in the dam and foundation must be kept within allowable design values. Whereas uplift is critical in overturning and sliding stability calculations, the worst case for compressive stresses is with no uplift. Without uplift the entire weight of the dam must be carried by the solid particles of the dam and foundation. Zero uplift must be considered as a possible loading condition. In fact it will occur as an initial loading condition before uplift develops, and will apply also if the measures taken to prevent uplift are completely successful.

Principal stresses occur on planes parallel to and perpendicular to free boundaries, and hence one plane of principal stress is perpendicular to the inclined downstream face of the dam. The stress on this plane, f' is often called the inclined stress to distinguish it from vertical stresses. The vertical stresses are not necessarily principal stresses, but they can be used to solve for them as follows: Consider a small triangular element abc on the downstream face of the dam, as shown in Figure 15. The principal stress f' acts on plane ab and is parallel to the downstream face. On plane bc there exists the vertical stress f and a shearing stress v. For static equilibrium the summation of vertical forces must be zero, whence

$$f'\overline{ab}\cos\alpha - f\,\overline{bc} = 0$$

Also $\cos\alpha = \overline{ab}/\overline{bc}$, so

[28] $$f' = f/\cos^2\alpha$$

Equation [28] shows that for $\alpha = 45^\circ$, the principal stress is twice the vertical stress. The vertical stress at the point in question may be solved by applying the combined stress equation, that is [23], excluding uplift. In this case $R_V = W$.

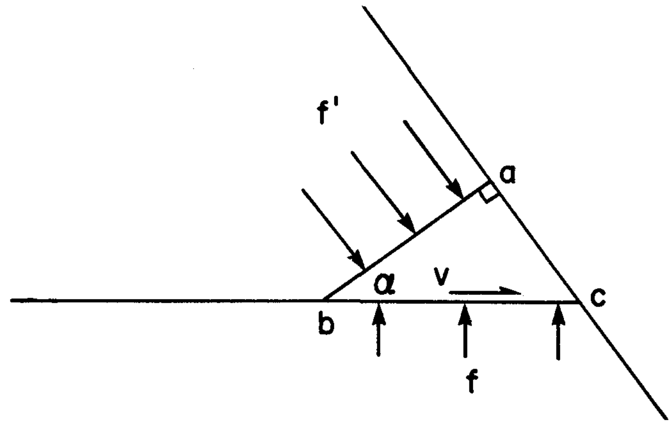

Figure 15. Vertical and Principal Stresses

At the toe of dam the stress f′ will exist in both the dam and the foundation. This stress must be checked against the allowable concrete and foundation stress. The allowable concrete stress usually ranges from 2000 to 4000 kPa. These stresses are much lower than used in building construction, since 10 MPa concrete may be used, in contrast to 20 or 30 MPa concrete in building construction. In a gravity dam high strength is not essential and dead weight is an asset rather than a liability. Use of lower strength concrete permits cost savings in terms of reduced cement content. The allowable foundation stress must be determined from tests on the foundation material. Some typical values are shown in Table 7. It is evident that the nature of the foundation has a significant influence on the permissible height of the dam. Only the best rock foundations are suitable for very high dams.

TABLE 7
ALLOWABLE FOUNDATION STRESSES

Foundation Material	Allowable Stress* (kPa)
Granite	4000 - 6000
Limestone	3000 - 4000
Sandstone	2500- 3500
Gravel	300- 600
Sand	200 - 400
Stiff Clay	200 - 400
Soft Clay	50 - 100

*Approximately 1/5 of the ultimate strength

The use of an allowable foundation stress as low as 1/5 of the ultimate strength may appear to be unduly conservative. However, such a high apparent factor of safety reflects the degree of uncertainty about the foundation. While concrete production for the dam may be subject to inspection and control, foundation weaknesses may not be revealed by the subsurface exploration. Conservative factors of safety are warranted for major dams, given the possible consequences of a failure. Such consequences may include serious damage and loss of life, in addition to loss of the investment.

D. DESIGN PROCEDURE

16. Calculation of the Base Width

As indicated in Section 13, the basic section for a concrete gravity dam is triangular with the vertical face upstream. It is possible to start the design from the top of the dam, satisfying the various stability criteria at selected increments of height as the design proceeds toward the base. This is called the multiple-step design. The multiple-step design usually gives non-planar surfaces for both the upstream and downstream faces. Alternatively, the single-step design procedure may be used, in which the base width is established first and other elevations checked later. Plane surfaces are assumed for the upstream and downstream faces. Although the cross section is somewhat more economic for the multiple-step design, the single-step design has been used even for major dams. The design takes less time, construction is simpler, and it may be preferred aesthetically.

In the single step design procedure, the height h of the basic triangle is selected so the apex is at the elevation of the maximum expected reservoir surface, including setup, if any. The base width b of the triangle necessary to satisfy overturning criteria is solved for the worst condition of loading. It may be necessary to check several different loading combinations to determine which one is most critical. In particular, the effect of tailwater cannot always be determined from inspection, since a high tailwater will increase the uplift but reduce the net horizontal water force.

The base width determined in the overturning analysis is used to check sliding stability. If the friction formula cannot be satisfied, then either the base must be increased, or the shear - friction formula must be applied. When the shear - friction formula is used, it must be understood that a more expensive construction procedure is required to develop the required sliding resistance.

Finally, the principal stress at the toe of the dam is solved. If this exceeds the allowable stress in the foundation either the base must be widened or a different type of dam investigated.

Regardless of the load combinations investigated, uplift must always be included for overturning and sliding analysis, and omitted for compressive stress analysis.

17. Freeboard and Top Width

It is evident that certain modifications to the basic triangle must be made in order to secure a functional design. The dam must have a top width in order to facilitate crossings by operators or traffic. In addition, a substantial top thickness is needed to resist ice pressure. The top width is usually selected between 0.1 h and 0.15 h. If a wider crest is needed to accommodate more traffic lanes, the extra width is obtained by a deck slab and buttress design of thin reinforced concrete sections. A massive top section is unnecessary, and, in fact, is undesirable if a significant earthquake load must be considered.

In order to prevent overtopping of the dam due to wave action, freeboard is required. Freeboard may be defined as the height of the dam crest above the maximum operating level of the reservoir. This level may be above the normal operating FSL, as usually there is some allowance for reservoir surcharge to accommodate flood storage. Since the design flood normally will be associated with severe storm conditions over the drainage area, there is some probability that the design wind will occur at the same time as the design flood. Accordingly, it may be necessary to consider setup in arriving at the maximum reservoir elevation.

The freeboard allowance must include the setup S, wave runup R, and some nominal extra height E, as expressed in

[29] $$FB = S + R + E$$

and as illustrated on Figure 16. The extra height E represents a margin of safety, but would be required in any case if it was intended to prevent all waves from overtopping. The reason for this is that the runup is based on the significant wave height h_w, and in a wave group the significant wave height is equalled or exceeded by 13% of the waves.

For a concrete gravity dam, a value of 0.5 m is adequate for E. This may not contain all the waves but an occasional splash blown over the dam during an extreme storm is not considered serious. Frequently, part of the height needed for freeboard is obtained by constructing a 1 m high vertical reinforced concrete parapet wall at the upstream face of the dam. In this way, the solid part of the dam crest can be placed 1 m below the elevation required for freeboard, and a significant saving in material can be made. It is beneficial to use a rollback lip at the top of the parapet wall. This is a small overhanging projection intended to intercept and deflect back into the reservoir any runup that exceeds the freeboard.

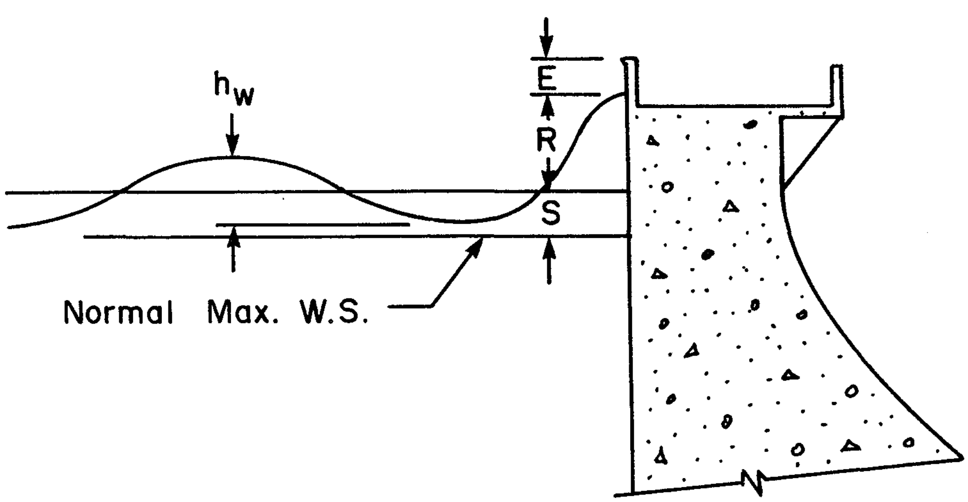

Figure 16. Freeboard Allowance

For earth dams the consequences of wave overtopping are more serious. It is more prudent to design for the 1% or 2% wave rather than significant wave height. Observations have shown that the height of the 1% wave is 1.60 times the significant wave height given by [12], and the 2% wave is 1.45 times the significant wave height.

When a wave reaches a barrier such as a causeway, breakwater, or dam, it will ride up the face of the barrier to some height above the mean water level. This is called wave runup, R, and is defined as the height in metres above the mean water surface that the crest of the wave will reach when it interacts with the barrier. The runup depends upon the slope and roughness of the face, the amplitude and steepness of the wave, and the water depth in the approach area. For storage dams the water depth in the approach area is usually deep enough so depth is eliminated as a significant variable, and the waves are considered as deep water waves. The error in this assumption is not significant as long as the water depth at the dam is not less than 1/4 of the wave length. Wave steepness affects runup in that for a given amplitude a flat wave has greater duration and hence a greater runup. Wave steepness is accounted for in terms of the wave length L_o in the ratio h_w/L_o. It will be found from [12] and [13] that wave steepness decreases, and hence relative runup R/h_w increases, as the fetch increases.

Figure 17 gives the relative runup for smooth sloping face dams according to Saville. These curves may be used to determine R for smooth face dams of steel, timber, or concrete, or earth and rock dams with a concrete facing. It is apparent that the greatest runup occurs for sloping faces that are not too flat. Runup is greatly reduced for very flat slopes because the wave breaks and its energy is lost before it reaches the embankment. If the upstream face of the dam is protected by riprap, Saville suggests that the runup ratios of Figure 17 can be halved.

Figure 17 does not give runup ratios for steep and vertical face dams. Wave runup on steep face dams is complicated by the fact that waves are reflected from the face, and these combine with the incoming waves to produce an infinite number of possible combinations. Research in this area has not advanced to the same point exemplified by Figure 17 for sloping face dams. Fortunately, vertical face dams are invariably concrete, and therefore inerodible, so the knowledge deficiency in this area is not a serious handicap. In theory a value of $R/h_w = 1.0$ would apply to the particular case of an approaching sine wave reflected at double the amplitude. However, natural storm waves are often trochoidal in shape. Trochoidal waves have sharper crests and longer shallower troughs, and therefore more than half thewave height occurs above mean water level. For design purposes a runup ratio $R/h_w = 1.5$, based on the significant wave height, is recommended for smooth vertical face dams.

18. Upstream Flare

In theory an upstream flare must be used on a concrete gravity dam in order to satisfy the Middle Third Rule with the reservoir empty. A reservoir empty condition may occur just at the completion of construction before the reservoir is filled, or it may occur later if the reservoir must be drained for any reason. The resultant force for the basic triangle alone will fall exactly on the upstream third point with the reservoir empty. However, the effect of the added top section for freeboard and top width is to move the resultant further upstream. An upstream flare, as shown in Figure 18, will widen the base in the upstream direction and move the third points accordingly. The flare is usually a minor modification to the section, but it can be very significant if it is necessary to consider earthquake load with the reservoir empty. In this case the earthquake load is applied in the upstream direction.

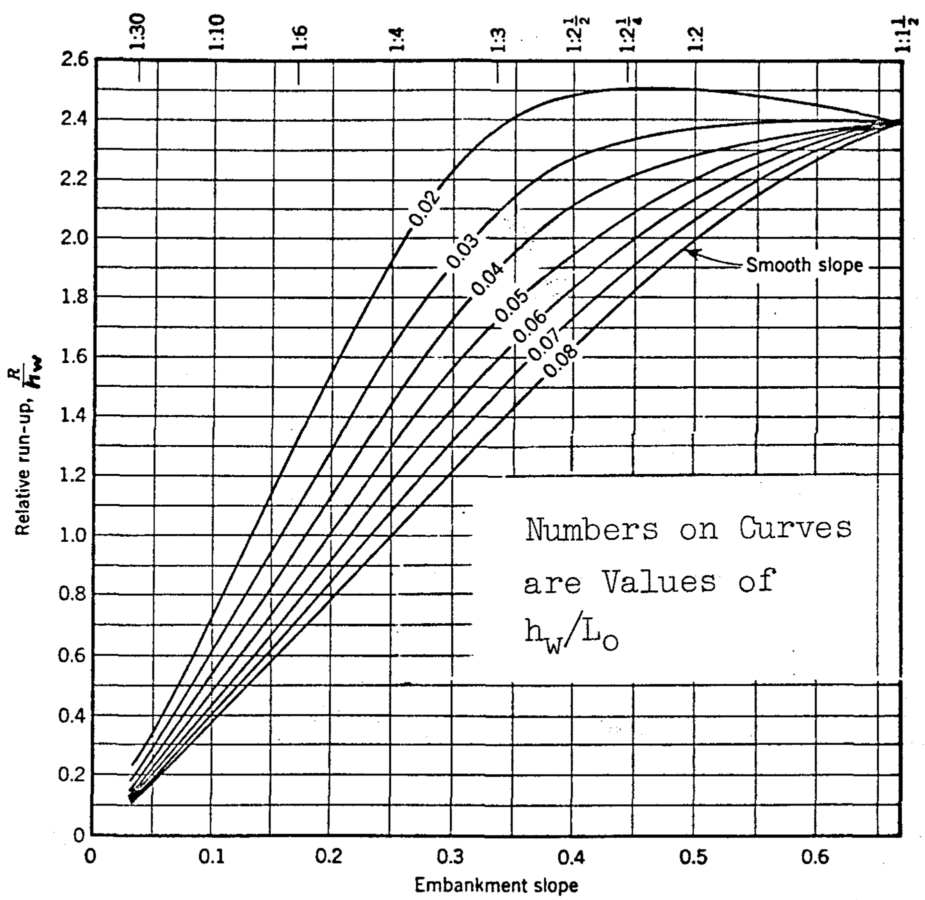

Figure 17. Wave Runup Ratios

The height of flare y may be located at the intersection of the upstream middle third line and the line of action for the weight of the shaded portion in Figure 18. This is the elevation below which the resultant for the top section and the basic triangle would fall upstream from the middle third zone unless the section is widened. The required width of flare x can be calculated by setting the summation of moments of weight forces, taken about the new upstream third point, equal to zero.

In practice it is not usually necessary to satisfy the Middle Third Rule with the reservoir empty. To begin with some water is normally ponded in the reservoir during construction, and in fact for a very high dam filling of the reservoir may be started before the dam is topped out in order to permit utilization of the completed project without delay. Also, it is permissable to permit some tension at the toe during construction. This tension would be small, temporary and in a region that would ultimately be under compression when the reservoir is filled. As a result the upstream flare, when used, normally takes the form of a local fillet, as shown in Figure 19, rather than a high tapered wedge.

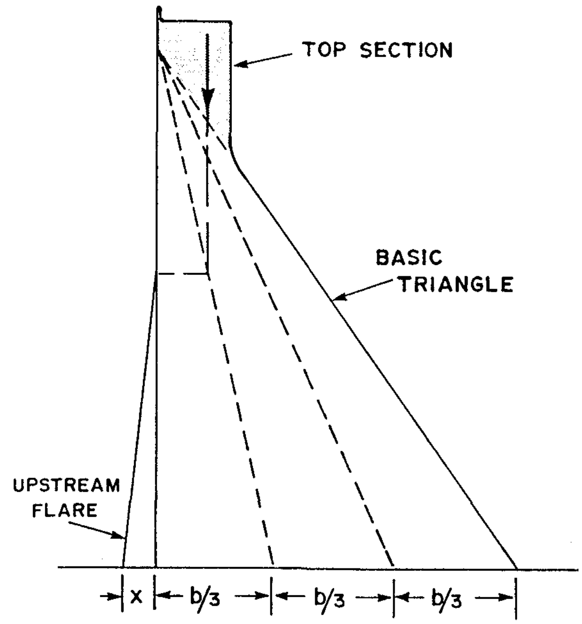

Figure 18. Upstream Flare

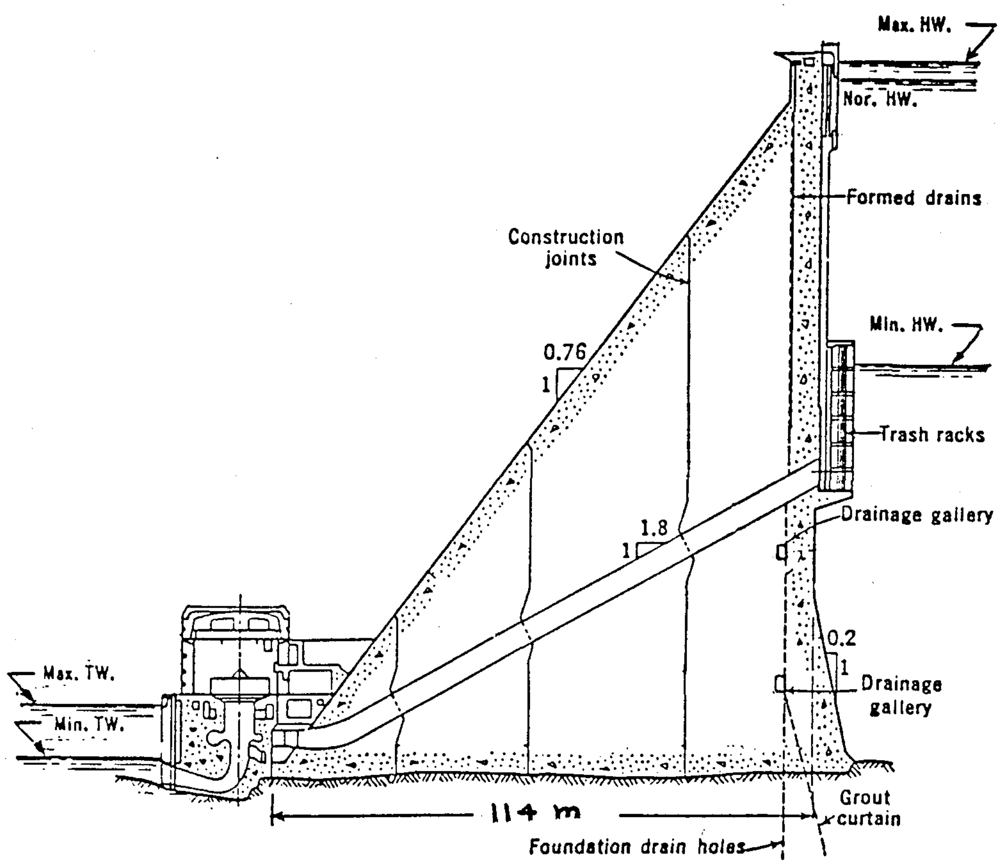

Figure 19. Cross Section for Fontana Dam, Tennessee

E. SINGLE ARCH DAMS

19. General

The single arch dam is also a solid concrete dam, but it requires only about one-fifth of the material that would be used for a gravity dam at the same site. Most of the hydrostatic load from the reservoir is transmitted by arch action into the abutments. The load is carried entirely in compression so steel reinforcement is not required.

The site requirements for the single arch dam are: (1) Firm unyielding rock abutments; and (2) a valley which is relatively narrow in proportion to its height. Ideally the span to height ratio for a single arch should be five or less, and in any case a value of ten is a maximum. A wide span is unsuitable for the single arch because the arch thickness would have to be greatly increased and the abutment loads would be excessive.

Any movement of an abutment is almost certain to result in failure of the arch. In this respect it is important to note that the arch dam must be treated as a complete unit in the stability analysis. The design is considerably more complex than for the gravity dam. The arch design is usually curved in elevation as well as in plan, and the radius of curvature may be constant or may vary with height. It may be necessary to use an unsymmetrical arch to suit the valley shape.

20. Arch Stresses

The stress equation for a submerged cylinder with an inner radius of r_i and an outer radius of r_o is

[30] $$f = p[r_o^2 + (r_i^2 r_o^2)/r^2]/(r_o^2 - r_i^2)$$

in which f is the tangential compressive stress at any radius r, and p is the external pressure acting normal to the cylinder wall. The maximum stress occurs on the inside wall, for which $r = r_i$, and since the wall thickness $t = r_o - r_i$, then [30] may be reduced to

[31] $$f_i = 2p\, r_o^2/[t(r_o + r_i)]$$

This is the equation for the tangential stress on the inside wall of a thick cylinder. If t is small compared to the radius, it may be assumed that $r_o = r_i$, and $f_o = f_i = f$ (average stress), and [31] reduces to the classical hoop stress equation for a thin cylinder

[32] $$f = pr_o/t$$

This equation gives results within 2% of [31] provided $r_o/t > 25$.

The cylinder theory cannot be applied exactly to an arch dam. In a complete cylinder the diameter shortens under load, the shape remains cylindrical, and [31] still applies. An arch dam is not a complete cylinder, and under load the span between the abutments will not shorten, but lengthen, or at best remain constant. The accompanying deflection of the arch will reduce the compressive stress on the intrados at the centre of the arch, but it will increase the compressive stress on the intrados at the abutments. Theoretically a somewhat thinner section could be used at the centre than at the abutments. Usually a constant thickness is used at one elevation, and hence the abutment stress governs the design.

Maximum abutment stresses at the intrados (f_{ai}), assuming no abutment yielding, have been calculated from elastic theory. These stresses are greater than given by the thin cylinder equation, but can be accounted for in design by multiplying the thin cylinder stress by a coefficient K, such that

[33] $$f_{ai} = K\, p r_o / t$$

The coefficient K depends upon both the central angle 2θ and the thickness ratio r/t, in which r is the mean radius of the arch. The relationship between these variables is shown by the family of curves on Figure 20. It is to be noted that the thin cylinder theory (i.e. K = 1) is approached for very large values of 2θ and r/t. Also, for values of K greater than two, tension will exist on the extrados at the abutment. The somewhat abrupt change in direction of the curves above the tension line is due to the assumption of negligible concrete strength in tension, and hence compressive stresses must be correspondingly increased to carry the bending moment.

Any shortening and accompanying flattening of the arch due to concrete shrinkage or temperature contraction will allow a further increase in the compressive stresses at the abutment intrados. This stress rise can be minimized by pressure grouting the vertical contraction joints in cold weather after the shrinkage has taken place. Temperature stresses of 700 to 1400 kPa are possible, depending on the central angle and thickness ratio.

The required arch thickness at any elevation h can be calculated from [33] and Figure 20. Given 2θ (say 120°) and span B between abutments, r_o may be solved from

[34] $$r_o = B / (2\sin\theta)$$

A trial value for t may be selected and r/t evaluated for use in Figure 20 (noting that $r = r_o - t/2$). Since $p = \gamma h$, the abutment intrados stress f_{ai} is solved from [33]. The temperature stress is added to f_{ai}, and this sum must not exceed the allowable concrete stress. Allowable concrete stresses for arch dams are usually in the range of 5000 to 8000 kPa.

21. Arch Design

The single arch dam may be designed as a constant radius dam, as shown in Figure 21. The upstream face is vertical and has the same centre of curvature at every elevation, and therefore appears as a portion of a cylindrical surface. The downstream face is inclined to account for the greater arch thickness required at lower elevations. The central angle 2θ varies from a maximum at the top to a minimum at the bottom where the chord or span is shortest.

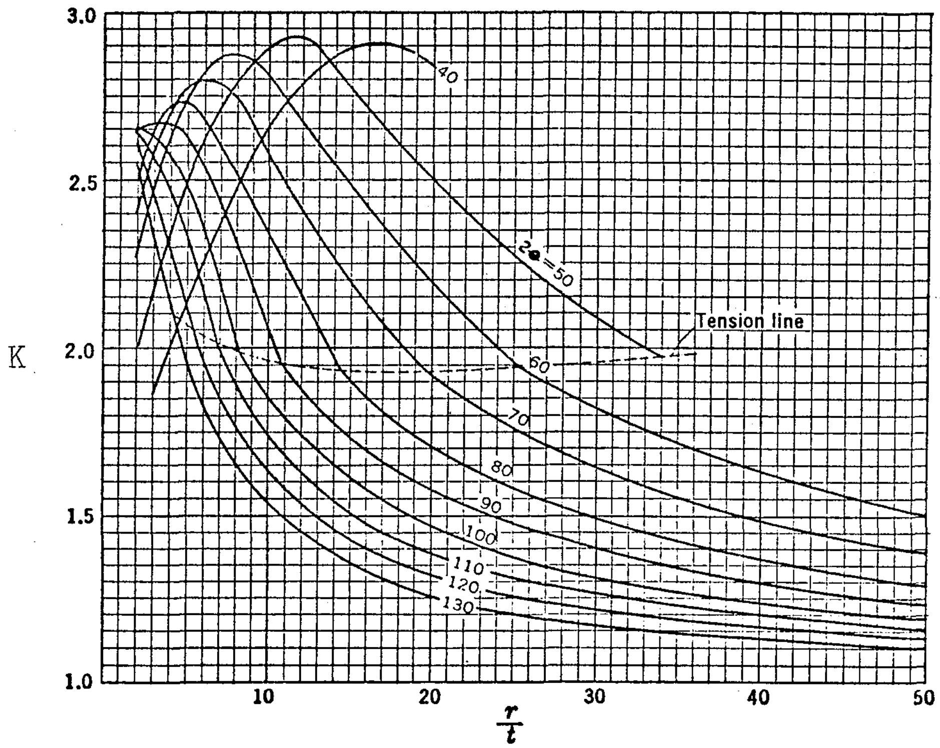

Figure 20 . Coefficient K for Arch Dam Abutment Stress

The constant radius single arch dam is not the most economical shape. In fact it can be shown by thin cylinder theory the least volume of concrete would result if the angle 2θ was 133° 34' at all elevations. The development is as follows:

Let B be the span or chord length between abutments, L the arc length of the arch, t the thickness of the arch, r_o the radius to the extrados, and 2θ the central angle. The cross-sectional area of concrete exposed at any elevation by a horizontal slice through the arch would be

[35] $$A = Lt$$

Since $L = 2\theta\, r_o$, $\sin\theta = B/2r_o$, and $t = pr_o/f$, then

[36] $$A = B^2\, p\, \theta/(2\, f \sin^2\theta)$$

At a given elevation, B, p, and f are constant. The area will be a function of θ alone and will be a minimum when $dA/d\theta = 0$. For $d/d\theta\, (\theta/\sin^2\theta) = 0$ the solution is $2\theta = \tan\theta$, for which $\theta = 0.37\pi$ radians, and the most economical central angle is 133° 34'.

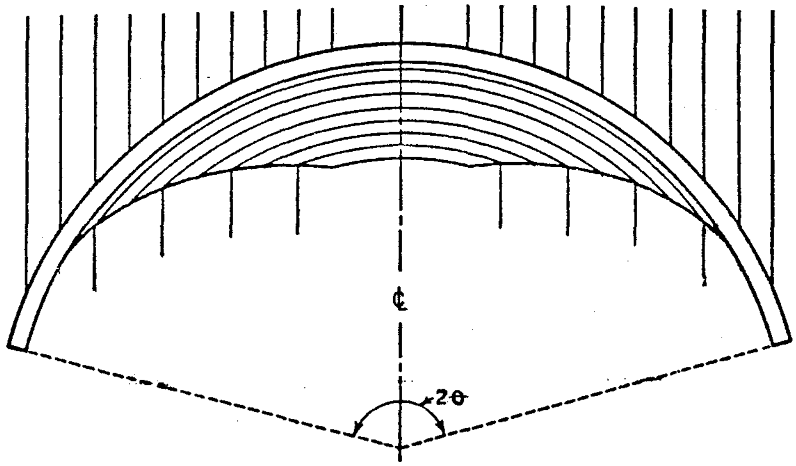

Figure 21 Constant Radius Arch Dam

The foregoing development leads to the concept of the constant angle arch dam, as shown in Figure 22. In this case the radius varies with height, being longest at the top and shortest at the bottom. The upstream face is curved both in plan and elevation.

Although the concrete volume may be reduced by 30% for the constant angle as compared to the constant radius dam, site conditions usually prevent full implementation of the constant angle method. To begin with, the most economical central angle according to the elastic theory for a thick arch is actually greater than 133° 34'. Yet maximum angles used in practice rarely exceed 110°. It is usually assumed, after Boussinesq, that the horizontal arches transmit the reaction into the rock abutment in such a manner that the stresses are contained in a 60° wedge, or 30° on either side of the line of action of the resultant. Therefore, in order to contain the stressed wedge within the rock mass, the maximum central angle for a valley with parallel sides would be 120°.

A second problem raised by the constant angle is upstream overhang. As shown in Figure 22, the crest of the dam in the vicinity of the abutments will actually overhang the base on the upstream side, and the section may be unstable with the reservoir empty. This can be taken care of by varying the central angle or using upstream buttresses.

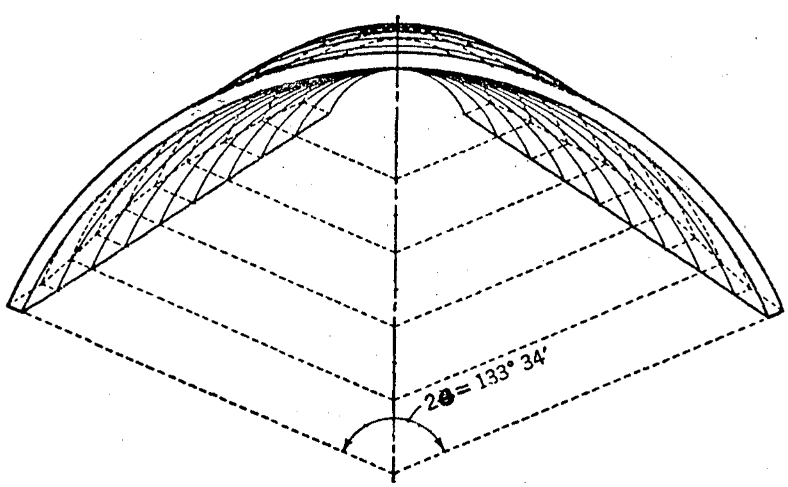

Figure 22. Constant Angle Arch Dam

The United States Bureau of Reclamation pioneered the trial load method of design. The total load is assumed to be carried jointly by arch action between abutments and cantilever action above the base. In the trial load method, the division of load between the horizontal and vertical strips is first estimated at various points on the face of the dam. Near the base the load is carried primarily by cantilever action, and near the top the load is almost entirely carried by arch action. The deflections of the arch and cantilever strips are calculated using the assumed division of load. If the deflections do not agree at each point, a new division of load is assumed, and if necessary, adjustments are made to the cross section of the dam.

More recently the membrane analogy has been used to determine the best shape for an arch dam. A flexible rubber membrane, with thickness proportional to the anticipated thickness in the actual arch dam, is placed across a model of the valley and reverse loaded. That is, lift forces are applied proportional to weight and a water load is applied to the downstream side of the membrane. The shape taken by the membrane is observed and used, with modifications, for the shape of the arch dam. Since the membrane cannot transmit any bending moment it carries the load in pure tension, and this shape should produce pure compression when the loads are reversed. It must be obvious that the bulge in the membrane under the influence of the water load will be of double curvature and not single curvature. This natural shape is more closely approximated by the constant angle dam than the constant radius, and this further explains why the former is more economical. More effective use is made of the load carrying capacity of the concrete in the dam.

F. OTHER TYPES OF DAMS

22. Buttress Dams

There are several types of buttress dams--the flat slab and buttress, diamond head buttress, round head buttress, and multiple arch. These dams may be broadly classified as hollow dams because they all have inclined upstream faces, and depend to a large extent upon the vertical component of water force on the upstream face for their stability. The amount of material is between one third and one fifth of the amount required for a gravity dam.

The flat slab and buttress dam was the first type of hollow dam to be used. The idea originated with a Norwegian, Ambursen, who patented the idea in 1903. A typical section of an Ambursen dam is shown in Figure 23.

The flat slab and buttress dam is a reinforced concrete structure. Reinforcement is required to resist tension in the relatively thin deck and footing. The buttress is in compression. In some respects the design is similar to a counterfort retaining wall, except that the load is on the opposite side. The load determination and stability analysis is the same as for the gravity dam. It is frequently necessary to add gravel ballast inside the dam in order to secure adequate sliding resistance.

The flat slab and buttress dam is suitable for poorer foundations. Bearing pressures are smaller and more uniformly distributed than for a gravity dam of the same height. The dam is somewhat more flexible and can accommodate a small foundation movement without distress. These advantages must be weighed against the extra cost of forming, reinforcing, higher cement content, and shorter useful life.

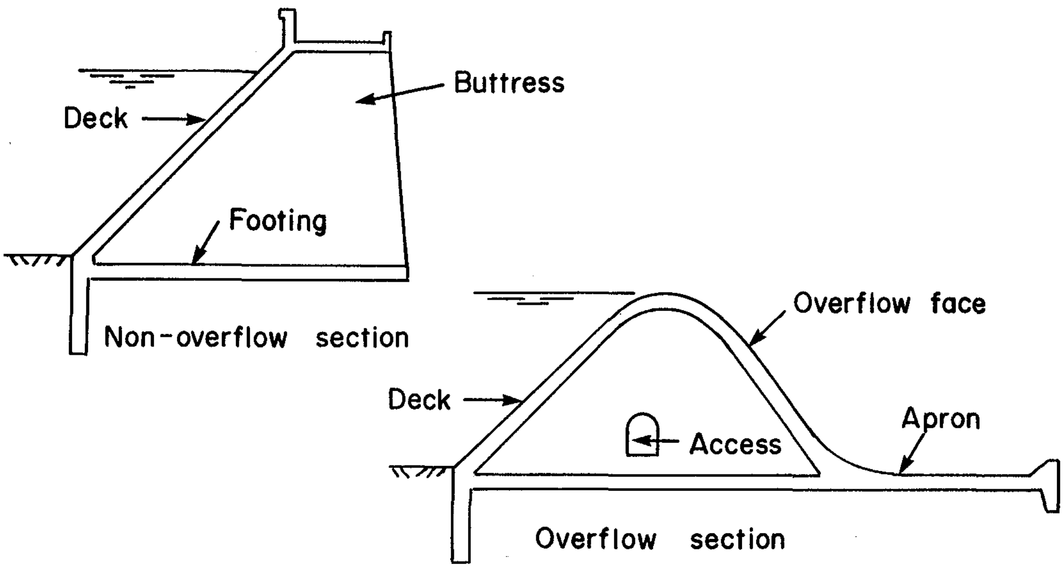

Figure 23. Flat Slab and Buttress Dam

The diamond head and round head buttress, as shown in Figure 24, are an adaptation of the hollow dam idea in such a way that tension is largely eliminated. Each buttress, which in itself is rather massive, supports a thick concrete head facing the reservoir. The head is so proportioned that the resultant water force can be transmitted into the buttress without tension. A series of diamond or round heads make up the inclined upstream face of the dam. More concrete is required than for the flat slab design.

A multiple arch dam may be used where the valley is too wide for a single arch. Basically, several arches are placed side by side. Since the load on each arch must ultimately be carried by the foundation, a buttress is required at each haunch to transmit the load to the base.

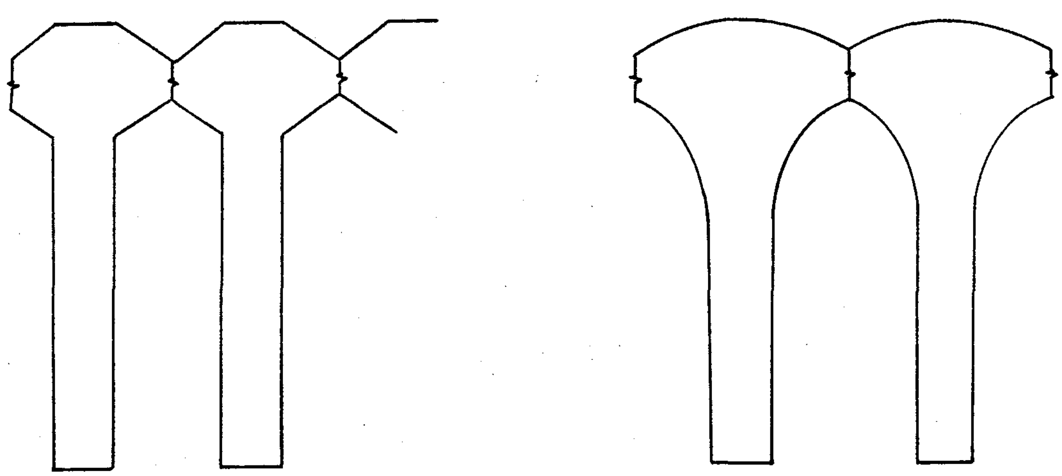

Figure 24. Diamond and Round Head Butresses

A diamond head buttress dam and multiple arch dam are shown in Figures 25 and 26. Manic 5, at 224 m, is the highest multiple arch in the world.

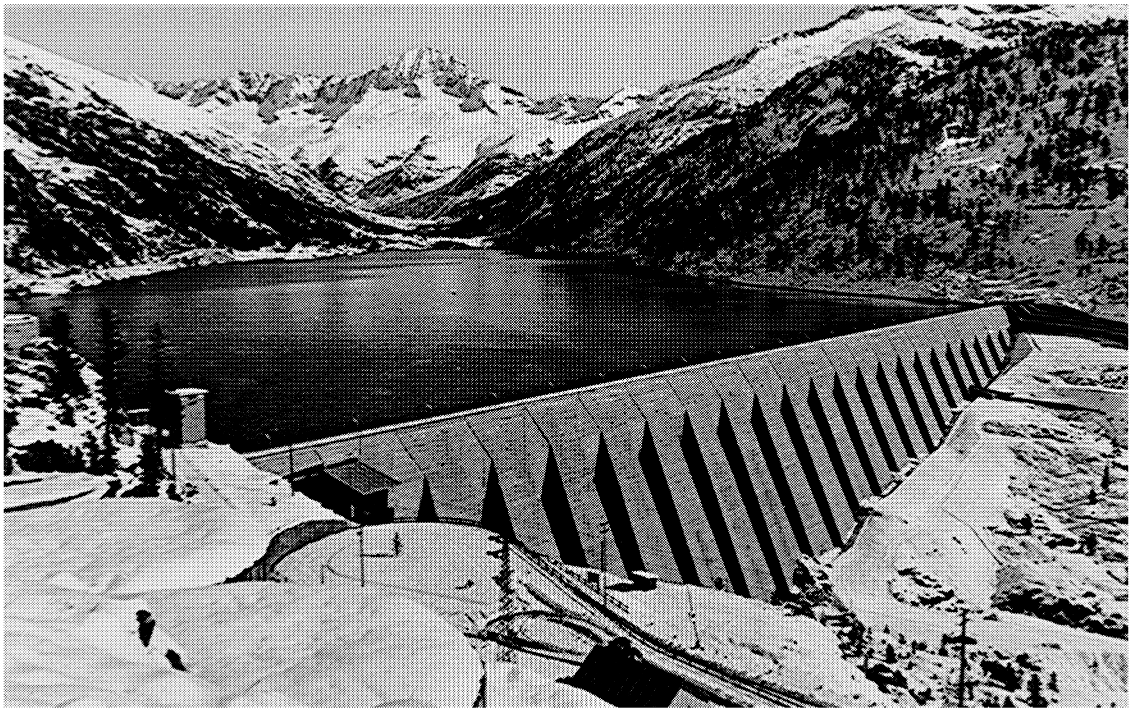

Figure 25. Diamond Head Buttress Dam, Malga Bissina, Italy

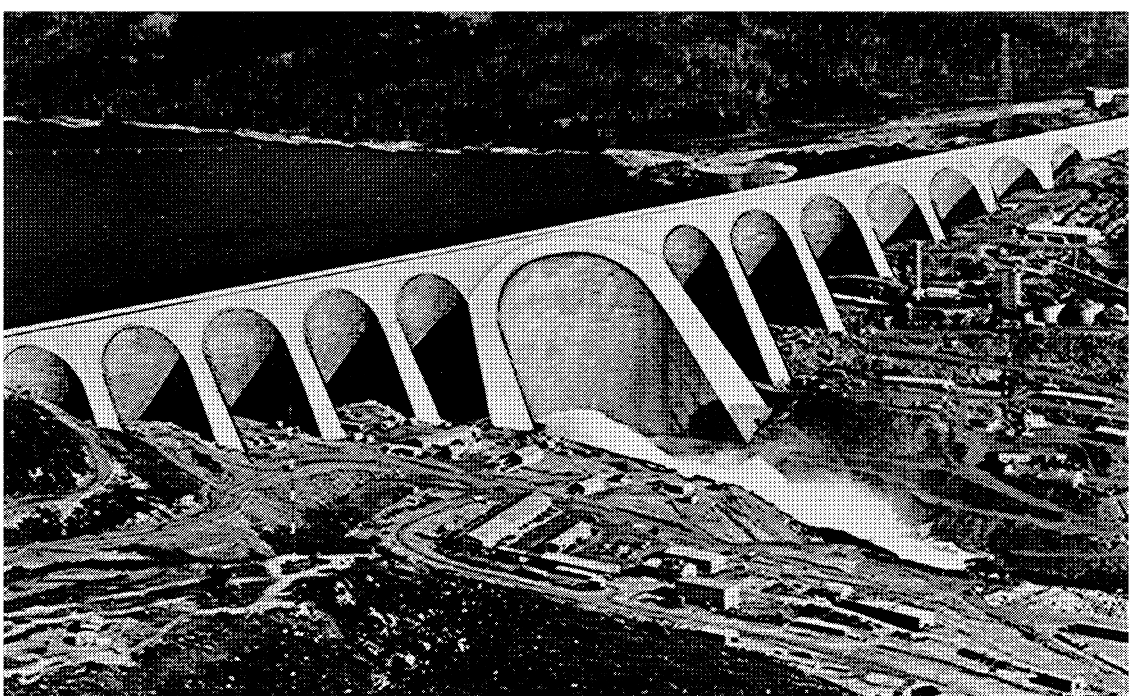

Figure 26. Multiple Arch Dam, Manic 5, Quebec

Timber or steel may be used as construction materials for low head dams. The principles of design are the same as for other hollow dams. The inclined upstream face is supported above the base by struts. The struts substitute for the buttresses used in the other types. A typical cross section of an A-frame type timber dam is shown in Figure 27.

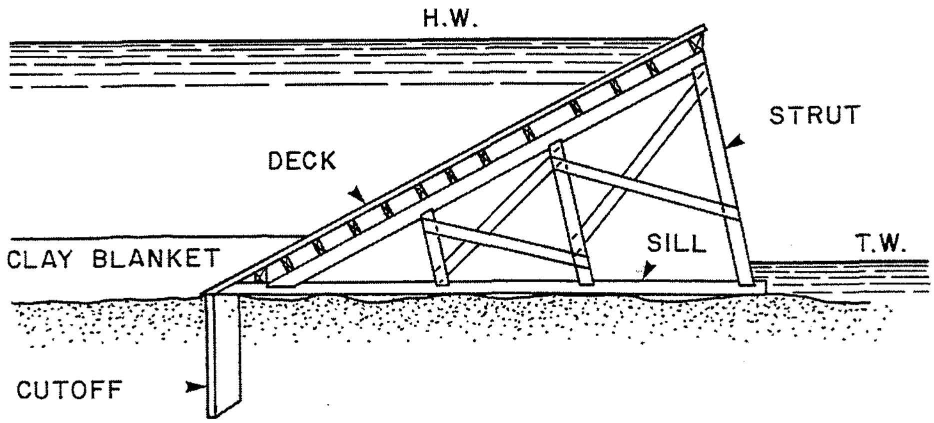

Figure 27. Timber Dam

23. Earth Dam

The earth dam is common in plains areas where foundations are unsuitable for any type of concrete dam. The dam has flat side slopes so the load is spread over a wide base area. The volume of materials is ten to twenty times more than for a concrete gravity dam of the same height. A typical cross section for a zoned earth fill of medium height is shown in Figure 28.

Rock riprap is required on the upstream face of the dam to resist wave action. This should extend over the operating range of the reservoir. An impervious zone, usually a compacted clay type material, is placed in the upstream portion of the dam for watertightness. The downstream portion is constructed of granular material. Granular materials usually have a higher shear strength and, therefore, will allow some reduction in the volume of materials required. They are also relatively free draining. A rock filter drain may be used at the downstream toe of the dam. This helps to lower the hydraulic grade line and reduce pore pressures in the dam and foundation. A cutoff, which is really an extension of the impervious zone of the dam, may be used to control underseepage. The cutoff material is placed in a core trench excavated below the river bed to rock or other impervious strata. Dewatering may be required to construct the core trench.

As an alternative to a cutoff, an upstream blanket may be used to increase the length of the seepage path. The blanket consists of a thick layer of impervious material which is connected to the impervious zone of the dam and extends upstream over the river deposits. A large volume of material may be required for the blanket; however, blanket construction does not require any excavation or dewatering, and is usually more economical than a cutoff if the depth to rock is great.

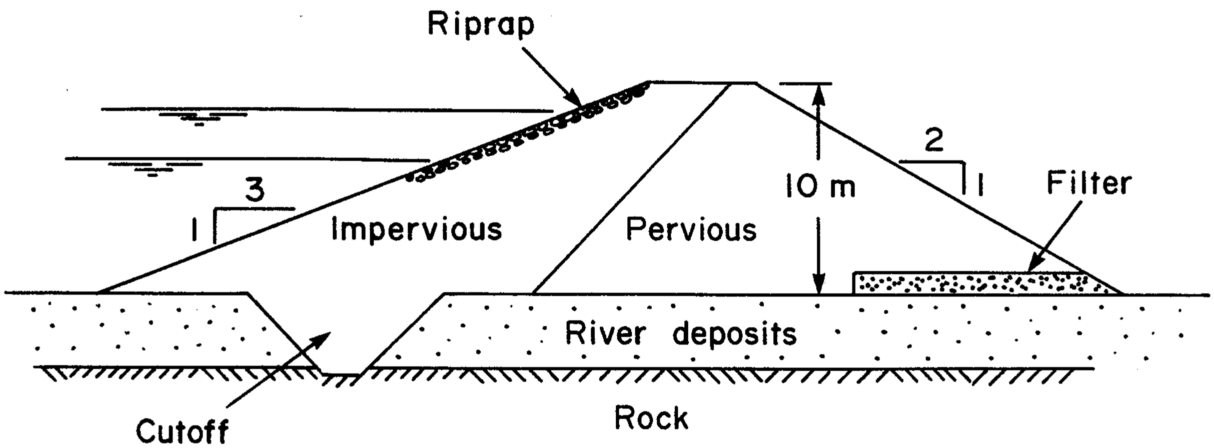

Figure 28. Zoned Earth Fill Dam

Stability analysis for the earth dam requires investigation of shear stresses (sliding) and bearing pressures. Overturning is not possible. Shear strengths and compressive strengths must be determined for the foundation and for the material to be used in the dam. The effects of long term consolidation and settlement must be taken into account. Although consolidation usually increases shear strengths, there is a possibility that shear cracks will develop due to differential settlements in the vicinity of a steep abutment. If a large settlement is expected, this must be allowed for in selecting freeboard.

24. Rock Fill Dam

Rock fill dams are used in preference to concrete dams when an abundance of suitable rock is available nearby, or in remote areas where the cost of cement would be high. Where a rock fill dam can be built, the foundation would usually support some type of concrete dam as well, so the choice is a question of economics. The side slopes of the rock fill must be somewhat flatter than the angle of repose for the rock, so more volume of material is required than for a concrete gravity dam. On the other hand, the volume is only one quarter to one half of the volume required for an earth dam. Concrete, steel, or clay must be used for a central core, or as a membrane on the upstream slope, in order to achieve watertightness. Figure 29 shows a rock fill dam.

Side slopes which must be used for fill type dams are dependent upon the shear strength of the material in the dam. For example, side slopes of 1 vertical to 1 1/2 horizontal can be used for rock fill dams, and this is independent of the height. However, for clay type embankments, the side slopes must be progressively flattened as the height increases. This may be explained as follows: Shearing resistance for a unit width on a potential sliding surface is calculated by

[37] $$SR = N \tan \phi + cL$$

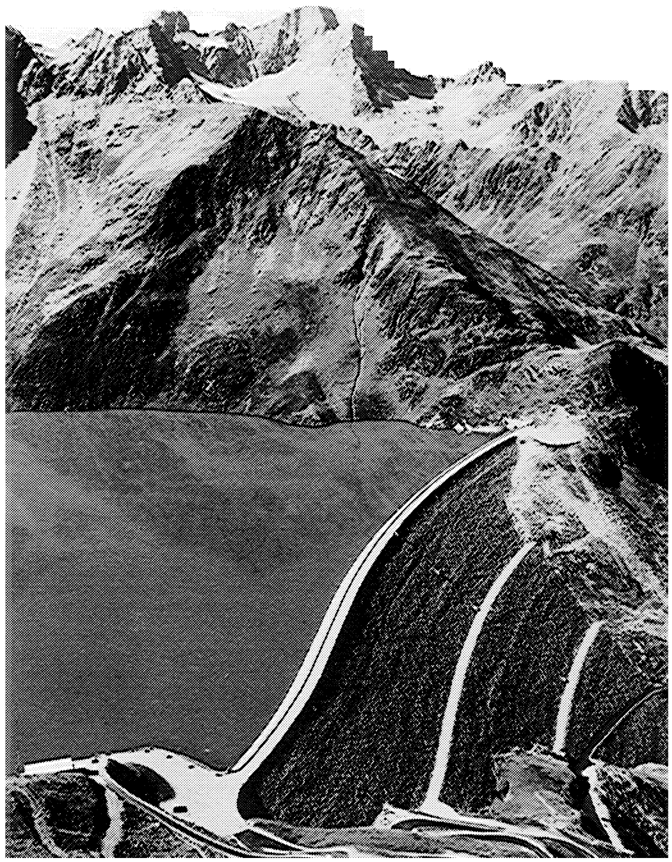

Figure 29. Rockfill Dam, Finstertal, Austria

in which N is the effective force normal to the sliding surface (due to the effective weight), ϕ is the angle of internal friction for the material, c is the cohesion, and L is the length of the sliding surface. In a clay dam the angle ϕ is small, occasionally zero, so the principal sliding resistance is due to cohesion. The horizontal water load, or sliding force, varies as the square of the height of the dam, so the side slopes must be flattened with increasing height to increase L and develop the necessary sliding resistance. For gravel or rock, the value of c is zero and typically $\phi = 35°$, so the sliding resistance is due to internal friction alone. Since the effective weight of the dam varies as the square of the height, the necessary sliding resistance is obtained without requiring any flattening of the side slope as the height increases.

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PROBLEMS

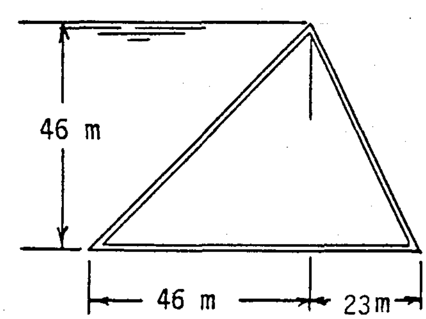
1. Show that [12] will result from [11] with units converted.

2. A rectangular shaped reservoir with an average depth of 9 m is 19.3 km long, 4 km wide, and faces a hollow concrete dam with a sloping 1 1/2 horizontal to 1 vertical upstream face. Calculate the freeboard allowance based on an over water wind of 80 km/h.

3. Prove for a gravity dam that if the resultant falls within the middle third of the base, there will be no tension in the section.

4. The basic section of a solid gravity concrete dam is triangular in shape with a vertical upstream face 120 m high. Calculate the width of base required to satisfy overturning criterion when the water level is at the top of the section. Assume 100% uplift intensity factor, and no tailwater.

5. The sketch shows the cross-section of a hollow reinforced concrete dam. The base has been made wider than normal to reduce the bearing pressure and to help equalize the bearing pressure over the base. Neglecting uplift and tailwater, calculate the vertical bearing pressure at the toe and also at the heel. Consider the weight of the section to be 25% of the weight of solid concrete, and assume the weight to be at the centre of gravity of the cross-section.



6. The basic section of a solid gravity concrete dam has a 30.5 m vertical upstream face and a base width of 24 m. Calculate the vertical pressure on the solid material of the foundation at the toe and at the heel under the following conditions: 30.5 m depth of water in reservoir, no tailwater, 60% uplift intensity factor. Is the dam safe against "overturning"?

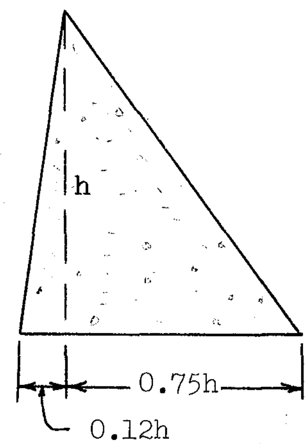
7. A certain concrete gravity dam may be assumed to be triangular in section with a vertical upstream face. The height is 120 m and the base width is 90 m. With water 3 m from the top, check for overturning and calculate the intensity of maximum stress in the foundation and concrete at the toe. Neglect uplift and tailwater. Assuming no resistance by shear, how low may the coefficient of friction be before the dam will slide on foundation?

8. Calculate the slope required for the upstream face of a hollow concrete dam in order to satisfy sliding criterion for stability. Use $f = 0.67$, $S_f = 2$, and consider external water pressure as the only load.

9. (a) The basic section of a concrete gravity dam has a 100 m vertical upstream face and a base width of 70 m. Calculate the vertical pressure on the foundation at the toe and heel when there is 100 m depth of water in the reservoir and no tailwater.

   (b) With uplift included, using an uplift intensity factor of 60%, is the dam safe against overturning?

10. The triangular section shown exactly satisfies the Middle Third Rule, reservoir empty. What apparent earthquake intensity factor was used in the design?



11. (a) Explain the purpose of the upstream flare on a gravity type dam.

    (b) Explain fully the procedure in determining the height and width of the upstream flare.

12. In Figure 20, the position of the tension line corresponds very closely to a value of $K = 2$. This value is particularly significant, and would be exactly 2 for a linear stress distribution. Why?

13. For a single arch dam calculate the required arch thickness at a depth of 30 m, given $B = 90$ m, $r_o = 55$ m, allowable stress 5,500 kPa, and temperature stress 700 kPa.

14. Is the double curvature arch dam constant radius or constant angle? Which requires the least volume of concrete?

15. Explain why constant side slopes may be used for a rock fill dam independent of the height.

CHAPTER II

SPILLWAYS

A. GENERAL

1. Purpose of Spillways

All storage dams must be protected by a spillway. The spillway is intended to discharge the excess river flow, during times of flood, in such a manner as to insure the safety of the dam and appurtenant works at all times. Although storage dams are relatively high dams and impound a significant storage volume, it is never economically feasible to build the dam high enough to store the low frequency high discharge floods in the reservoir. Some provision must be made to get the excess flow safely through, over, or around the dam. A spillway is used for this purpose.

2. Types of Spillway

The type of spillway used in a given case depends upon the kind of dam, the magnitude of the spillway design discharge, the topography, and the nature of the foundation. Flow may be allowed to pass directly over a portion of a concrete gravity dam. Such a spillway is called an overflow spillway. In some cases an overflow spillway may be used with a concrete buttress dam or an arch dam. Often, however, a separate structure is used with an arch dam. In case of space limitations, as for a narrow arch, a side channel spillway or shaft spillway may be used. A separate structure must always be used for a rock fill or earth fill dam. In this case, a concrete chute spillway is most common. On smaller dams a simple drop inlet or box inlet spillway may be more economical than the other types.

Regardless of the type, every spillway has three basic components -- a crest section, a conveyance section, and a discharge section. The crest section is the inlet to the spillway, situated at or near the reservoir level. From the crest the flow must be conveyed in a chute or tunnel to a level near the natural river level on the downstream side of the dam. The spillway terminates in a discharge section, from which the flow re-enters the channel. If necessary, the discharge section may also be the means of dissipating the excess kinetic energy of the flow.

B. PRELIMINARY CONSIDERATIONS

3. Selection of Design Flow

Calculations for the component parts of a spillway are based on the assumption of a particular discharge called the design flow. In certain types of structure, such as for irrigation or water supply, the maximum flow is subject to control and can be predicted with good accuracy. In these cases it is common to take the design flow equal to that maximum flow. In spillways for river dams the design flow is less definite. In the first place, the probable maximum inflow discharge must be calculated from hydrometric records, sometimes with large possibility for error. Secondly, the design flow may be taken at less than the maximum flow. Finally, different safety factors may be used to design different parts of the structure. In the final analysis, the value which is used for design depends upon the dependability of the hydrological analysis, a consideration of the size and importance of the dam, the consequences of a failure, and the philosophy of the design.

The spilling design flow for a very large important dam, where the consequences of a spillway failure would be catastrophic, may be taken as the required outflow from the reservoir corresponding to the probable maximum flood (PMF). The control section should be able to safely handle this discharge; however, the stilling basin may very well be designed for a smaller discharge. The philosophy of this approach is based on the fact that it is possible to have a partial failure of stilling basin without seriously jeopardizing the dam or remainder of the spillway over the duration of a single flood. The design involves the calculated risk that the PMF will not occur in the life of the structure, thereby permitting a substantial reduction in the capital cost of the stilling basin.

Where circumstances permit, is often economical to take advantage of an emergency earth spillway, or bypass channel, to discharge part of the outflow during a PMF event. The spillway design flood can be reduced, for example, to the 0.1% frequency flood, and when this flood is exceeded, the difference can be discharged over the emergency spillway. Of course, it is necessary to do a rate of erosion study on the emergency spillway to ensure that the erosion channel which develops will not jeopardize the project. For example, a bypass channel constructed through silt and fine sand would be expected to erode very rapidly, starting at the downstream end, and unless it is very long, might be expected to breach through the reservoir during a large flood event.

On smaller, less important dams, it is not usually economically feasible to design for the PMF. For example, a spillway for a small 6 m earth fill at a remote site might be designed for a 2% flood simply because it would be less expensive to replace the fill than to build a spillway to protect it.

4. Evaluation of the Tailwater Rating Curve

The establishment of the stage discharge relationship at the spillway site, known as the tailwater rating curve, is an important item in the design for the discharge section. The safety and economy of the design both depend upon the accuracy of this evaluation.

As a general rule, the stilling basin is designed for a discharge exceeding the maximum on record. The probable river stage for this high flow must be computed. This computation may require a water surface profile analysis based on measurements of the elements of the river channel, as determined from a field survey. These elements are the shape, area, slope, alignment, and roughness of the channel.

If the channel is very irregular there may be considerable uncertainty in the selection of the elements. In these cases any available stage discharge records or high water marks should be thoroughly analyzed for possible leads to the effective values of the slope and channel roughness. It is advisable to calculate upper and lower limits of the stage using a range of values which conservatively bracket the most probable value. In addition to the above factors, it may be necessary to give separate attention to such items as the presence of islands, junctions, roads, bridges, culverts, possibilities of ice jams, and future degradation of the downstream channel. The stilling basin must be designed to operate satisfactorily over the complete range of tailwater so determined.

5. Spillway Layout

The spillway layout must be determined before any detailed design is made. The layout study frequently involves the investigation of several alternatives. The important considerations in selecting the layout are the foundation conditions, and amount and type of excavation, the alignment of the approach and discharge channels, and the location of the spillway with respect to the dam and other structures (conduit or power-house or both). The foundation conditions alone often require a detailed study, with consideration given to such items as faults, seepage, swelling, rebound, settlement, weathering, frost heaving, allowable bearing pressure and allowable shear stress.

6. Hydraulic and Structural Design

The detailed hydraulic design to establish the dimensions of the structure can be made once the design discharge, tailwater rating curve, and layout have been determined. This part of the design is concerned with the lengths, widths, wall heights, elevations, shapes, and positions of the various components of the structure. Actually, some of these dimensions are usually established approximately in a preliminary hydraulic design, since obviously studies on the outflow hydrograph and layout must be closely related to the hydraulic design.

The structural design is the final phase of the complete spillway design. This involves assessment of the magnitude and distribution of the ice, earth, and water pressures, calculation of the loads, selection of permissible steel and concrete stresses, and hence calculation of the concrete thickness and steel design. This phase also includes the design of a number of very important details, such as the drainage system, joints, cutoffs, anchors, gates, hoists, and bridges.

C. OVERFLOW SPILLWAY

7. Description

The overflow spillway is combined with, and a part of, the main dam. In a concrete gravity dam this part is often referred to as the overflow section, in contrast to the non-overflow section. The components of the overflow section are the crest, spillway face, and the energy dissipating device at the bottom. A hydraulic jump basin, submerged roller bucket, or free flip bucket may be used for this purpose, depending upon the circumstances.

8. Crest Design

It is common practice to design a pressure free profile for the crest shape of large high head spillways. A pressure free profile is one on which the pressure at design flow is neither positive or negative, but essentially at atmospheric pressure. This is achieved by shaping the crest to fit the underside of the nappe as it would occur when discharging freely from a sharp crested weir at the design head. The X and Y coordinates for this shape are given in Table 1. These coordinates give the nappe shape for a high weir such that reservoir conditions exist upstream and the total energy line coincides with the reservoir surface. The total head measured above the sharp crest is defined as H_s, as shown in Figure 1.

TABLE 1
VERTICAL WEIR LOWER NAPPE COORDINATES
(Negligible velocity of approach)

X/H_s	Y/H_s	X/H_s	Y/H_s	X/Hs_s	Y/H_s
0.00	0.0000	0.35	0.1060	0.90	-0.140
0.05	0.0575	0.40	0.0970	1.00	-0.215
0.10	0.0860	0.45	0.0845	1.20	-0.393
0.15	0.1025	0.50	0.070	1.40	-0.606
0.20	0.1105	0.60	0.032	1.60	-0.850
0.25	0.1120	0.70	-0.016	1.80	-1.132
0.30	0.1105	0.80	-0.074	2.00	-1.451

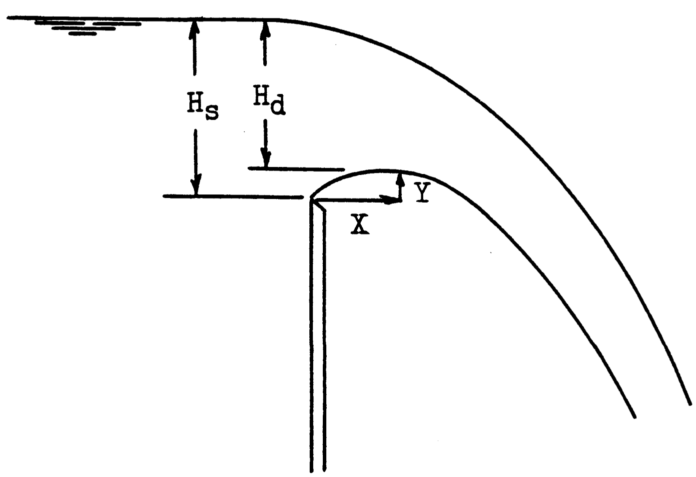


Figure 1. Crest Shape for an Ogee Weir

The discharge for this weir is given by the well-known equation for a sharp crested weir

[1] $$Q = C_d \, 2/3 \sqrt{2g} \, L \, H_s^{3/2}$$

in which Q is the discharge in m^3/s, L is the net crest length in m and C_d is the coefficient of discharge. The standard value of C_d for a sharp crested weir, as originally determined by Francis in 1850, is 0.622, and hence the discharge equation becomes

[2] $$Q = 1.837 \, L \, H_s^{3/2}$$

The value 1.837 is called the weir coefficient, and has the units of $\sqrt{g}$.

When the area under the nappe is filled in with solid material, the crest of the weir is effectively 0.112 H_s above the origin of the coordinates for the curve. This point is H_d below the reservoir surface. If H_d is used in the discharge equation, the corresponding weir coefficient will be greater than 1.837 and can be computed by equating 1.837 L $H_s^{3/2}$ to CL $H_d^{3/2}$, where C is the weir coefficient corresponding to head H_d. Noting that $H_d = (1 - 0.112)H_s$, then $C = 1.837/(1 - 0.112)^{3/2}$, from which

[3] $$Q = 2.195\, L\, H_d^{3/2}$$

The actual coefficient may be reduced slightly due to the additional frictional resistance which results when a solid boundary is substituted for the space under the nappe. This correction may amount to 1 or 2% in the usual case - the smaller value being applicable to larger spillways where crest friction is relatively small.

Now it is evident that if the space under the nappe is filled in with solid material, then it will only fit the nappe for the one head for which it was designed. At a lower head the nappe will be supported, positive pressures will develop on the crest, and the coefficient will decrease. At a head greater than the design head negative pressures will develop and the coefficient will increase. The effect on the coefficient in terms of the ratio H/H_d is given in Table 2, in which H_d is the head used for the design of the crest shape and H is the head on the crest at any time.

Table 2 also gives the drop in pressure head, P/γ, as the head exceeds the design head. The accompanying increase in discharge coefficient is often desirable in that a given flow can be discharged with less head or less crest length. However, there is a limit to how low the pressures on the crest may be allowed to go. Although the cavitation limit may be near -10 m (depending on the atmospheric pressure and water temperature), it is not desirable to approach this limit in hydraulic structure design. The actual boundary pressures are transient, continually fluctuating above and below the average value, and the range of fluctuation increases as the flow velocityincreases. If the concrete surfaces are unusually true, smooth, and dense, a design pressure of -5 m may be acceptable. In general, however, the minimum pressure should be limited to about -2 m.

TABLE 2
VARIATION IN COEFFICIENT OF DISCHARGE WITH HEAD

H/H_d	C	$(P/\gamma)/H_d$
0.2	1.825	--
0.4	1.941	--
0.6	2.041	--
0.8	2.123	--
1.0	2.195	0.00
1.1	2.228	-0.15
1.2	2.261	-0.31
1.3	2.294	-0.48
1.4	2.322	-0.65
1.5	2.349	-0.83

Table 2 shows that negative pressures on the crest develop rapidly as the head increases above the design head H_d. If a limit of -2 m is used for P/γ, it will be found that H/H_d values significantly greater than unity cannot be allowed unless the values of H_d are relatively low.

The spillway crest may be controlled or uncontrolled. The uncontrolled crest is set at the reservoir full supply level (FSL) and discharges freely whenever the reservoir rises above it. The controlled crest is gated, and is set with the reservoir FSL near the top of the gates. The uncontrolled crest has the advantage of automatic discharge. However, if the permissible pool rise or crest length is limited for any reason, it may be necessary to use the gated crest in order to develop sufficient head on the weir crest to pass the required discharge. The discharge, head, and width are related by an appropriate weir equation. A trial and error procedure may be required to determine the best combination of these variables, and whether the crest is to be free or gated. In selecting the width of the control section, some consideration must also be given to the width of the other components of the spillway. It is often desirable to have parallel sidewalls throughout the length, so the width selected will be determined by the requirements of the discharge section as well as the control section.

Example 1:

Calculate the net spillway crest length needed to discharge 565 m^3/s with a maximum head of 4 m.

A value of H_d = 3 m may be used, since, from Table 2 for H/H_d = 1.33, $(P/\gamma)/H_d$ = -0.53, or P/γ = -1.59 m, which is within the allowable limit of -2 m.

Hence C = 2.302 and

$$L = Q/CH^{3/2} = 565/(2.302 \times 4^{3/2}) = 30.68 \text{ m}$$

9. Spillway Face

The face of the overflow section is part of the natural downstream face of the gravity dam, and therefore the slope of the overflow section and non-overflow section are the same. The shape given by the coordinates in Table 1 is made tangent to the slope of the dam. This slope is usually between 0.7 and 0.8 horizontal, to 1.0 vertical. When the shape is properly positioned with respect to the downstream face of the dam, it may be found that the origin for the coordinates does not lie in the same plane as the vertical upstream face of the non-overflow section. In this case the upstream face of the overflow section may be offset locally, as shown in Figure 2. Provided the height "a" is equal to or greater than 1/2 H_d, the offset will have no effect on the discharge coefficient.

Parallel sidewalls, or training walls, as shown in Figure 3 and Figure 4, are used to contain the flow discharging over the crest. These walls are constructed of reinforced concrete and are designed as cantilevers out of the mass section. The flow may be significantly greater than the theoretical clear water depth due to bulking of the flow caused by air entrainment. This fact may have to be considered in selecting sidewall height.

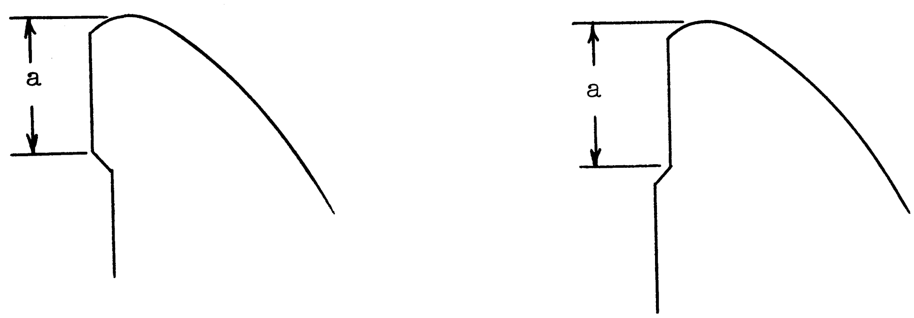


Figure 2. Offset Crest Details

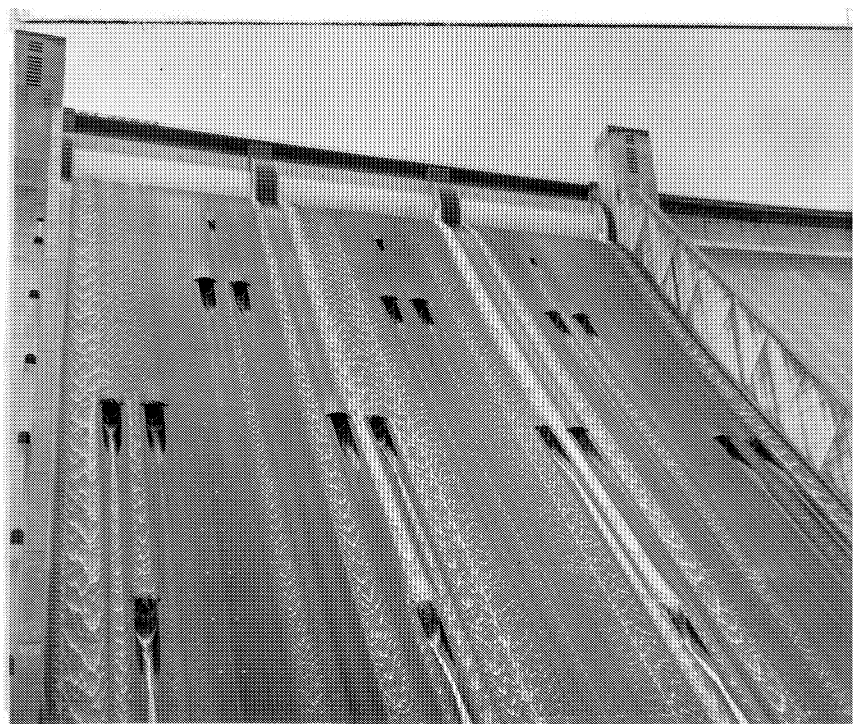


Figure 3. Overflow Spillway Face, Shasta Dam, California

Data for computing depths and velocities for flow down steep slopes is still inconclusive. The theoretical velocity, neglecting losses, is given by

[4] $$V_t = \sqrt{2g\,(H + h)}$$

in which H is the head on the spillway crest and h is the height of the drop from the crest down to the elevation of the point where the velocity is calculated. The actual velocity V_a will be less than the theoretical due to friction losses. A reasonable estimate for the actual velocity may be determined from Figure 5, adapted from the U.S. Bureau of Reclamation, which gives the ratio of V_a/V_t for the various heads and drop heights. This figure applies only to steep face overflow dams, and not to other types of spillway.

Figure 4. Sluice Outlet Discharging Into Flip Bucket, Pine Flats Dam, California

10. Flip Bucket

The flip bucket or "ski jump" spillway, as shown in Figures 4 and 6, has application primarily to the case of relatively inerodible formations. The spillway overflow is discharged into atmosphere completely above the tailwater level. Where it can be used, the flip bucket design permits a substantial reduction in cost, as the high walls required for a hydraulic jump basin or submerged roller bucket are eliminated. The objective of the flip bucket is to throw the jet as far downstream away from the dam and spillway as possible. Since the entire energy of flow must be dissipated downstream from thespillway, it follows that there is considerable attack on the bed and banks of the discharge channel. Some of the energy is dissipated along the trajectory in the atmosphere, but most of the energy is dissipated by turbulent mixing in the scour hole which forms downstream.

The significant variables in design are the elevation, radius and angle of the bucket. The lip of the bucket may be placed at any elevation above maximum tailwater; however, if the material is erodible, the lowest possible setting is desirable in order to throw the jet a maximum distance. If the bucket was located only halfway down the slope, the jet would land much closer to the toe of the slope. This approach would be satisfactory only if material in the impact area is rock.

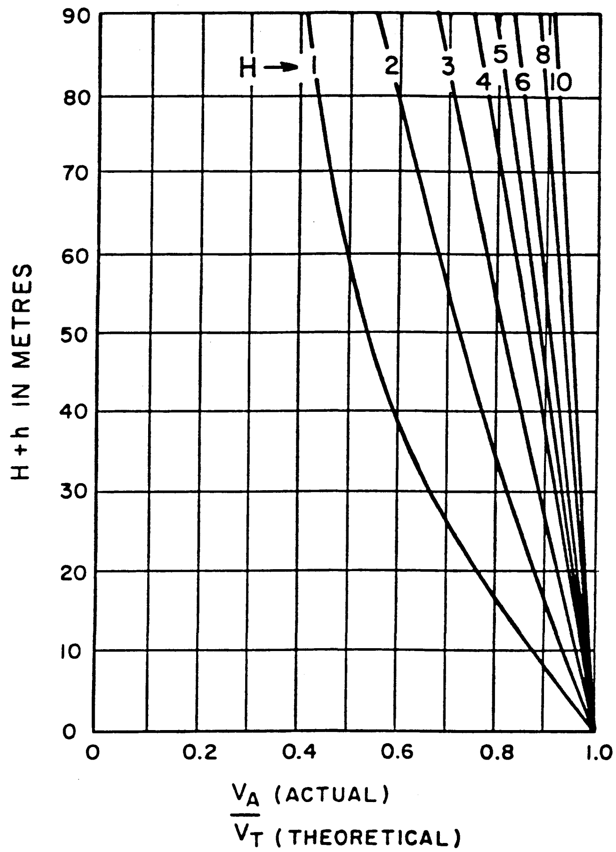


Figure 5. Spillway Velocities for Steep Slopes (0.6:1 to 0.8:1 horizontal to vertical)

Partial submergence of the bucket lip by the tailwater must be avoided. Model tests have shown that partial submergence will interfere with the free passage of the jet, and may result in adverse hydraulic conditions and severe channel scour. Of course, if the submergence is large enough the hydraulic action will become similar to the case for the submerged roller bucket, for which the performance will be good. However, it is not possible to design a bucket for both types of operation on the same spillway. Evidently the bucket should either be completely submerged or completely free.

The radius of the bucket must be large enough to effectively guide the flow around the curve of the bucket. This can be done with a radius of not less than four times the depth of flow entering the bucket. This depth must include any bulking effect on the flow due to air entrainment.

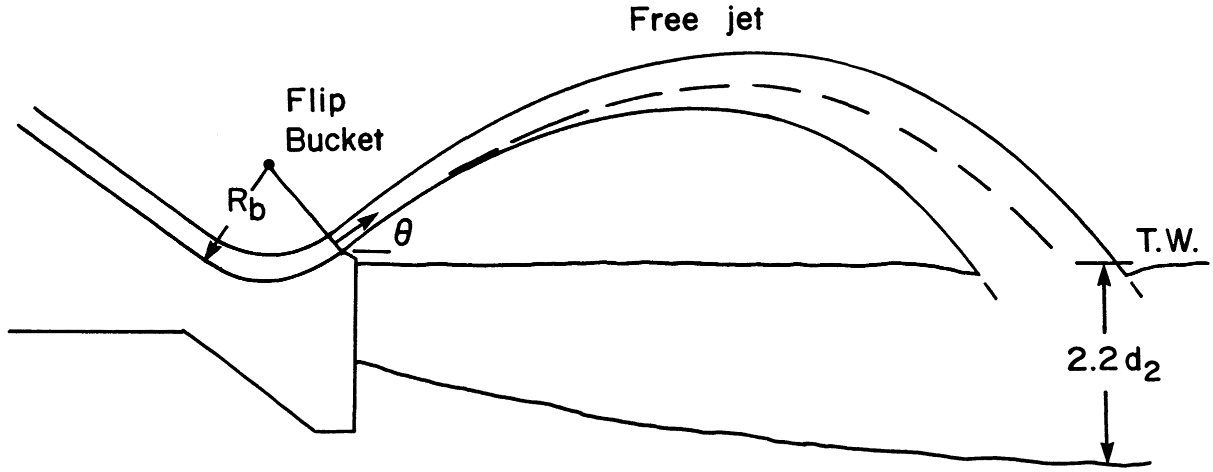


Figure 6. Flow Pattern for Flip Bucket

It is desirable to use the minimum allowable radius for the bucket since a longer radius gives both a longer structure and a greater depth of bucket between the bottom of the bucket and the lip. The disadvantage of a deep bucket is the effect on lower discharges. Ordinarily a pool will form in the bucket when discharge commences. This will be sufficient to produce a small hydraulic jump in the bucket itself, with the discharge spilling over the end into the discharge channel. The discharge may have to be increased to 10 - 15% of design discharge before the jump is washed out and the jet allowed to flip properly.

The designer may have to take into account the additional hydraulic load on the bucket due to the centrifugal force of the flowing jet. In the absence of a flow net analysis or model test, the approximate unit pressure around the curve is given by

[5] $$p = \rho V^2 d/R_b + \gamma d = \gamma d[V^2/(gR_b) + 1]$$

in which d is the equivalent clear water depth, and V is the mean flow velocity. Model tests have shown that [5] is exact if $R_b \geq 4d$ and the total angle change is equal to or greater than 90 degrees.

The angle of the bucket lip determines the angle of impingement of the jet at the tailwater level, and also, for a given velocity, the distance which the jet is thrown. If the bucket lip is at the tailwater level, then, theoretically, the maximum horizontal throw would occur for a lip angle of 45 degrees and would be equal to twice the velocity head of the jet at the bucket lip. Model test results show reasonable agreement with theory, although the jet does not follow a well defined trajectory. This is due to the variation in velocity distribution both in the vertical and across the chute. The flow near the walls is not thrown as far as the flow over the central portion of the chute, with the result that the side elevation of the discharging jet is fan shaped, giving the illusion of great thickening. By measurement, the maximum throw at the centre is about equal to twice the velocity head, based on mean velocity. Hence, most of the discharge lands short of this distance.

Vertical scour is largely dependent upon the vertical velocity component of the impinging jet, and lateral scour upon the horizontal component. Hence a 45 degree angle of impingement gives a good compromise between lateral and vertical scour potential in the jet. In completely erodible (cohesionless) material, the scour hole depth below the tailwater level may be double the depth required to form a hydraulic jump. This raises the very difficult problem of designing an adequate downstream cutoff, and is the principal reason why this type of design is usually limited to the case of relatively tough foundations.

It must be pointed out that air resistance modifies both the throw and the angle of jet impingement on the prototype. Although jet dispersion is slight on a model, it is an important factor on the prototype. As jet dispersion increases along the path of the trajectory, air resistance also increases. The result is that themaximum height is reached sooner, and the jet drops more steeply, than the theoretical. The maximum throw and 45 degree angle of impingement then occur for some lip angle less than 45 degrees. Many spillways have been designed using an angle of only 30 degrees. This angle requires somewhat less construction material, the jet will start to flip at a smaller discharge, and the trajectory performance is almost identical.

<u>Example 2:</u>

By what percent is the theoretical horizontal throw reduced if a 30 degree lip angle is used for a flip bucket instead of 45 degrees?

The x and y coordinates for the jet trajectory are given by

$$x = V_x t \text{ and } y = V_y t - 1/2\ gt^2$$

Eliminating t, there results

$$y = x \tan\theta - [gx^2/(2V^2\cos^2\theta)]$$

This equation may be solved for x at the top of the trajectory, noting that dy/dx = 0 at that point, or

$$x = \sin\theta \cos\theta\ V^2/g$$

Hence the horizontal distance to the highest point of the trajectory is zero for $\theta = 0°$ or $\theta = 90°$. By setting $dx/d\theta = 0$, it may be shown that x will be a maximum for $\theta = 45°$.

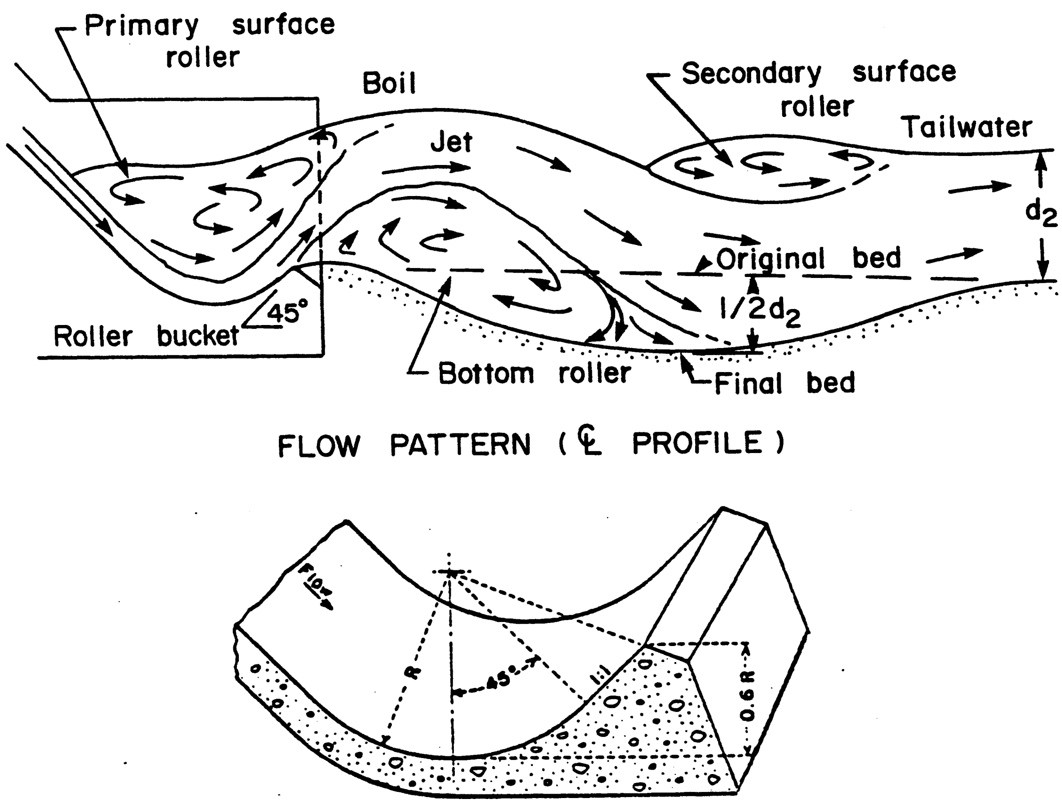
$$\text{For } \theta = 45°,\ x = V^2/2g$$

$$\text{For } \theta = 30°,\ x = 0.868\ V^2/2g$$

Therefore, theoretically, the horizontal throw for a 30° lip angle is 13% less than for 45°. Practically the difference is even less because air resistance has a greater influence on the higher trajectory from the 45° lip.

11. Submerged Roller Bucket

The most effective means of dissipating excess kinetic energy of flow is by the hydraulic jump. Accordingly a hydraulic jump stilling basin is used wherever the channel into which the spillway discharges is readily erodible. However, since concrete gravity storage dams are invariably founded on rock, the discharge channels often have appreciable natural erosion resistance. In this case it may be possible to use a less expensive treatment than a hydraulic jump basin. One of these alternatives is the submerged roller bucket, consisting of a smooth constant radius upturned bucket at the base of the overflow section. The submerged roller bucket was extensively investigated in 1933 for Grand Coulee Dam, where it has been used successfully to dissipate energy at the rate of 22 GW during maximum flood. The flow pattern produced by the roller bucket is shown in Figure 7. The path followed by the jet results in the creation of several rollers which absorb the excess energy of flow and cause the jet to expand. The pronounced upward deflection of the jet prevents scouring and undercutting near the lip. In fact, the reverse flow along the bottom, associated with the bottom roller, often results in a build up of material near the toe of the structure. Except for the primary surface roller in the bucket, the energy is dissipated downstream from the structure. If the bottom is erodible, the scour hole downstream may ultimately reach a depth of 50% of d_2. This is considerably greater than is the case for the hydraulic jump basin, but it is quite acceptable in some situations.



BUCKET DETAILS

Figure 7. Submerged Roller Bucket

The design variables for the submerged roller bucket are the lip elevation, bucket radius and bucket angle. The lip of the bucket should be located at a depth of d_2 below the tailwater, where d_2 is the sequent depth as calculated for the jet velocity and depth on the spillway face at the tailwater level. The value d_2 is the hydraulic jump depth (see Section 24). Although the hydraulic action shown in Figure 7 is not a true hydraulic jump, it has been found that the required depth corresponds closely to the hydraulic jump depth.

The radius of the bucket must be at least four times the flow depth

[6] $$R_b \geq 4d$$

and the bucket angle should be 45 degrees. A shorter radius will not effectively turn the flow. A 45 degree bucket angle produces the best combination of rollers.

The United States Bureau of Reclamation has developed a slotted bucket which consists essentially of a row of teeth with large spaces in between, as shown in Figure 8. This bucket performs somewhat better hydraulically than the solid bucket, but negative pressures have been found to occur on the downstream face of the teeth, and the design is more susceptible to cavitation damage.

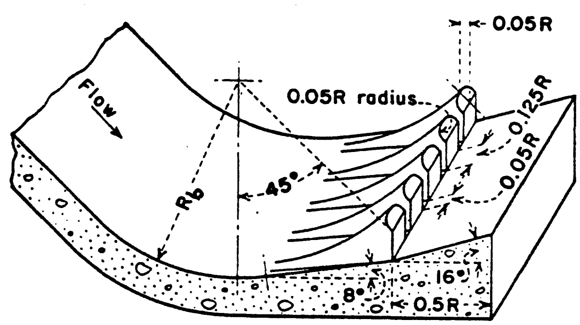


Figure 8. Slotted Roller Bucket

The flow picture shown in Figure 7 is two-dimensional and indicates the nature of the flow pattern and scour as it would occur over the central region of a wide spillway. At the position of the sidewalls, however, a third dimension effect will develop, in that a large eddy will form in the discharge area adjacent to the jet. In the absence of downstream wingwalls a bottom cross current will form under the jet, consisting of a lateral flow which is drawn into the space under the jet by the strength of the jet itself. The cross current will interfere with the proper formation of the bottom roller, and deep scour may develop in the channel immediately at the end of the bucket. This is illustrated in Figure 9, which shows the contoured scour pattern produced in an erodible bed on a model of a submerged roller bucket. In this case the structure was so narrow that the third dimension effect extended in from each sidewall a sufficient distance to scour the channel bed at the toe of the structure over the entire width. The scour depth shown as -3 units represents 65% of d_2.

In order to minimize this scour tendency, it is necessary to largely eliminate the cross current. This can be done by using right angled downstream wingwalls, as shown

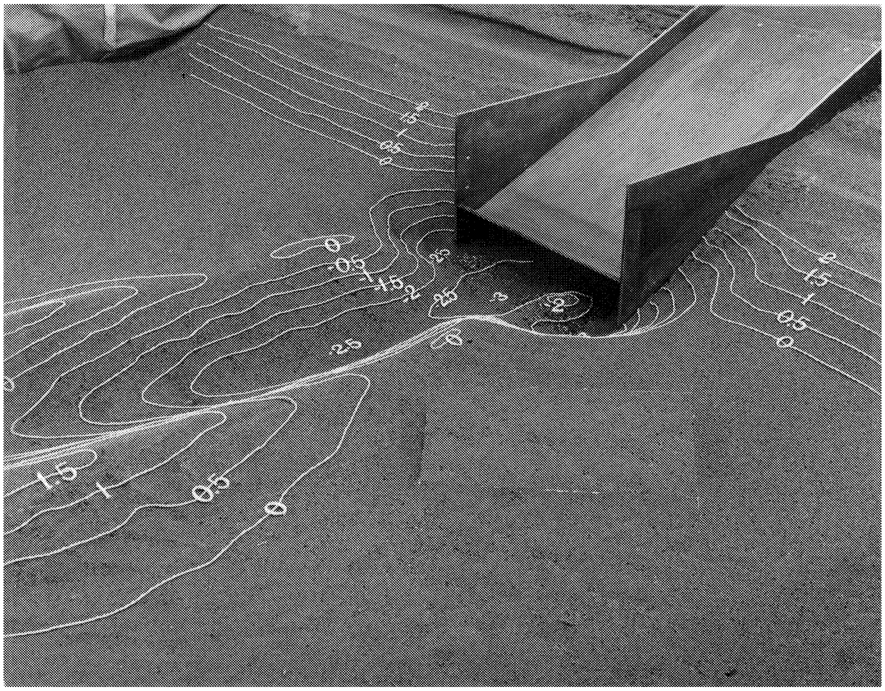


Figure 9. Scour Pattern for Roller Bucket - No Wingwalls

Figure 10. Scour Pattern for Roller Bucket - Right Angled Wingwalls

by Figure 10. The tops of the wingwalls are sloped down at 45° to save material since it is only the bottom current and not the surface current that must be dealt with. These wingwalls effectively intercept the flow path that would otherwise be followed by the bottom current, and interference with the bottom roller is localized at the sides of the structure. End scour is eliminated over the central portion and limited to 0.25 d_2 in the vicinity of the wingwalls. Bed scour still occurs downstream from the structure, but even this is reduced when wingwalls are used, as shown by a comparison of Figures 9 and 10.

D. CHUTE SPILLWAY

12. Description

Chute spillways are separate spillways used commonly with earth or rockfill dams. The spillway may be located in one of the abutments, or even at a site along the reservoir which is remote from the dam. The major components of the chute spillway are the control section, the chute, and the discharge section, as indicated in the definition sketch Figure 11.

The control section is that portion at the head end of the spillway which admits the discharge to the chute. It is comprised of an upstream cutoff, wingwalls, abutment walls, floor, and weir. It may or may not have gates and piers, depending upon whether the discharge is to be controlled or uncontrolled.

The chute is that portion of the structure which conveys the flow from the control section to the discharge section. It is comprised of a floor slab and sidewalls and a suitable subfloor drainage system.

The discharge section is that portion of the spillway which releases the flow to the downstream channel. The most common discharge section for large spillways in erodible material is the hydraulic jump stilling basin, comprised of a level floor with baffle blocks and end sill, and parallel sidewalls. This type of design is intended to dissipate the excess kinetic energy due to the drop within the confines of the structure and release the flow in a relatively tranquil state.

13. Entrance

There must be an approach section between the upstream wingwalls and the weir. This is required to contain the higher velocities at the entrance within the confines of the structure, and to establish two-dimensional flow at the weir. According to model tests this can be done with a minimum entrance length equal to twice the maximum head on the weir; i.e.,

[7] $$L_e = 2H_{max}$$

In some cases this length may have to be increased for a controlled crest in order to accommodate the gates, piers, and bridge, or in case increased length is necessary for sliding stability of the crest section.

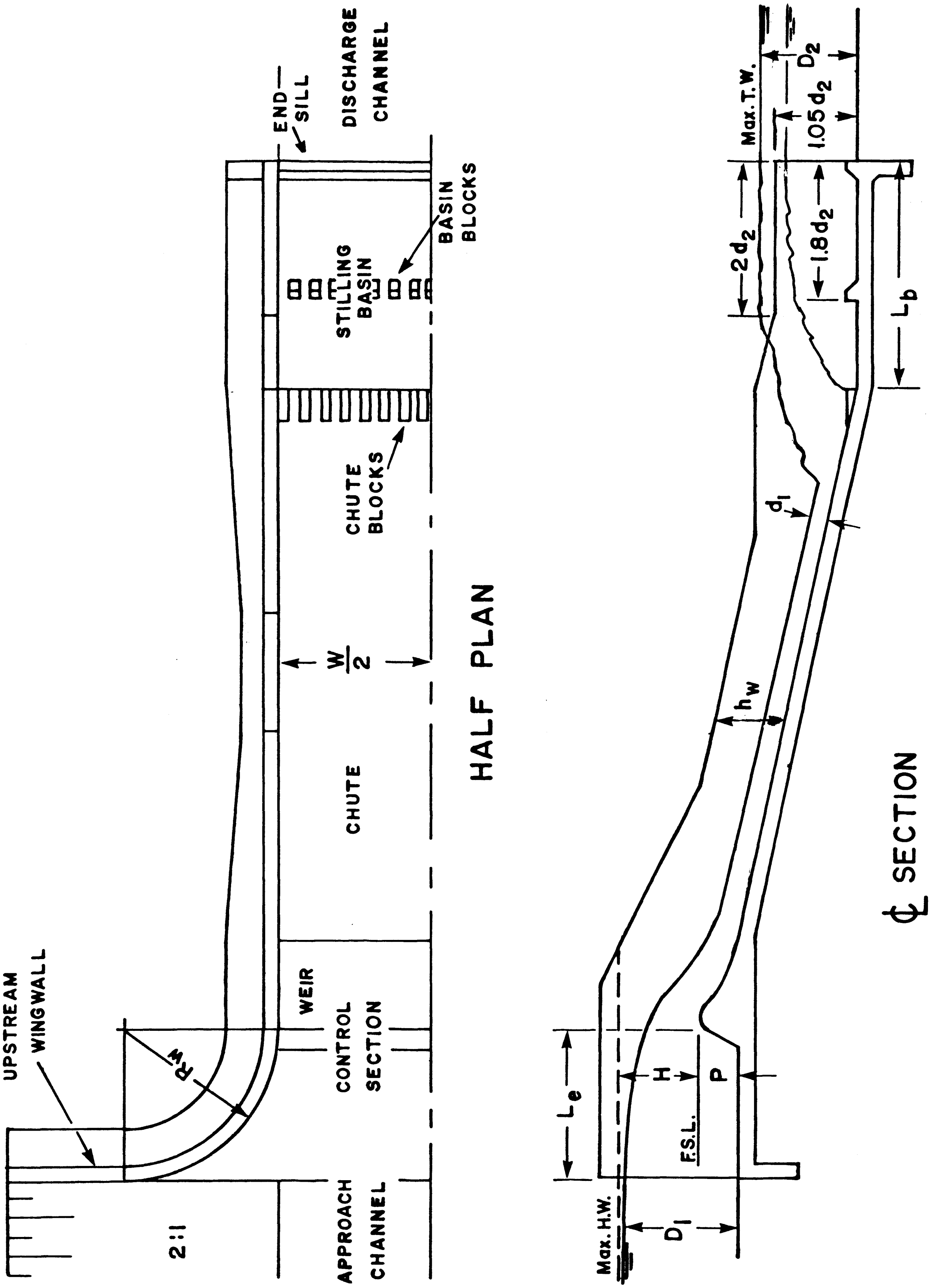


Figure 11. Definition Sketch for a Chute Spillway

It is desirable to connect the wingwalls and the abutment walls of the control section with a simple curve. Such a curve is more pleasing in appearance than a 90° corner, and it eliminates entrance losses and reduces the flow concentration and scour potential at the inlet. A radius equal to twice the design head is satisfactory for this curve. A typical spillway entrance is shown in Figure 12.

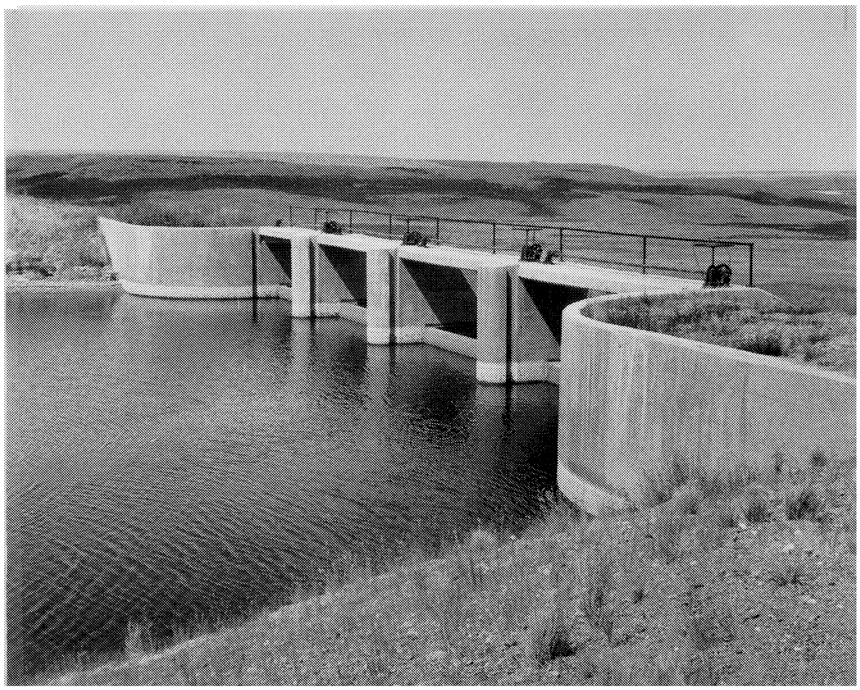


Figure 12. Spillway Entrance, Gouvenor Dam, Saskatchewan

The upstream cutoff is a very important component of the control section. This cutoff is all that stands between the reservoir and the underside of the sloping chute downstream from crest. If the cutoff cannot be extended to rock or other relatively impervious strata, it must at least be deep enough to reduce underseepage to a small amount. Excess seepage can lead to troubles with piping, uplift, and possible failure of the chute slab.

The upstream wingwalls are simply an extension of the cutoff laterally into the abutments flanking the control section. The seepage resistance around the end of the wingwalls should be just as great as the seepage resistance under the vertical cutoff. Neglect of this factor has resulted in some failures. The designer must avoid two-dimensional thinking when tracing the possible seepage paths.

14. Crest Design

It is evident that a weir with high coefficient gives a more economical design because the crest length can be reduced for the same head and discharge. In this regard it has been common practice to use an ogee weir. The name is derived from the fact that the weir profile follows an S-shaped or ogee curve. The initial part of the curve fits the underside of the free nappe and is tangent to the reverse curve comprising the bucket or fillet at the base of the weir.

In the case of high overflow dams, the weir height is the same as the dam height, and the velocity of approach is negligible. In the case of chute spillways, the abutment wall height must include the weir height P, measured above the bed of the approach channel. In order to economize on wall height, P is often made less than the design head H_d, and the velocity of approach is significant. This affects the shape of the nappe and the weir coefficient at design head. If $H/P \Rightarrow 0$, $C = 2.195$ as in Section 8. If the ogee weir is eliminated, $H/P \Rightarrow \infty$, and the control is essentially that of a broad crested weir, for which $C = 1.705$. Hence, the weir coefficient at design head must steadily decrease as H_d/P increases. This trend is indicated in Table 3. Data for the ogee weir with a 45 degree sloping upstream face is also included. This shape has a higher coefficient for low weir heights and it is frequently used for that reason.

TABLE 3
WEIR COEFFICIENT FOR AN OGEE WEIR AT THE DESIGN HEAD

H_d/P	Vertical Upstream face C	Sloping 45° upstream face C
0	2.195	2.150
1	2.140	2.140
2	2.057	2.085
3	1.952	1.985
4	1.848	1.897
5	1.759	1.820

The decrease in the weir coefficient may be attributed to two causes. In the first instance, a higher velocity of approach tends to produce a flatter nappe and a smaller Y coordinate to the highest point on the underside of the nappe. This fact alone would decrease the coefficient to less than the ideal 2.195. The second factor reducing the coefficient is related to the development of positive pressure at the base of the ogee weir on the downstream side. This positive pressure develops on the curved fillet which turns the nappe horizontal. If the weir height is low the zone of influence of this positive pressure will extend up to the crest of the weir itself, thereby further reducing the coefficient.

A desirable upper limit for the H_d/P ratio is about 4. At values of 5 and greater the flow in the approach section will be so near critical depth that an unstable surface may result. Also the weir coefficient will be so near the value for a broad crested weir that little would be gained by using the ogee weir at all.

The most comprehensive data on ogee weir profiles is contained in the 1948 Bureau of Reclamation Boulder Canyon Project Report, Bulletin 3, Studies of Crests for Overfall Dams. These reports can be used to determine weir profiles and discharge coefficients under a variety of conditions. Coordinates for some weir profiles are given in Table 6 of Chapter 3.

15. Piers

Most large spillways are of the controlled or gated type and the control section is used as the crossing point for a highway or railroad. Piers are required to support the gates and the roadway.

The required pier thickness t may be estimated from

[8] $$t = KB\sqrt{H_g}$$

in which B is the span between piers, and H_g is the gate height. The factor K varies between 0.05 and 0.08 for most designs. The pier thickness according to [8] will be more than would be necessary to carry the known loads based on a "balanced design" in reinforced concrete. However, in hydraulic structures, it is desirable to have considerable inertia in all water handling components. There are unknown loads due to vibration, impact of floating ice or debris, and unequal gate operation. The pier thickness must also accommodate slots for emergency bulkheads and for sliding gates, or for pin anchorage in the case of radial gates. As sliding stability is often critical, the weight of the piers and superstructure is an asset in providing sliding resistance.

In order to avoid flow contractions and interference at the weir crest, the nose of the piers should be semi-streamlined and situated 1.0 H upstream from the origin of the coordinates for the weir crest. A projectile shaped nose is ideal from a hydraulics standpoint, but the pointed nose is subject to damage unless it is protected by a steel plate. A semi-circular nose of radius 1/2 t is recommended for unprotected pier noses. The semi-circular nose is hydraulically efficient and the bow wave assists in deflecting debris around the pier.

The pier tails are in supercritical flow downstream from the weir. Shock waves, often referred to as "rooster tails", develop downstream from each pier, but these are local and so distributed as to be of little consequence. It is impractical and uneconomical to attempt to eliminate these waves by streamlining the pier tail. The pier tails should be square, as in Figure 13.

The crest length used in the discharge equation for the ogee weir must be net crest length L, where L = nB, in which n is the number of spillway bays. Since there is one less pier than the number of spans, the width of spillway between abutment walls is given by

$$W = nB + (n - 1)t \qquad (9)$$

The important point here is that the unit discharge q, in $m^3/s/m$ is equal to Q/L for flow over the weir, but equal to Q/W for the flow in the chute and stilling basin.

16. Chute Width

In short spillways the width of the chute is governed by the width of the control section and stilling basin. Frequently, all three parts are made equal in width to avoid the need for any transitions. Such a spillway is parallel sided throughout.

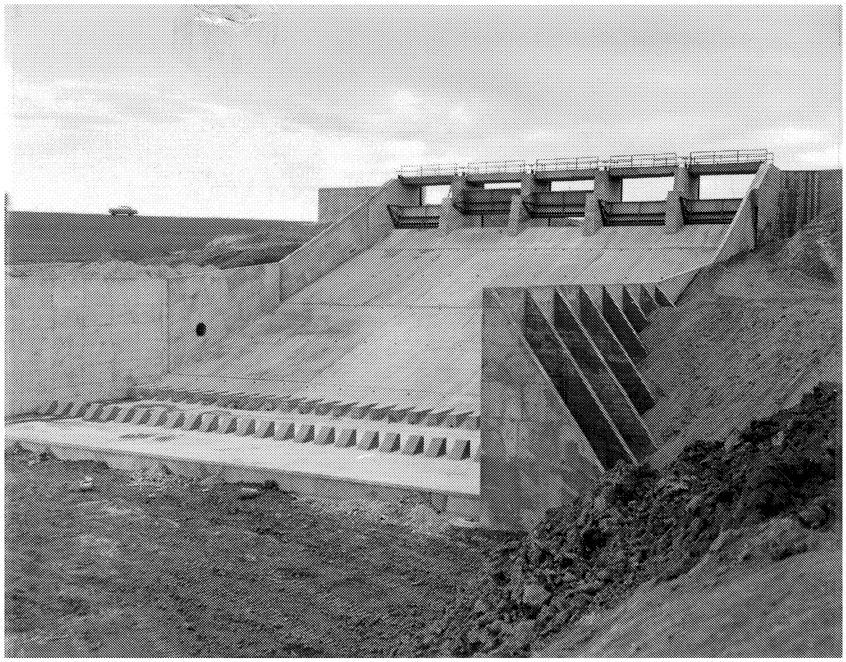


Figure 13. Chute Spillway, LaFleche Dam, Saskatchewan

In order to reduce costs for very long spillways, the chute width may be selected independently. The most economical cross section may be found by trial, whereby flow depths, and hence wall heights and excavation quantities are calculated for various widths. The chute width so determined may be considerably less than either the width of the control section or stilling basin. In this case, a transition from the control to the chute, and from the chute to the stilling basin, would be required. Transitions for flow at supercritical velocities cannot be precisely calculated from theory owing to the great number of variables which have an effect. These variables are the initial width, depth and velocity of flow, the length, slope and roughness of the chute, and the final width at the basin. Such a design should always be tested on a hydraulic model, as the adverse development of shock waves may exceed the freeboard in the chute or cause an asymmetrical jump in the stilling basin.

17. Chute Velocities and Depths

A determination of the flow velocity and depth in the chute is required for the design of vertical curves and wall heights. Velocity and depth are also required at the end of the chute in order to calculate the size of the hydraulic jump. Although this does not affect the design of the chute itself, it is a very important phase of the design, since the design of the entire discharge section is related to the size of the hydraulic jump.

The normal procedure to determine the chute velocity involves calculation of the water surface profile down the chute by a step method starting with the known velocity and depth at the start of the chute. This initial depth may be computed from the known head, discharge, and weir height. Since the energy loss due to friction over the ogee weir can be neglected, the specific energy at the base of the weir on the downstream side may be taken as H + P + Y, in which Y is the drop in floor elevation from the upstream to the downstream side of the weir, if any. This is numerically equal to the sum of the depth plus the velocity head, from which

[10] $$E = H + P + Y = d_1 + q^2/(2gd_1^2)$$

Equation [10] is a cubic, having 3 solutions for d_1, one of which is negative. The smaller positive value for d_1 is the supercritical flow depth on the downstream side of the weir, and may be found from Figure 14.

Chart values can never be read to 3 significant figures, and since interpolation is frequently necessary this further reduces accuracy. However, the chart value for d_1 in Figure 14 will give 3 significant figure accuracy for v_1 if v_1 is calculated from

[11] $$v_1 = \sqrt{2g(E_1 - d_1)}$$

The reason for this is that d_1 is small compared to E_1, and the square root further minimizes any error in v_1. This value of v_1 may then be used to calculate the exact value for d_1 from the continuity equation ($d_1 = q/v_1$).

The equation for gradually varied non-uniform flow is used to calculate the remaining depths down the chute, that is

[12] $$\Delta x = (E_2 - E_1) / (S_o - S_{fa})$$

Here, Δx is the length of chute between two sections 1 and 2, E_1 and E_2 are the specific energies at sections 1 and 2 respectively, S_o is the slope of the chute and S_{fa} is the average rate of friction loss for the reach between sections 1 and 2, as given by

[13] $$S_{fa} = (S_{f1} + S_{f2}) /2$$

The rate of friction loss at each section is solved by Manning's equation, written in the form

[14] $$S_f = V^2n^2/R^{4/3} = 2gn^2h_v/R^{4/3}$$

in which R is the hydraulic radius and h_v is the velocity head at the section in question.

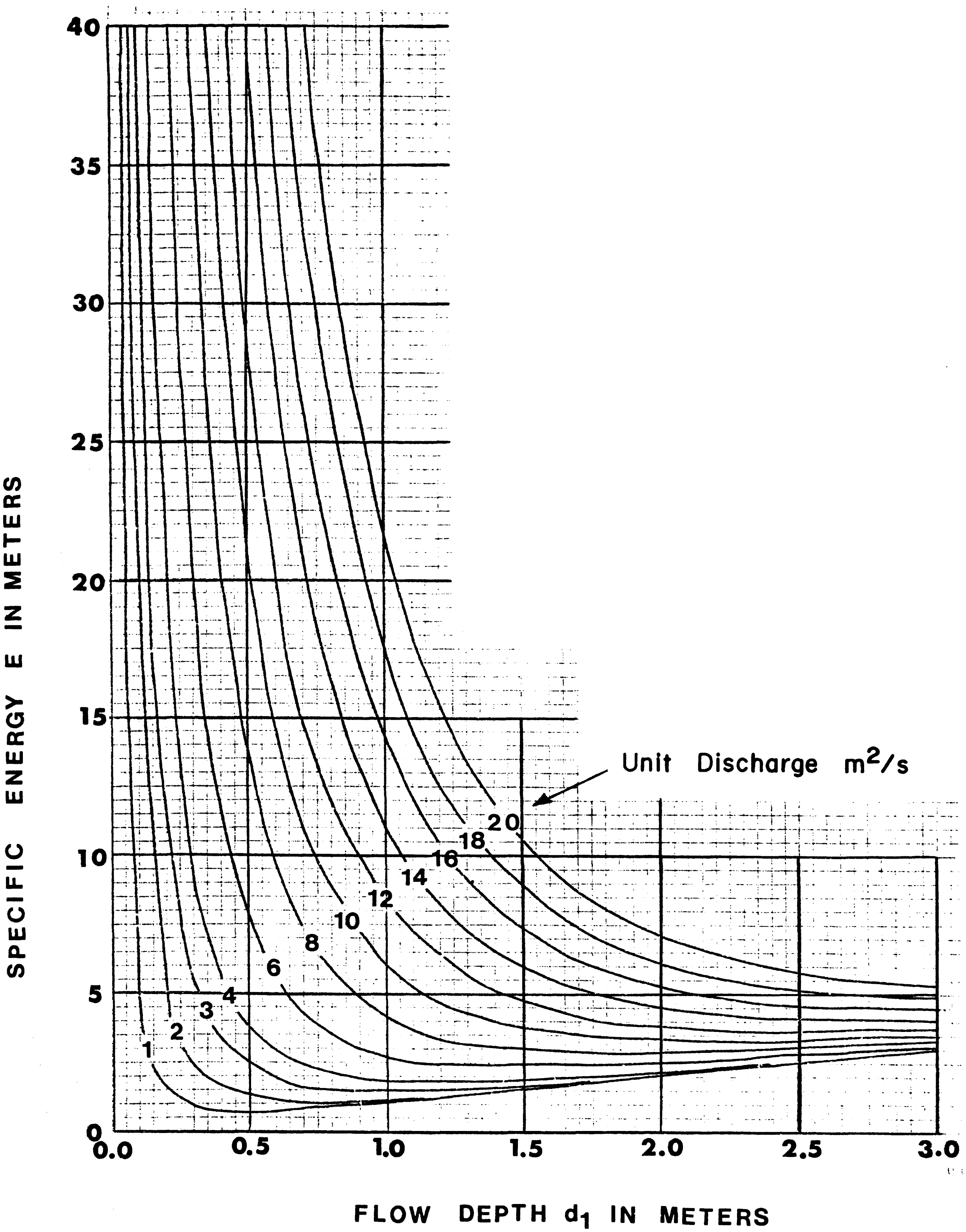


Figure 14. Depth versus Specific Energy and Unit Discharge for Supercritical Flow

The step method solution can proceed without trial and error if [12] is solved for Δx rather than E_2. The values of d_1 and v_1 are known, so E_1 and S_{f1} are computed. A value for d_2 is assumed, from which v_2, E_2, S_{f2}, and S_{fa} are computed. This permits direct solution of Δx, which is the length of the reach to where the assumed depth will be located. A tabular solution greatly facilitates the calculations, as shown in Example 3. This computation procedure can be easily programmed if it is necessary to calculate many profiles.

It is evident that the reliability of the water surface profile calculation depends largely upon the correctness of the value assumed for n. It is assumed that values used for subcritical flow can also be used for supercritical flow, and a value of n = 0.012 is recommended for design.

Example 3:

Evaluate the water surface profile for a discharge of 1400 m^3/s flowing down a 235 m long concrete chute 70 m wide. The chute has a slope of 0.100 and Manning's n = 0.012. The initial depth at the top of the chute is 3 m. From [14]

$$S_f = 2g\, n^2\, h_v/R^{4/3} = 0.00283\, h_v/R^{4/3}$$

Other equations which apply are A = Wd, R = A/P, V = Q/A and E = d + $V^2/2g$, from which:

d	A	P	R	$R^{4/3}$	V	h_v	s_f	Av. S_f	S_o-S_f	E	ΔE	ΔX	Sta.
m	m^2	m	m		m/s	m				m	m	m	
3.0	210	76.0	2.76	3.88	6.67	2.27	0.00166			5.27			0+00
								0.00200	0.0980		0.23	2.3	
2.7	189	75.4	2.51	3.40	7.41	2.80	0.00233			5.50			0+02.3
								0.00287	0.0971		0.44	4.5	
2.4	168	74.8	2.25	2.94	8.33	3.54	0.00341			5.94			0+06.8
								0.00396	0.0960		0.47	4.9	
2.2	154	74.4	2.07	2.64	9.09	4.21	0.00451			6.41			0+11.7
								0.00534	0.0947		0.69	7.3	
2.0	140	74..0	1.89	2.34	10.00	5.10	0.00617			7.10			0+19.0
								0.00745	0.0926		0.99	10.7	
1.8	126	73.6	1.71	2.04	11.11	6.29	0.00873			8.09			0+29.7
								0.01076	0.0892		1.45	16.3	
1.6	112	73.2	1.53	1.76	12.50	7.96	0.01280			9.56			0+46.0
								0.0163	0.0837		2.24	26.8	
1.4	98	72.8	1.35	1.49	14.29	10.40	0.01975			11.80			0+72.8
								0.0263	0.0737		3.56	48.3	
1.2	84	72.4	1.16	1.22	16.67	14.16	0.03284			15.36			1+21.1
								0.0383	0.0617		2.59	42.0	
1.1	77	72.2	1.07	1.09	18.18	16.85	0.04375			17.95			1+63.1
								0.0520	0.0480		3.44	71.7	
1.0	70	72.0	0.97	0.96	20.00	20.29	0.0611			21.39			2+34.8

18. <u>Air Entrainment</u>

Entrainment of large volumes of air is a common occurrence with high velocity flow, as in the case of flow down a steep slope. In the design of overflow spillways or chutes it is necessary to account for the effect of the entrained air on both depth and velocity. The increase in depth must be accommodated by the sidewall height of the structure. Any change in velocity must be considered in the design of the energy dissipation structure at the end of the slope.

Air entrainment in water flow down a steep slope occurs when the intensity of the turbulent fluctuations at the water surface is so great that small masses of water are thrown above the mean surface, entrap air, and fall back into the water phase. The air becomes diffused in the water in the form of bubbles. The resulting mixture of air bubbles and water produces a "white water" condition. An example of this process is shown in Figure 15.

The air concentration varies through the depth of flow, being a maximum at the surface and decreasing with depth. This occurs because the entrapped air is subjected to buoyant force as soon as it becomes submerged. Buoyancy tends to move the air bubbles to the surface. At equilibrium the net gain by entrapment at the surface equals the net loss by expulsion due to buoyancy.

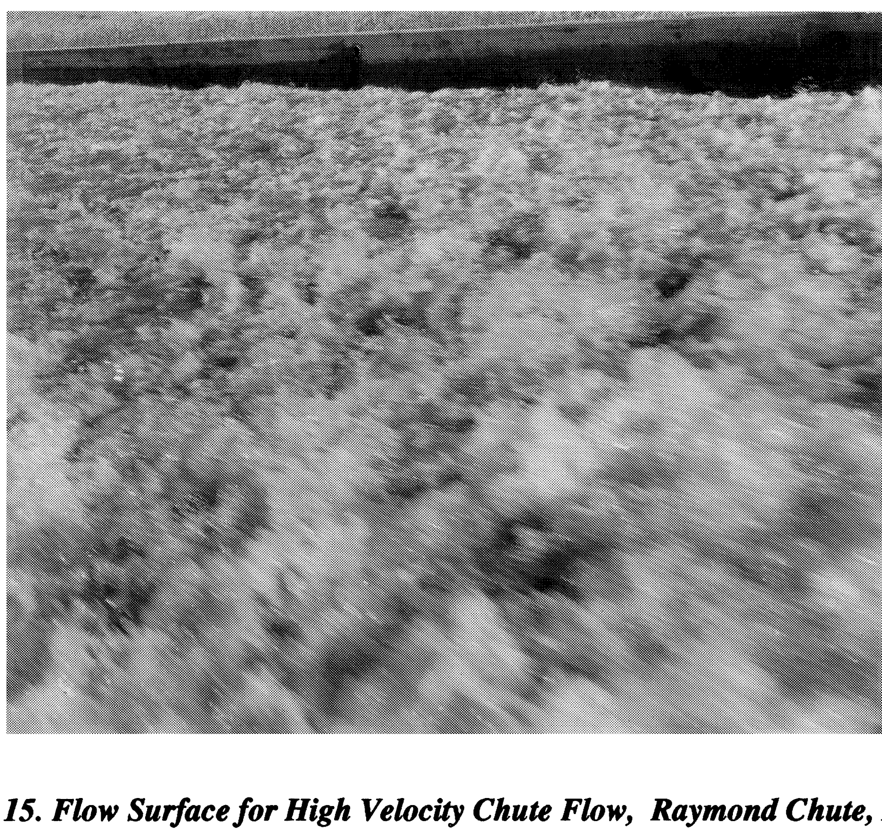


Figure 15. Flow Surface for High Velocity Chute Flow, Raymond Chute, Alberta

Air entrainment requires complete development of the turbulent boundary layer and a high intensity of turbulent energy at the water surface. The latter condition is satisfied if there is simultaneous occurrence of high Reynolds and high Froude number.

The turbulent boundary layer begins to develop at the beginning or entrance to the slope. It increases in thickness until at some point, if the chute is long enough, it becomes equal to the depth of flow. Up to this point the water surface is smooth and transluscent, and appears dark. Downstream from this point the water surface becomes rough, and, if air is entrained, becomes "white water."

A further very important factor in the air entrainment process is the slope of the channel. The pressure head at any depth for open channel flow on an inclined chute is less than hydrostatic, in fact it is $d \cos \alpha$, in which d is the depth perpendicular to the slope and α is the slope angle relative to the horizontal. The reduction in buoyant force associated with an increase in α allows a much greater air concentration to develop. Overflow spillways frequently have values of α in the 50 degree to 60 degree range. Chute spillway slopes are much flatter, usually in the 5 degree to 15 degree range.

Bauer made a detailed laboratory study of the development of the turbulent boundary layer on steep slopes, and, supported by prototype data by Hickox, his work yields the equation

[15] $$\delta/L = 0.0175 - 0.0025 \log (L/k)$$

in which δ is the thickness of the turbulent boundary layer, L is the flow length from the beginning of the chute, and k is the equivalent Nikaradse sand grain roughness. Typically the value of k may be taken as 0.001 m. The actual water depth d_w along the chute may be determined from the water surface profile, as evaluated by the step method using a Manning's n = 0.012. Air entrainment may be assumed to begin where $\delta = d_w$, referred to as the critical point.

The mean air concentration is defined as the ratio of air volume in the air-water mixture to the total volume. This may be expressed in terms of depths d_a and $(d_a + d_w)$ in

[16] $$C = d_a/(d_a + d_w)$$

For example, if C = 50%, the bulked depth $(d_a + d_w)$ is twice as great as the water depth d_w alone with no air entrainment. The U.S. Corps of Engineers equation for air concentration is (in S.I. units)

[17] $$C = 0.826 + 0.7 \log (\sin\alpha/q^{0.2})$$

in which q is the unit discharge in m^2/s. If C is negative, as for large discharges on a flat slope, air entrainment is neglected. The sidewall height of the chute must contain the air entrained flow with ample freeboard allowance. It is recommended that the wall height be 1.5 $(d_a + d_w)$ plus a velocity allowance for splash, shock waves and cross winds of 0.05 m per each 1 m/s velocity. One limitation of [17] is that there is apparently no lower limit to q. Yet if q is small enough, no air entrainment will occur at all because of the viscous damping associated with the small Reynolds number. The Reynolds number may be written as $4q/\upsilon$, with υ the kinematic viscosity at typically $10^{-6} m^2/s$. In order to have a Reynolds number of 10^6 or greater, q must be equal to or greater than 0.25 m^2/s. In addition, in applying [17], it is recommended that $\alpha<60°$.

The effect of α is quite profound. For a given q, an increase in α decreases the buoyant force, decreases the flow depth and increases the velocity. Turbulence intensity at the surface is greater for shallower and higher velocity flow. Hence, given $q = 1 m^2/s$ for example, an increase in α from 10 degrees to 40 degrees, increases C from 29% to 69% and more than doubles $(d_a + d_w)$.

Terminal velocity is reached when the component of water weight parallel to the slope is balanced by the boundary shearing resistance around the perimeter, including the air-water interface. If air entrainment occurs, the wetted perimeter increases, and the air resistance at the surface becomes appreciable. On the other hand, since the viscosity of air is less than 0.01 of that for water, the frictional resistance at the concrete boundary is reduced by the lower viscosity of the air-water mixture. Data seems to indicate that the net effect is an increase in velocity when compared to the velocity without air entrainment. However, at small air concentrations, air bubbles may not penetrate to the floor, and the effect on velocity is minimal. Data from Straub and Anderson show that the increase in velocity is less than 5% up to a mean air concentration of 40%, but increases rapidly for higher concentrations (for example a 30% velocity increase for a 70% air concentration). Such high air concentrations could occur for high steep faced overflow dams, but are not expected for low dams or chute spillways with flatter slopes. In lower height overflow dams the critical point may not be reached, and in chute spillways the smaller α value will significantly reduce the mean air concentration.

The exact effect of high air concentrations on flow velocity on prototypes is difficult to determine and is not well established. Fortunately the problem does not arise in most cases because air concentration is usually small. If necessary, conservative values must be assumed for the design of flip buckets, roller buckets, or hydraulic jump basins.

19. Vertical Curves

The slope of the chute is usually governed by stability or topography requirements, and may have any value. Hydraulic design involving the slope is limited to calculation of vertical curves.

Changes in slope to a flatter gradient may be made abruptly, although a short simple curve is often used. A curve is not really necessary unless the velocity is high and the change in slope is pronounced. A curve radius equal to four times the depth of the chute flow would be ample in any case. Changes in slope to a steeper gradient should be connected with a smooth vertical parabolic curve. The curve should be designed to fit the trajectory which the jet would follow if the vertical acceleration were only two-thirds gravity. The computed trajectory for the vertical curve is based on the average chute velocity V. A full gravity curve should not be used because owing to uncertainties in the value of n, the exact velocity may not be known, and in any case the maximum velocity (on the surface) may be up to 15% greater. The use of a two-thirds gravity curve will insure that the jet is positively supported, thereby avoiding negative pressures and eliminating the risk of separation of the jet or overtopping of the walls.

The coordinates of the two-thirds gravity curve, with x measured horizontally and y measured vertically downward, are found from the equations of motion $x = V_x t$ and $y = V_y t + 1/3\ gt^2$, giving

[18] $$y = xS_1 + gx^2 (1 + S_1^2) / 3V^2$$

in which V is the chute velocity and S_1 the initial chute slope. These coordinates define a curve which comes tangent to the final slope S_2 in a horizontal length of

[19] $$x = 3(S_2 - S_1)/(1 + S_1^2)\ V^2/2g$$

20. Joints and Drains

The success or failure of a chute spillway depends as much upon the proper design and construction of the slab, joint, and drainage details as it does upon the hydraulic design itself. An elaborate hydraulic design cannot compensate for a poorly designed or installed drainage system. The chute portion of the spillway in particular requires special attention.

It is almost impossible to prevent water from getting under the chute slab at some time or another. It may come from below in the form of seepage from the reservoir, or it may come from above as leakage through the slab (large concrete slabs cannot be made watertight). There is the risk of erosion of the subgrade, or a pressure buildup under the slab, or both. Should fast flowing water gain entry to the underside of the slab through a crack or opening, failure will almost certainly result. A velocity of only 7 m/s, for example, if converted to pressure head, would be capable of lifting a floor slab 1 m thick. Since these slabs are usually about 0.3 m to 0.5 m thick, this possibility must be avoided.

A common design for the joint and drainage details for a chute slab is shown in Figure 16. The joints are intended to control cracking due to shrinkage, temperature drop or foundation movement. The joint filter must accommodate thermal expansion of the slab during hot weather. The drainage layer is designed to collect seepage from below and leakage from above. The crosswall intercepts seepage in the drainage layer of each floor panel. The water is collected in a perforated lateral drain and conveyed to a longitudinal drain running parallel to the spillway outside of the sidewalls. The longitudinal drain may discharge through the sidewall above the floor slab at a point further down the slope. The joint is stepped, dowelled, and bevelled, as indicated, in order to ensure positively that the edge of the downstream slab will not project or rise above the upstream slab. Model tests have shown that the pressure which will develop at the skip joint with a 0.01 m depression is approximately $-0.05\ V^2/2g$. To avoid the risk of cavitation damage, the skip should not be used where velocities may exceed 25 m/s. The dowel must be a smooth coated rod with a slip end equal in length to the thickness of the joint filler so that the slab can freely expand without producing stress concentrations in the concrete at the end of the dowell.

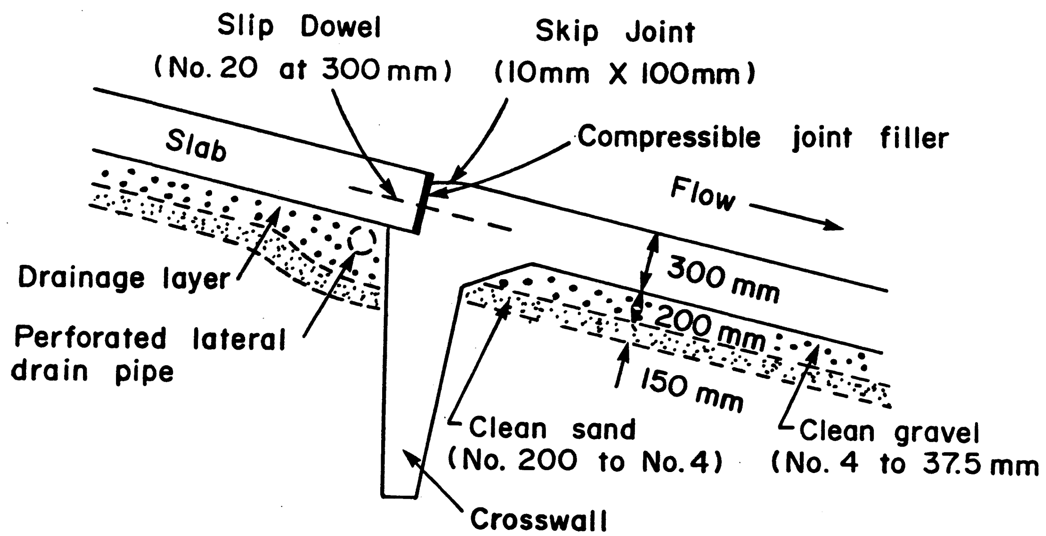


Figure 16. Joint and Drainage Detail for a Spillway Chute

The crosswall is poured first, with reinforcement extending through the top to overlap the downslab reinforcement. The downstream slab may then be poured, with the dowel in place. An asphalt impregnated compressible fibre board may be used for the joing filler, and serves as a spacer between the ends of the slab at the joint where the upstream slab is poured. The thickness of the spacer must be enough to accommodate the thermal expansion of the slab ΔL, given by

[20] $\Delta L = \alpha \cdot \Delta T \cdot L$

in which α is the coefficient of linear expansion in m / m / °C, ΔT is the seasonal temperature change in °C (from average minimum to average maximum), and L is the length of the slab between joints. Given $\alpha = 10 \times 10^{-6}$/°C, $\Delta T = 50$°C and L = 10 m, then [20] gives $\Delta L = 0.005$ m, for which a spacer thickness of say 10 mm would be recommended.

21. Downslope Creep

The theoretical downslope creep of an unanchored slab caused by seasonal contraction and expansion due to temperature change, sometimes referred to as "thermal creep", may be determined by locating the change in the position of the knickpoint on the slab. The knickpoint is the point on the slab which remains fixed as the remainder of the slab moves upslope or downslope as the slab expands or contracts, and may be located by static equilibrium analysis as follows:

Consider a slab of length, L on a slope of θ degrees , subjected to thermal contraction. The knickpoint will be at position x_c from the lower end, as shown on Figure 16, where x_c represents the downstream length of slab which contracts upslope. The slab will contract toward the knickpoint from both ends, producing frictional resistance opposing the motion as indicated by the shear vectors shown at the base. The net downslope force will be

[21] $$W\sin\theta + \frac{x_c}{L} W (\cos\theta)\,\mu$$

in which W is the slab weight, $W\sin\theta$ is the tangetial component of weight, $W\cos\theta$ is the normal component of weight, and μ is the coefficient of friction at the base. The upslope force will be

[22] $$\frac{L - x_c}{L} W (\cos\theta)\,\mu$$

Since these forces must be equal, for static equilibrium, then x_c may be solved as

[23] $$x_c = \frac{L}{2}\left(1 - \frac{\tan\theta}{\mu}\right)$$

If the slab is horizontal, $x_c = L/2$, which must be the case. If the slab is at the angle of repose, $\tan\theta = \mu$, and $x_c = 0$, which also must be the case. It is of interest to note that the solution is independent of slab weight, and therefore independent of slab thickness.

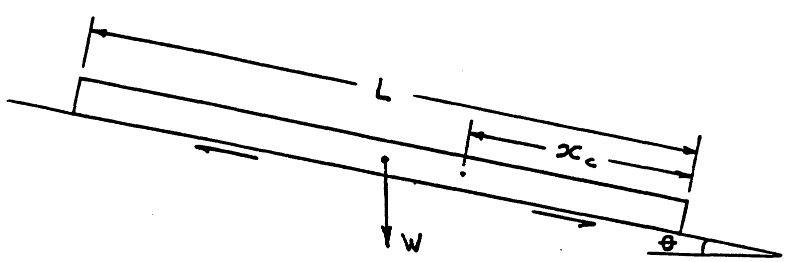


Figure 16. Slab Contracting

When the slab expands, the frictional forces on the base are reversed, in which case

[24] $$W\sin\theta + \frac{L - x_e}{L} W (\cos\theta)\mu = \frac{x_e}{L} W (\cos\theta)\mu$$

where x_e represents the length of slab which expands downslope, giving

[25] $x_e = \frac{L}{2}(1 + \frac{\tan\theta}{\mu})$

Since $x_e > x_c$, there is greater downslope movement at the end of the slab during expansion than is recovered during contraction. The effective slab length which moves only downslope during a thermal cycle is the length between the kickpoints, $x_e - x_c$, or $L(\tan\theta)/\mu$. Applying the coefficient of linear expansion α and a seasonal temperature change of ΔT (from average minimum to average maximum), then the net downslope average creep ΔS becomes

[26] $\Delta S = \alpha\,\Delta T\;L\,(\tan\theta)/\mu$

On spillways the actual downslope movements tend to be less than the theoretical value because the full depth of the slab may not undergo the complete temperature change ΔT, paricularly for thicker slabs (> 300 mm). Also, there is usually some edge restraint at the walls. Edge restraint may completely offset the creep for flatter slopes (< 10%), converging sidewalls, or narrow structures. Usually the worst cases of downslope creep occur for thin slabs with diverging sidewalls on steep slopes.

Regardless of how small the actual creep may be, the important point to note is that it does occur and it is cumulative, and if allowed to continue may result in joint openings upstream, shear of waterstops at the walls, and spalling at joints subjected to compression downstream.

In order to eliminate downslope creep the slab must be anchored. This may be done by designing the crosswall at the upstream end of the slab as an anchor wall. The depth of the wall must be sufficient so that the difference betwen active pressure on the upstream side of the wall and the passive pressure on the downstream side will produce enough force to pull the entire slab upslope as it contracts. The required force T is equal to the downslope weight component of the slab plus the friction force at the base of the slab, given by

[27] $T = \gamma_c\,(\sin\theta + \mu\cos\theta)\,Lt$

in which T is the tension force in kN/m of slab width, γ_c is the unit weight of concrete (23.6 kN/m^3), and t is the slab thickness in m.

The active and passive earth pressure coefficients K_a and K_p may be estimated from Figure 17, taken from Design Manual 7.02, "Foundation and Earth Structures", U.S. Naval Facilities Engineering Command, Virginia, 1986. The coefficients are for Coulomb's equation for sloping backfill, and thus account for the fact that there is a positive surcharge on the active pressure side of the crosswall and a negative surcharge on the passive pressure side. The required depth of the crosswall, H, must satisfy the relationship

[28] $T = (K_p - K_a)\,\gamma\,H^2/2$

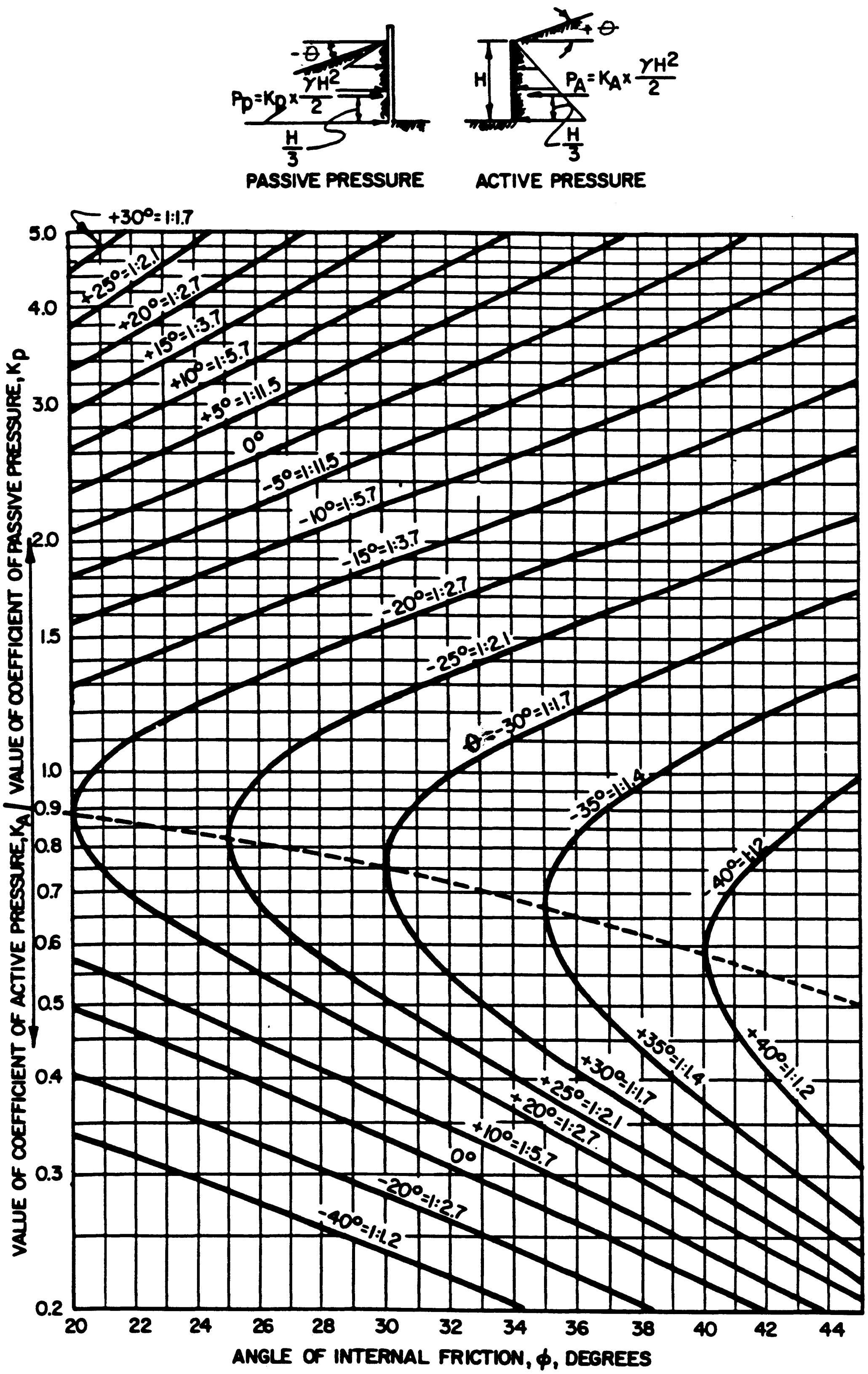


Figure 17. Active and Passive Coefficients, Sloping Backfill (Granular Soils)

in which γ is the unit weight of soil (usually 16 to 18 kN/m^3). The coefficients given in Figure 17 are based on the assumption that the subgrade consists of granular soil. If the crosswall projects into firm clay or clay till, the resistance will be increased and the values from Figure 17 will be conservative.

It should be noted that continuous unjointed slabs can also be used for spillway chute slabs. In this case, however, the chute slab should be anchored over the entire length. Such anchorage can be achieved by anchor rods grouted into rock or concrete anchor piles. In structures which are sufficiently narrow (< 10 m) so that the walls and floor may be monolithic, the wall backfill alone may provide sufficient resistance to prevent downslope creep. In short structures, downslope creep of the chute slab may be prevented by the sliding resistance of the stilling basin at the bottom, but the slab will be under compression and must be checked for the possibility of upward heave (buckling). Normally, the slab weight and top layer of reinforcement will be sufficient to offset any tendency to buckle.

Example 4:

Determine the required depth of a crosswall necesary to prevent downhill creep of a sloping spillway slab 12 m long and 350 mm thick, placed on a 17.5% slope (10 degrees).

From [27], using $\mu = 0.6$, then $T = 23.6 (\sin 10 + 0.6 \cos 10°)\ 12 \times 0.35 = 75.8$ kN/m.

From Figure 17, for $\theta = 10$ degrees and $\phi = 35$ degrees, then K_a is 0.3 and K_p is 2.65. The required wall height H, is found from $(2.65 - 0.3)\ 16\ H^2/2 = 75.8$, giving H = 2 m. This could probably be reduced to 1.5 m for a clay till subgrade.

22. Stilling Basin

The hydraulic jump stilling basin is almost invariably used for chute spillways for earth dams, because the earth dam is often constructed at sites where rock foundations are not available. The presence of weak or erodible material in the discharge channel requires that the excess energy of flow be dissipated within the structure, unlike the case for the flip bucket or roller bucket. Since the hydraulic jump basin is frequently used on many other spillways and hydraulic structures as well, it is covered in detail in Part E of this Chapter.

E. HYDRAULIC JUMP STILLING BASINS

23. Basin Width

The most common design for the lower end of a chute spillway discharging into erodible material is a rectangular hydraulic jump stilling basin with parallel sides and a level floor. The significant hydraulic dimensions are the basin width, length, height of wall, and the depth of the floor below the tailwater.

Most of the hydraulic dimensions can be related directly to the size or height of the hydraulic jump d_2, and since d_2 depends upon the width of the basin, this width must be selected before the other hydraulic dimensions can be calculated. Generally, the wider the basin the better the hydraulic performance. The reason for this is that: (1) the discharge concentration and velocity are decreased; (2) the size and strength of the downstream eddies adjacent to the discharging flow are reduced; (3) the intensity of the wave action generated by the hydraulic jump is reduced; and (4) uplift pressures associated with the hydraulic jump are reduced.

One approach would be to select a basin width which would produce a jump height curve to match the tailwater rating curve. Normally, this practice would result in excessive width and cost, as most natural rating curves are rather flat. The more common procedure is to use a narrower basin and depress the floor elevation below the natural bed. In fact, there are some major spillways where more of the depth required to force the hydraulic jump at the end of the basin is obtained by excavation below the natural bed than is available from natural tailwater above it.

The predominant factor which determines the overall size of the basin is the design discharge. If all other factors were equal, the width could be expressed in terms of this discharge in a relation of the form

[29] $$W = K\sqrt{Q}$$

A value of K = 1.8 frequently gives the desired combination of cost and performance and this figure may be used as a guide in selecting the width. In a particular case, of course, there are a number of factors which may dictate a wider or narrower basin than given by this relation. For example, any limitation on the discharge velocity (due to a highly erodible channel), on the depth of the excavation (due to groundwater or hard strata), or on the height of the walls (due to structural or foundation reasons), would automatically put a lower limit on the width of the basin irrespective of the foregoing relationship. Other factors which could have a bearing either directly or indirectly on the selection of the basin width, are the width of the control section, the shape of the tailwater rating curve, and the topography.

It should be noted that in short spillways it is common to use parallel sidewalls throughout, in which case the width of the crest section, chute and stilling basin are all equal. This is done to avoid the need for transitions from one width to another, which, if used, would require a model test. When a constant width is used, it is not uncommon to use [29] to select this width. In effect the stilling basin, which is a major component, tends to govern the width of the entire structure.

24. Hydraulic Jump Equation

The momentum equation written for the hydraulic jump for a unit width on a plain level floor is

[30] $$\gamma d_2{}^2/2 - \gamma d_1{}^2/2 + F_f = q\rho\,(\beta_1 V_1 - \beta_2 V_s)$$

in which F_f is the floor friction force and β_1 and β_2 are the momentum correction factors. It is commonly assumed that F_f can be neglected and $\beta_1 = \beta_2 = 1.0$. The errors in these assumptions are small and tend to compensate. The equation then becomes

[31] $$\gamma d_2{}^2/2 - \gamma d_1{}^2/2 = q\rho(v_1 - v_2)$$

Since $v_1 = q/d_1$ and $v_2 = q/d_2$, this may be written as

[32] $$q^2/g = d_1 d_2 (d_1 + d_2)/2$$

Equation [32] is one form of the hydraulic jump equation. However, in the usual application q and d_1 are known and d_2 is to be found. Equation [32] is a quadratic and may be solved for d_2, giving

[33] $$d_2 = - d_1/2 + \sqrt{(2v_1{}^2 d_1/g) + d_1{}^2/4}$$

A variation of [33], is

[34] $$d_2 = (\sqrt{8F_1{}^2 + 1} - 1)\, d_1/2$$

in which F_1 is the Froude number at the toe of the jump (i.e., $F_1 = v_1/\sqrt{gd_1}$).

25. Basin Length

It is evident that the length of the stilling basin should bear some relationship to the length of the hydraulic jump. Although the length of the jump (as defined by the length of the surface roller) varies slightly with the Froude number, it may be taken closely as $5d_2$. A basin length of $6d_2$ will effectively confine the jump longitudinally, and this figure has often been used in the past. Numerous model studies have shown that with baffle blocks and acceptance of somewhat less than ideal conditions, much shorter basin lengths can be used. A basin equipped with floor blocks and end sill and a length of $3d_2$ has been selected from model and prototype studies as a suitable standard for most designs. The performance of such a basin gives entirely adequate protection to the structure and the downstream channel. The appearance of a model of this basin in operation at design flow is shown in Figure 18, and a prototype in Figure 19.

26. Elevation of Basin Floor

The floor of the stilling basin must be set a sufficient depth below the minimum tailwater to confine the hydraulic jump to the basin at all times. For a plain level basin, the minimum allowable depth is the sequent depth d_2 as given by [34]. If floor blocks are used, the jump can be held in the basin with less tailwater, in fact it can be held in the basin with the floor level only 0.8 d_2 below the tailwater level. The depth 0.8 d_2 is called the sweepout depth because the jump is on the verge of being swept out of the basin. However, since the function of the basin is reduction in the velocity, it's purpose would be partially defeated if the exit depth were reduced to the minimum of 0.8 d_2. Performance is considerably superior with the floor elevation placed a full d_2 below the available tailwater level, and this is recommended for design.

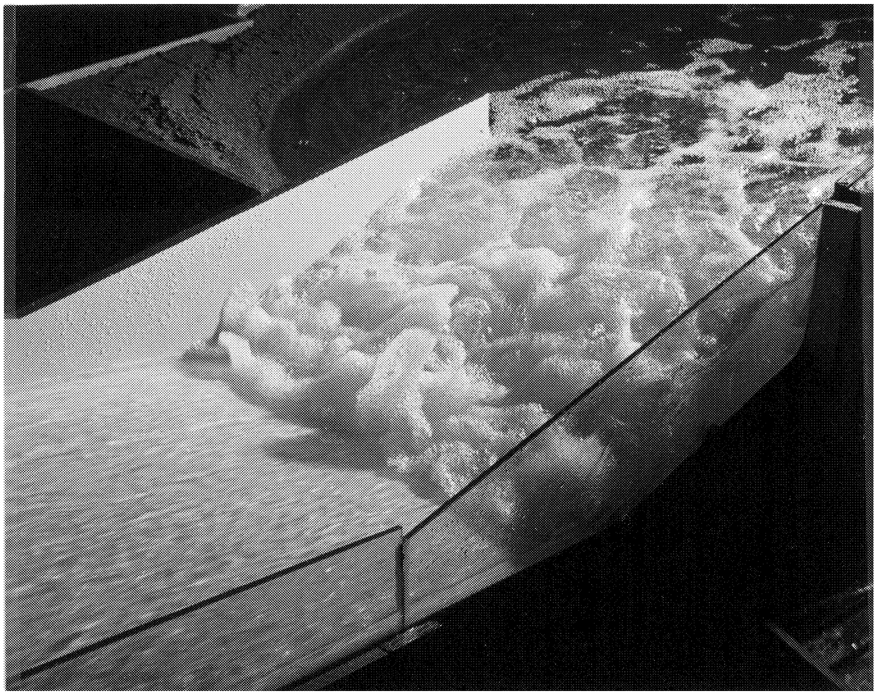


Figure 18 Model of Hydraulic Jump Stilling Basin, $L_b = 3d_2$

Figure 19   Chute Spillway with Hydraulic Jump
Duncairn Dam, Saskatchewan

As described here, the basin at the design condition utilizes the capacity of the floor blocks to shorten the length of the jump (and basin), but apparently not the capacity to reduce the height of the jump. This is not actually true. The difference between 1.0 d_2 and 0.8 d_2 is a valuable safeguard in the event of future retrogression of the tailwater level. Some allowance should always be made for degradation of the stream channel below a dam built across an alluvial stream. Frequently the amount of this allowance is a matter of speculation. In the absence of specific data on a given project, it is suggested that the retrogression allowance be 0.2 d_2, and that the permissible tailwater depth for this possibility be 0.8 d_2. The philosophy of this approach is that it would be too severe an imposition on the design to place the basin floor the full value of d_2 below the tailwater level for the case of both maximum discharge and ultimate retrogression.

27. Basin Wall Height

The walls must be high enough to exclude tailwater from the jump area, otherwise the tailwater will interfere with the jump action. This height must be at least 1.05 d_2. It may seem that this recommended wall height does not provide for much freeboard. Actually, freeboard is a rather meaningless term when applied to a basin of this design, in that tailwater covers the entire area outside the basin adjacent to the hydraulic jump. Under these circumstances an occasional splash over the walls is of no consequence, and additional height above 1.05 d_2 does not contribute anything to the safety or performance.

Owing to the uncertainty in the prediction of the tailwater, generally the basin must be designed to operate satisfactorily for a range of downstream water levels. The maximum tailwater may very well exceed 1.05 d_2 above the floor of the basin. In the past a common solution to this problem has been to place the top of the level stilling basin wall above the maximum high tailwater. The sidewall height would have to include the freeboard, the range of tailwater variation, the retrogression allowance, and the depth for the hydraulic jump. This would often result in excessive wall height. More recently, model tests have shown that when the tailwater is excessive, the toe of the jump is pushed up the slope of the chute entering the basin. The most turbulent part of the jump is then completed even before the flow reaches the level basin. Under these circumstances a wall height of h_w = 1.05 d_2 is quite adequate for the level basin despite excessive tailwater, provided the chute walls are high enough to exclude tailwater from the area at the toe of the hydraulic jump on the slope. This requirement can be met if the top of the chute wall is constructed parallel to the floor, and made to intersect the level basin wall at the length of 2 d_2 from the end of the basin, as shown by the definition sketch, Figure 11.

28. Details of Blocks and End Sill

Chute blocks, located at the toe of the chute, are intended to induce the rapid onset of fine grain turbulence and improve the efficiency of the jump. As the pressure in the lee of the chute blocks is subatmospheric, this is also a good position to empty the basin subfloor drainage system. Concern has been expressed that because the pressure fluctuates due to surges in the jump, a pumping action could occur in the drain which may remove foundation material. This can be prevented by connecting the lateral header in front of the chute blocks to a vertical air vent at the ends of the lateral at each side wall. In this way the downstream face of each chute block containing a drain is vented to atmosphere.

The basin blocks are responsible for producing most of the resistance to flow which results in shortening the length of the jump. The resistance is actually an induced drag force which results when the floor jet impinges on the vertical upstream face of the basin blocks. The function of the end sill is to deflect the high bottom velocity away from the bed. This results in a ground roller which keeps bed material tight against the downstream cutoff.

The specifications for the blocks and sill are as follows:

Chute blocks: Height = d_1 or $d_2/9$, whichever is greater;

Width = spacing = 0.75 to 1.00 times height.

Basin blocks: Height = d_1 or $d_2/8$, whichever is greater;

Width and spacing the same as for the chute blocks, but staggered;

Location = 0.4 L_b from start of basin.

End Sill: Height = $d_2/10$.

A design based on the above specifications is illustrated in Figure 13.

If the basin blocks are set too close to the toe of the chute, they will be only slightly submerged below the surface roller and the impinging floor jet may be deflected vertically and rise above the stilling basin walls. Yet, if the basin blocks are set far back in the basin, the floor jet will have so thickened by the time it reaches the blocks that they will only intercept the bottom layers of the jet and a great deal of their effectiveness will have been lost. According to tests, if the blocks are more than 0.4 L_b from the toe of the chute, then the basin should be longer then $3d_2$ to get equivalent end results.

Piezometers have been installed around the blocks on a model to determine the pressures which occur while the structure is in operation. These pressures vary with both the discharge and the tailwater. The complete results of the pressure tests are superfluous, but it is of interest to quote the results for the case which gives maximum high and low pressures; that is, full capacity discharge and sweepout tailwater depth (80% of d_2). For convenience the pressures are expressed as a coefficient of the jet velocity head at the toe of the chute. The coefficients are as follows:

+0.3 at the beginning of the chute blocks at the change in slope from the chute (at 1V:3H) to horizontal, and measured from the water surface;

-0.15 behind the chute blocks, measured from the top of the blocks;

+0.24 between the chute blocks at the intersection of the chute floor and basin floor, measured from the top of the chute blocks;

+0.74 in front of the basin blocks, measured from the floor of the stilling basin (note that the maximum possible coefficient with tailwater removed, would be +1.00, corresponding to stagnation pressure);

+0.12 behind the basin blocks, measured from the floor of the basin.

The means by which the floor blocks are able to reduce the tailwater depth needed to produce a hydraulic jump can be shown by application of the momentum equation, including the drag forces due to the presence of the blocks. The drag force F_B is given by the drag equation

[35] $$F_B = C_d A_b \rho v_1^2/2$$

in which A_b is the projected area of the blocks perpendicular to the direction of flow, and for each row of blocks is equivalent to $d_1/2$ m^2 per metre of basin width. The coefficient of drag is much larger for the basin blocks than the chute blocks because of the high pressure which occurs on the vertical upstream face of the basin blocks. The coefficient is about 0.3 for the chute blocks and close to 1.3 for the basin blocks. However, while the reference velocity V in [35] is equal to v_1 for the chute blocks, it is only about 3/4 of v_1 for the basin blocks because of the head loss at the chute blocks and the expansion of the jet before it reaches the basin blocks. The combined effect of the two rows of blocks is to produce a total drag of

[36] $$F_B = \rho d_1 v_1^2/4$$

The momentum equation for the jump, including this force, is

[37] $$\gamma d_1^2/2 + q\rho v_1 = \gamma d_2^2/2 + q\rho v_2 + \rho d_1 v_1^2/4$$

If [37] is solved for d_2, it will be found that this depth is substantially less than given by [33] or [34], and is typically about $0.85d_2$ with the toe of the jump at the downstream face of the chute blocks. With the tailwater at $0.8d_2$ the toe of the jump will move downstream and the block drag force will increase. At a depth less than $0.8d_2$ the blocks will not be submerged and the portion of the jet striking the face of the blocks will be deflected vertically upward, reaching an elevation above the height of the sidewalls.

29. Uplift Pressure

The possible existence of uplift pressure under a stilling basin while operating with a hydraulic jump in the basin has frequently been overlooked by inexperienced designers. Tailwater surrounds the basin at the end and two sides, and may be considered as a head of water standing over the downstream and side cutoff. The potential uplift pressure head under the stilling basin slab is equal to the depth of the tailwater above the bottom of the slab. When the spillway is operating with a hydraulic jump in the basin, the downward force is equal to the weight of the jump and the weight of the floor slab. Since the toe of the jump is depressed well below the tailwater level, the downward force may not balance the uplift force. Neglect of this fact may lead to an uplift failure of the stilling basin floor slab. There are several cases on record of a failure of this type. The result of such a failure is shown in Figure 20.

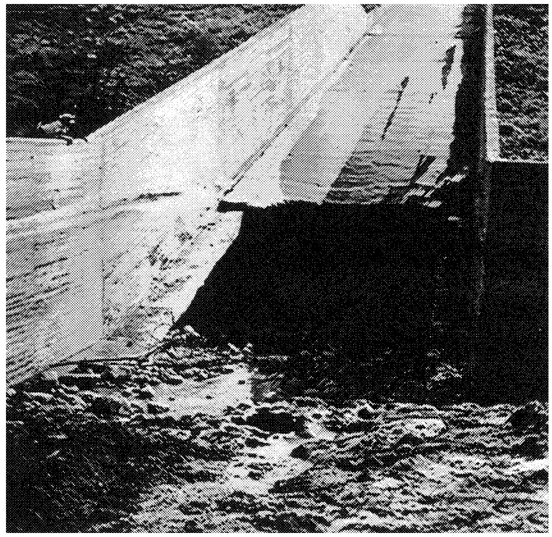


Figure 20. Uplift Failure of a Stilling Basin Floor

The worst case for potential uplift will occur at maximum discharge and with the tailwater at sweepout depth, because this gives the greatest difference in area between the flow profile and the tailwater level. This area is indicated by the approximate triangle abc in Figure 21. The height of the jump h_j, which is the vertical height of the triangle, will be 0.8 d_2 - d_1. This height and the area of the uplift triangle will be smaller at the design tailwater level because the toe of the jump will form upslope on the chute slab. The triangle abc is called the potential uplift triangle, because if no attempt is made to relieve uplift pressure, then the net vertical force to be resisted per unit width of basin will be equivalent to γ times the area of abc, minus the buoyant weight of the slab. The buoyant weight of the slab is only 23.6 - 9.81 = 13.79 kN/m^3, so each 0.1 m of slab thickness will only balance a 0.137 m head of water. Hence for a 10 m jump height it would require a 7 m thick slab to offset the differential vertical water force at the toe of the jump. Usually the slab will have sufficient beam strength that the uplift triangle can be carried by the whole slab, and since the average height of the triangle is only 1/2 h_j, then a 3.5 m slab would be required to balance the uplift force due to a 10 m jump height. It is evident that it is not economical to attempt to resist uplift by weight of slab except for small structures. Usually stilling basin floor slabs range in thickness from a minimum of 0.3 m up to 1 m, and other means are used to combat uplift.

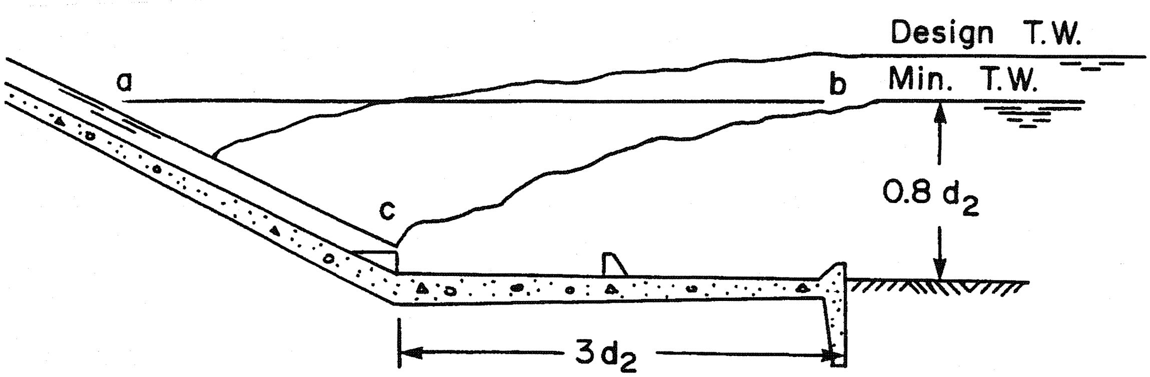


Figure 21. Potential Uplift Triangle

Example 5:

Calculate the total potential uplift force per unit width for a standard hydraulic jump stilling basin, given $d_1 = 1.2$ m, $v_1 = 15$ m/s, $S_o = 0.25$, and minimum tailwater depth.

From [33], $d_2 = -1.2/2 + \sqrt{(2 \times 15^2 \times 1.2)/9.81 + 1.2^2/4} = 6.84$ m

At sweepout, $d_2 = 0.8 \times 6.84 = 5.47$ m, so $h_j = 5.47 - 1.2 = 4.27$ m. The length of the uplift triangle is $4 \times 4.27 + 3 \times 6.84 = 37.6$ m. The uplift force is, then,

$$U = 9.81 \times 37.6 \times 4.27/2 = 787.5 \text{ kN/m}.$$

As indicated in Example 5, the magnitude of the potential uplift is directly dependent upon the height of the uplift triangle. However, an equally important factor which must be taken into consideration is the position of the uplift triangle. Basin floor slabs for spillways are almost invariably depressed below the natural downstream bed (e.g., 4.6 m for Gardiner Dam spillway). This depression is necessary to provide sufficient tailwater depth above the basin floor to satisfy the jump requirements at maximum discharge. At smaller discharges, therefore, the tailwater will be more than required and the jump will be pushed up the slope preceding the basin. The uplift triangle may well be centered over the chute, so that the chute slab may be subjected to greater uplift force than the basin. Neglect of this factor led to the chute failure of Karnafuli Dam spillway in Bangladesh. It becomes necessary in design to determine the location of the uplift triangle, and corresponding uplift, for a range of discharges and tailwater levels.

Figure 22 gives a jump classification according to position. For the A-jump the toe of the jump is at the toe of the chute, as would be the case for maximum discharge and the tailwater at the sweepout depth. For the B-jump the toe of the jump is upstream on the slope, and the jump straddles the chute and basin. The B-jump is by far the most common on spillways because the probability of maximum discharge within the life of the structure is remote, and the A-jump is a rarity. The C-jump ends at the toe of the chute, and the D-jump is entirely on the slope. For practical purposes, the jump characteristics for the C-jump and D-jump are identical. The D-jump is common on

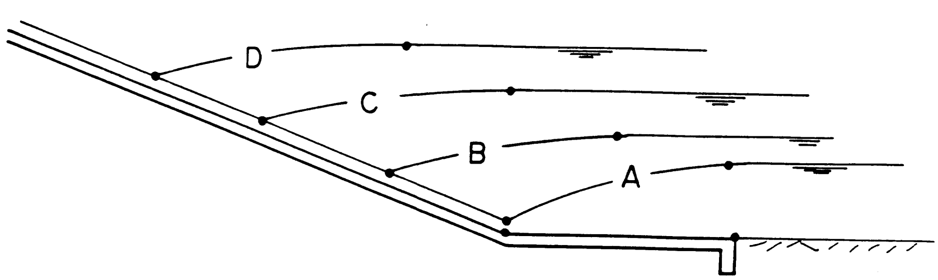


Figure 22. Jump Classification

reservoir inlet structures where there is no level stilling basin, and the jump occurs on a continuous slope (see Chapter 6, Part E, page 6-16).

The height of the jump is smaller on a sloping floor than on a level basin, so as the jump is pushed upstream the value of h_j decreases. Also, h_j is smaller on steep chute slopes than on flatter slopes. These effects are accounted for in the jump height charts produced by Gui in 1992. These charts give h_j values for A, B, C and D jumps for a range of Froude numbers and for entry slopes into the basin of 0.1, 0.2 and 0.3, and are shown on Figures 23, 24 and 25. The position of the toe of the jump can be located on the surface of the inflow jet at a point where it is h_j below the tailwater level. Values of h_j for entry slopes other than 0.1, 0.2 and 0.3 can be determined by interpolation from the values in Figures 23, 24 and 25.

Part of the uplift force corresponding to the uplift triangle will be offset by the submerged weight of the slab. Slab weight alone may be sufficient for smaller irrigation structures. Also, where the chute and stilling basin are sufficiently narrow that the floor slab is cast monolithically with the sidewalls (i.e., no contraction joint), uplift may be resisted by the combined weight of floor slab, sidewalls and backfill on the sidewall footing. Of course the slab must have sufficient transverse beam strength to transmit the excess load to the sidewalls. For wide spillways or where there are contraction joints between the floor slab and sidewalls, the floor slab must be independently stable against uplift.

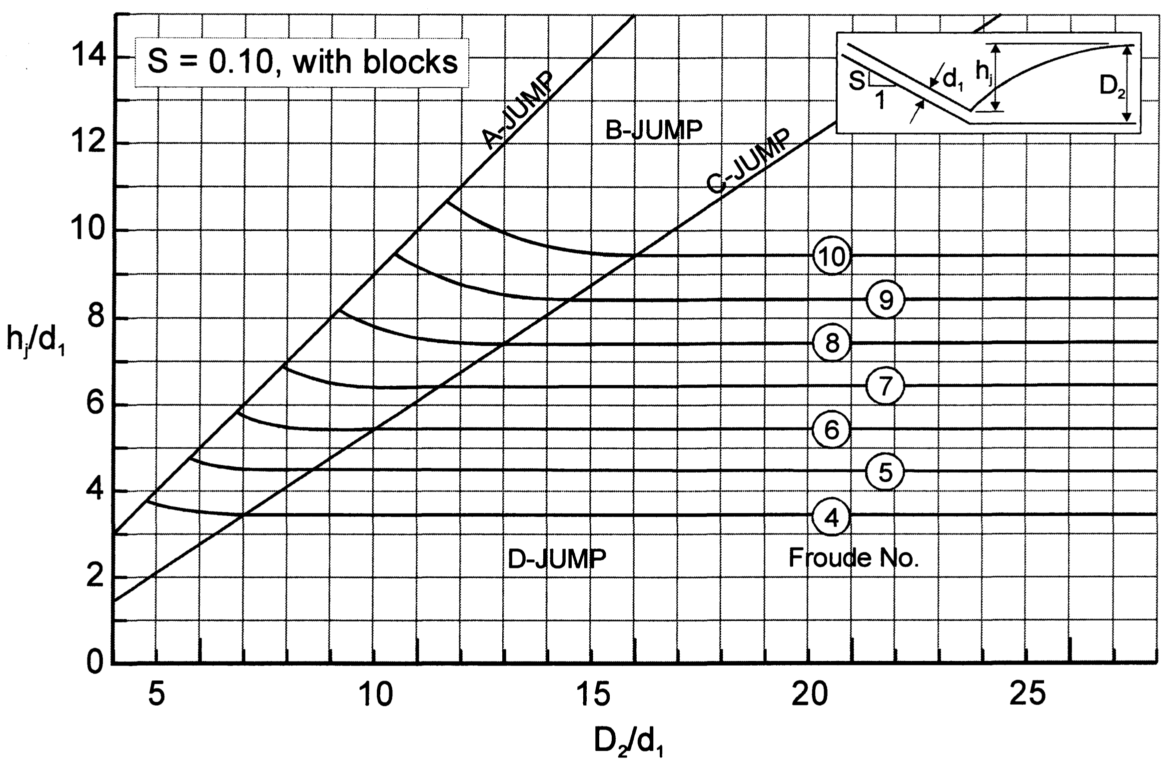


Figure 23. Jump Height Ratio vs. Tailwater Depth Ratio (S=0.1)

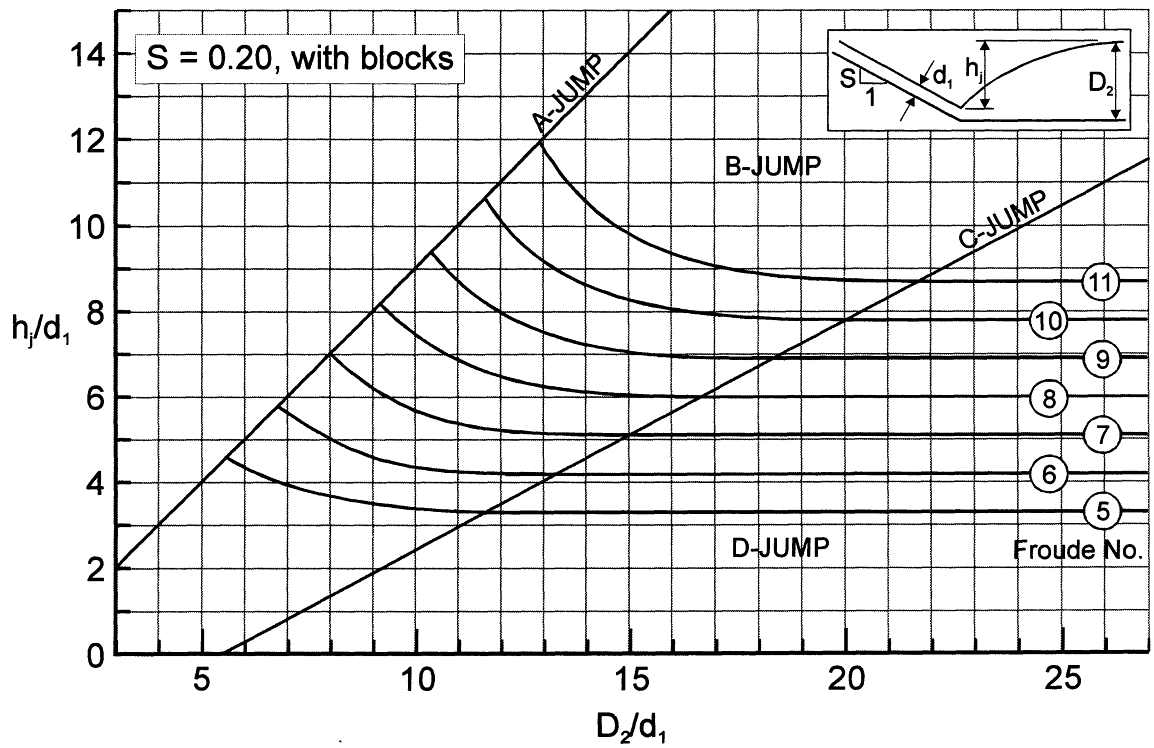


Figure 24. Jump Height Ratio vs. Tailwater Depth Ratio (S=0.2)

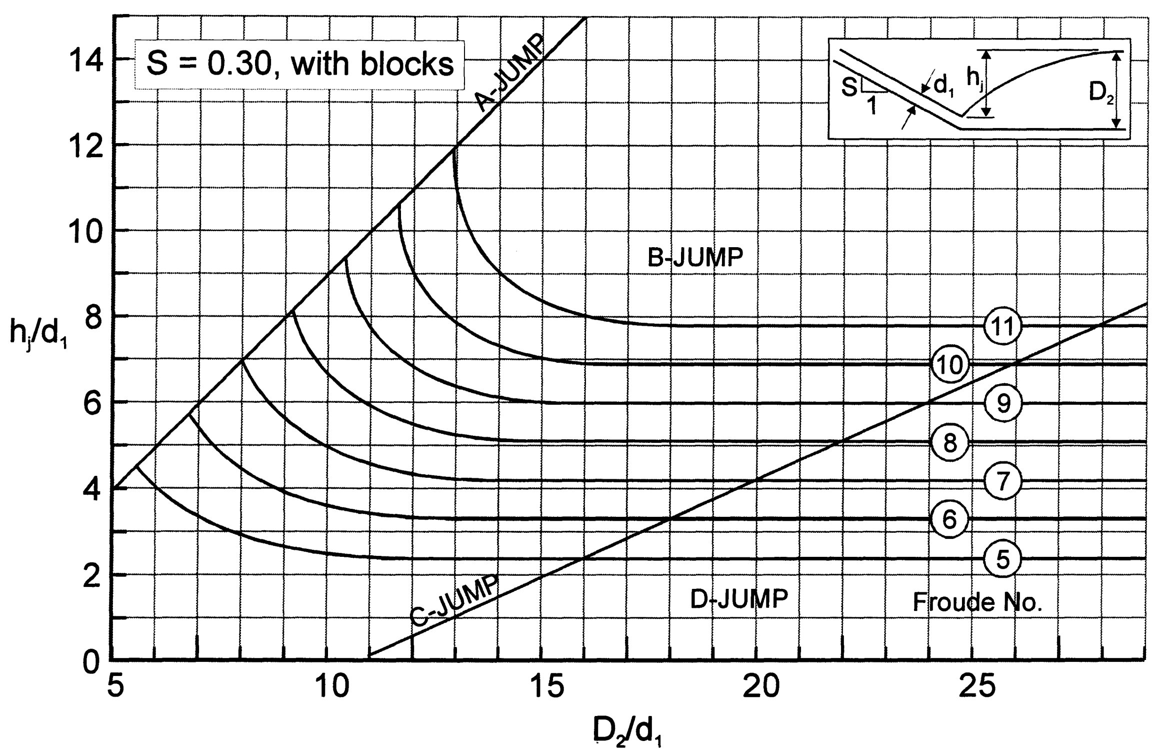


Figure 25. Jump Height Ratio vs. Tailwater Depth Ratio (S=0.3)

The horizontal leg of the uplift triangle upstream from the toe of the jump is always h_j/S in length. The length of the downstream leg (L) varies considerably, depending primarily on the tailwater depth ratio D_2 / d_2. At sweepout depth, with $D_2 / d_2 = 0.8$, the toe of the jump will be located between the chute blocks and basin blocks. The face of the jump will be relatively steep, giving L / d_2 about 2.75. As the tailwater depth is increased, the toe of the jump will move upstream on the slope away from the influence of the floor baffles in the basin. Since h_j decreases at the same time, the face of the jump becomes much flatter and the downstream leg of the uplift triangle becomes longer. These effects are shown in Figure 26, which gives L/d_2 versus D_2 / d_2.

At the design tailwater depth ($D_2 / d_2 = 1$) with the standard basin $L/d_2 = 3.5$. On a plain basin without floor baffles, with $D_2 / d_2 = 1$, the toe of the jump would be at the toe of the chute, for which $h_j = d_2 - d_1$ and $L/d_2 = 4.5$. When D_2 / d_2 reaches 1.5, the jump is so far up the slope that the floor baffles, now in a region of deep low velocity flow, become redundant. At this point the value of L/d_2 reaches its maximum value. Any further increase in tailwater depth simply moves the entire jump up the slope with no change in length. The length of the jump roller is somewhat longer than the L values from Figure 26. For example, when a jump occurs completely on a slope, the roller is about 6 d_2 in length. However, the last d_2 of length is essentially horizontal and does not contribute any area to the uplift triangle, so a value of 5 d_2 is appropriate for calculating uplift.

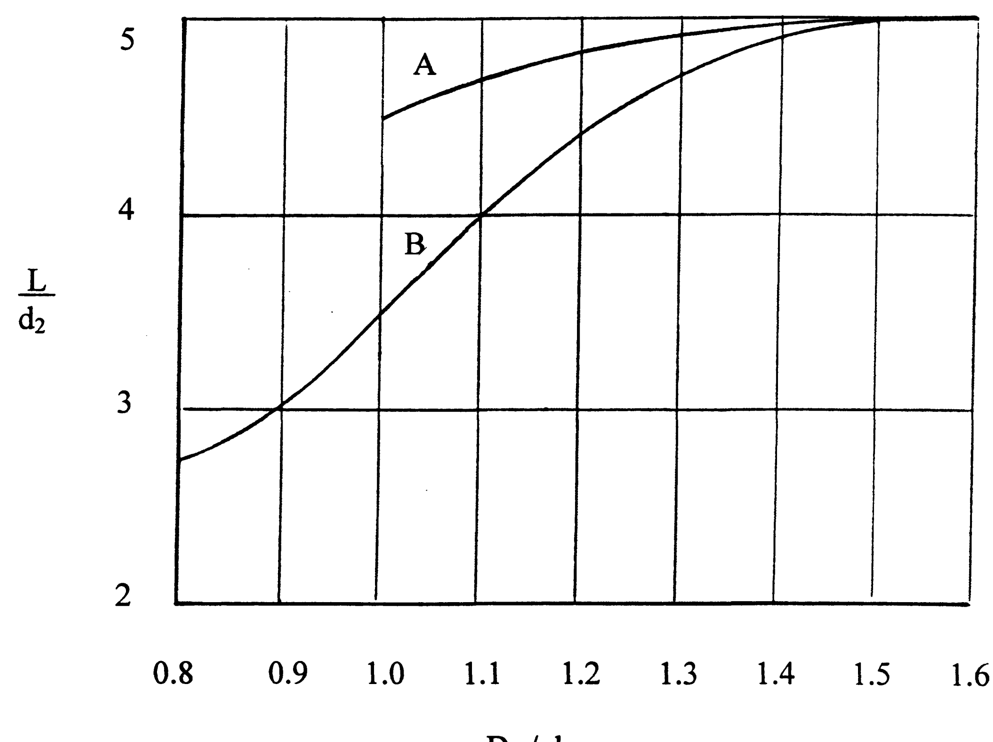


Figure 26. Length of Downstream Leg of the Uplift Triangle
A - Plain Basin ***B - 3d_2 Basin With Blocks***

It should be noted that apart from uplift considerations the slab required for the basin and the lower part of the chute on which the hydraulic jump may form usually must be thicker than for the conveyance part of the chute upstream from the terminal structure. Extra slab thickness is desirable to offset the vibration and stress reversal due to the dynamic loading associated with the jump. According to tests at the St. Anthony Falls (SAF) laboratory, reported by Toso and Bowers in 1988, pressure fluctuations above and below average on the floor slab just downstream from the toe of the jump could exceed $\pm 0.4 V_1^2 / 2g$ at a Froude No. of 6. A typical distribution from the SAF tests is shown in Figure 27. The average pressure on the floor follows the jump profile, and agrees with piezometer readings. Because of the inertia of the liquid in free standing piezometers, they are useful in determining time average values of the pressure. By the same token, they cannot be used to determine high frequency transient pressures. The pressure fluctuations were read with quick response electronic transducers. The peak of the positive pressure fluctuation is greater than the tailwater level and the negative fluctuation results in sub-atmospheric pressure on the floor.

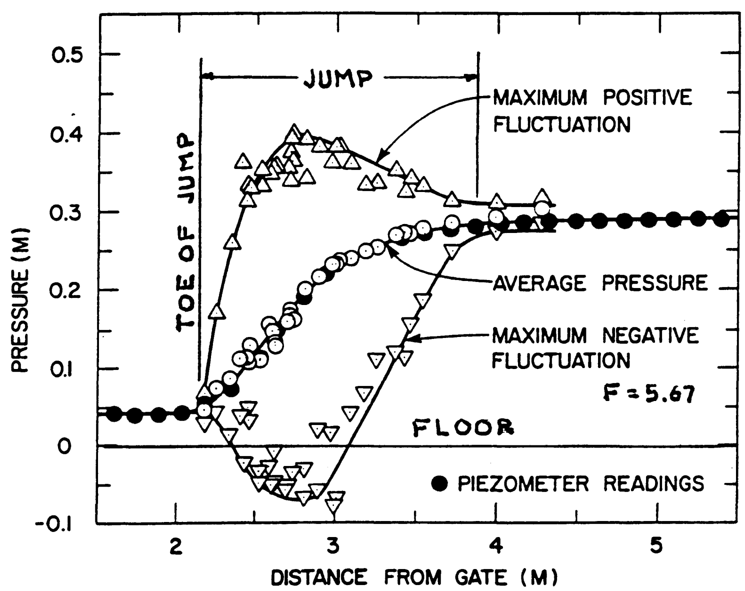


Figure 27. Pressure Fluctuations Under a Hydraulic Jump (after Toso and Bowers)

The upper and lower pressure curves on Figure 27 are envelope curves. It is important to note that the pressure distribution under a jump at one instant of time will show both high and low pressures at different positions across the jump and in the direction of flow. It is not possible to have either the envelope high pressure or envelope low pressure over the full length and width of the jump at the same time. The average load on the slab as a whole will correspond to the time average value, but to provide stability to offset the effects of transient pressures, it is customary to use extra slab thickness. A minimum slab thickness of 0.125 d_2 is recommended (i.e., 1.25 m for a 10 m sequent depth). A 2.4 m slab for a 16 m jump was used for Gardiner Dam spillway. A

2.4 m slab was also used for Imperial Dam spillway (Figure 1, Chapter 5, page 5-3) for a 5.5 m jump, but in this case the slab design was intended to resist the entire uplift by slab weight alone.

The required slab stability for uplift can be obtained by slab weight alone, anchorage, or pressure relief. Slab weight alone would require a slab thickness of 0.5 h_j, which is more than double the 0.125 d_2 thickness required for the dynamic loading of the hydraulic jump. It is common practice to taper the slab to a reduced thickness at the end of the basin where the potential uplift and the dynamic load each become insignificant. Also the chute slab thickness can be tapered to the normal chute slab thickness at the level of the maximum tailwater. The minimum slab thickness should not be less than 300 mm, and this may govern on smaller structures.

Anchorage can be used to resist the residual uplift force not offset by slab weight. On rock foundations steel anchor rods can be used. These are grouted into the foundation and tied into the slab reinforcement before the slab is poured. On clay foundations belled anchor piles may be used to mobilize part of the weight of the foundation.

Pressure relief may be achieved by a pumped drainage system or by eductors. By either method the basin and chute area where pressure relief is required is surrounded by a perimeter cutoff. The cutoff is intended to minimize seepage inflow from the tailwater. By the pumped drainage system method a drainage collection system inside the cutoff discharges the seepage by gravity flow to a sump located at the stilling basin sidewall. The water level in the sump is maintained at a level below the basin floor slab by automatic level controlled sump pumps. Usually 2 pumps are used - one at each sidewall. By this method the uplift pressure is eliminated under the chute and basin under all conditions of operation (i.e., with or without flow over the spillway). Pumped drainage systems are usually restricted to major multi-million dollar spillways where on-site operators will be available on a daily basis for monitoring performance of the dam and appurtenant structures. A pumped drainage system was used for Gardiner Dam spillway stilling basin.

30. **Pressure Relief by Eductors**

The gravity drainage system used for the chute, as described in Section 20 (page 2-27), cannot be used for the portion of the chute slope located below the maximum tailwater level. Such a system would allow direct transmission of the tailwater pressure to the underside of the slab. Thus a drainage system for the terminal structure must be completely separate and isolated from the chute drainage system. In the absence of sufficient slab weight, anchors, or a pumped drainage system, pressure relief for the area below the maximum tailwater may be achieved by eductors.

Eductors have a long history of application. They were used in 1932 on the downstream face of the Imperial Dam hollow overflow section to evacuate seepage which might otherwise collect inside the dam (see Figure 1, Chapter 5, page 5-3). In the 1940's it became common USBR practice to connect the stilling basin sub-floor drainage system to horizontal pipes running through the chute blocks and terminating at the downstream face of the blocks. The rationale for this approach was that the sub-atmospheric pressure which occurs at the downstream face of the chute blocks when the spillway is operating would lower the pressure under the floor slab. This rationale is quite correct when the jump forms near the bottom of the chute, such as might occur at maximum discharge. However, in the case of a B-jump the chute blocks would be submerged under the jump, and subjected to lower velocities and greater depths, and in the case of a C-jump or D-jump the full tailwater pressure could be transmitted to the underside of the chute slab through the chute block outlets. In these cases an additional row (or rows) of eductors may be required at an upstream position.

An eductor is a small horizontal pipe which exits through the slab into the high velocity flow on the slab. The suction at the eductor outlet lowers the pressure under the slab, even though the eductor may be located below the tailwater level. When there is no discharge but water is ponded in the stilling basin, any eductor below the still water level will admit this pressure to the underside of the slab. However, there will be no unbalanced pressure because of the opposing pressure on top of the slab due to the water weight. Thus, the only time the eductor operates is when the spillway is passing discharge, but this is the only time it is needed.

A design for a chute eductor is shown in Figure 28. Since each lateral drains only a local area, a few small eductors is all that is needed. One 50 mm eductor for each 5 m of lateral is recommended. A vertical standpipe behind or imbedded in the sidewall is located at each end of the lateral drain. The top of the standpipe may be turned into the chute above the tailwater level and covered with a screen to prevent entry of birds or foreign objects. The lateral and standpipe should be 100 mm in diameter for chutes up to 25 m wide, 150 mm for chutes up to 50 m wide, and 200 mm for wider structures. PVC can be used for all piping components except the horizontal leg of the eductor, which should be galvanized steel pipe.

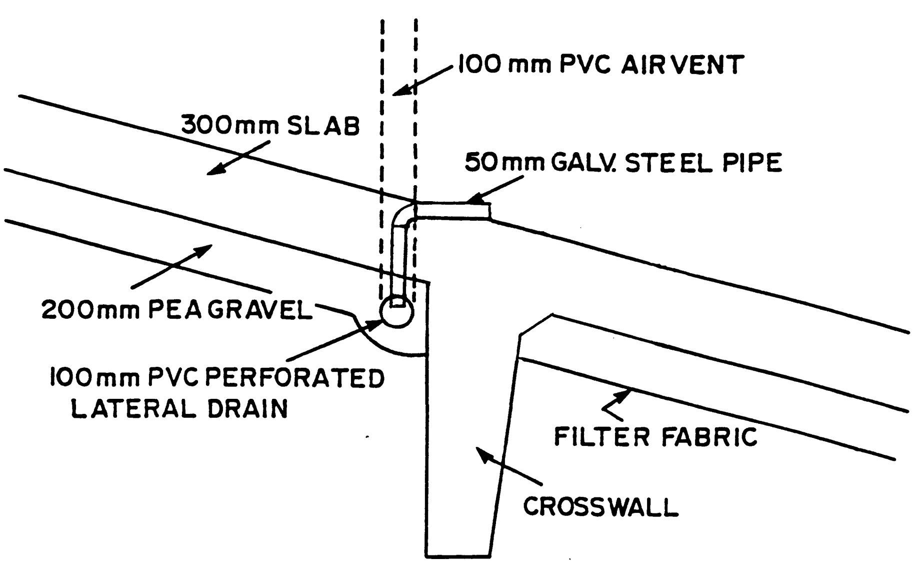


Figure 28. Eductor Design

The eductor should extend horizontally through the sloping slab so that the crown of the pipe projects above the floor slab at least one-fifteenth of the jet flow depth over the eductor. If the flow depth is large, this requirement may require an eductor pipe larger than 50 mm diameter. Such a projection will induce a pressure drop at the end of the eductor when discharge is passed down the chute.

The purposes of the standpipe are twofold. First, the standpipe acts as an air vent, admitting air to the lateral and the eductors whenever the pressure at the eductor outlet drops below atmospheric pressure. This prevents development of suction pressures in the drainage layer below the slab. Second, the standpipe acts as a surge relief well when the eductor outlet is subjected to highly transient pressures, such as will occur when the eductor becomes submerged under a hydraulic jump.

Some designers have been reluctant to place any kind of a drain under a hydraulic jump because of their concern about the effect of fluctuating pressures in the drainage system under the slab caused by turbulent pulsations in the jump. In this respect, the addition of air vents as a component of the system is of great importance. Basically the transients in the drainage system are eliminated and the sub-floor pressure is stabilized at the level at which the water is standing in the standpipe.

As with the pumped drainage system, the area to be drained must be surrounded by a perimeter cutoff. Basically the downstream cutoff must extend along each side of the structure under the sidewalls, and across the chute at the upper end. In addition to the perimeter cutoff, a crosswall will be required under the sloping portion of the chute for each row of eductors that is required, and must be located just downstream from the eductors, as shown in Figure 29. The purpose of the crosswall is to prevent a high rate of seepage up the slope through the drainage layer from a downstream lateral which could be under the jump and operating under a higher pressure.

Figure 29 shows a layout for a two row eductor system. Eductors are not required upstream from the highest position of the toe of the hydraulic jump because the entire portion of the chute upstream from this position will have atmospheric pressure under the floor slab, regardless of the depth of submergence below the tailwater level.

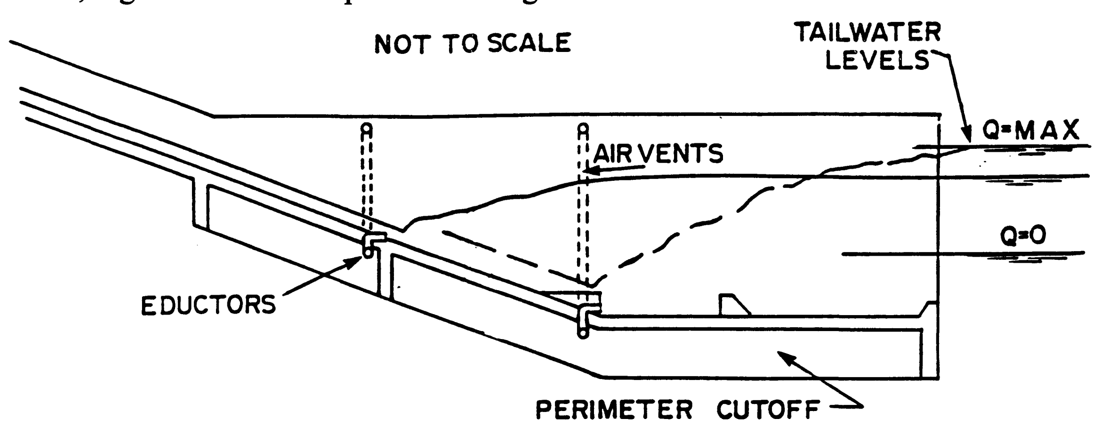


Figure 29. Layout for Pressure Relief by Eductors

The average pressure head y_p at an eductor outlet is given by

[38] $$y_p = y - C_p v^2/2g$$

in which y is the depth of the eductor below the water surface of the jump, v is the jet flow velocity at the position of the eductor, and C_p is the pressure coefficient. According to experiment $C_p = 0.16 + 0.22\ (S - 0.1)^{1/2}$, where S is the chute slope on which the eductor is located. At the toe of a jump, $y = d_1$ and $v = v_1$, and y_p will be negative. For example, with $S = 0.2$, $d_1 = 0.5$ m, and $v_1 = 15$ m/s, $y_p = -2.13$ m. The eductor will suck air and the actual pressure in the drainage system will be atmospheric. Generally the underslab pressure will not become positive until the toe of the jump forms at least 1 d_2 upstream from the eductor outlet.

The pressure head at a row of eductors under the jump, including the eductors at the face of the chute blocks, can be calculated from [38] by estimating y and v from a sketch of the jump profile and jet depth under the roller. In this way the reduced uplift force due to eductor action can be determined. All things considered, it is recommended that a factor of safety of 1.3 be applied in uplift calculations. In other words, the submerged weight of slab which must offset the unbalanced uplift force should be 1.3 times this force. Normally the recommended slab thickness of 0.125 d_2 would be sufficient.

F. OTHER SEPARATE SPILLWAYS

31. <u>Side Channel Spillway</u>

A side channel spillway is a separate spillway built at the end of a dam in a narrow canyon. It is used where high or steep abutments will not accommodate a chute spillway, and the dam itself is not adapted to the use of an overflow spillway. Flow enters the spillway over a gated or free weir crest which is parallel to the direction of flow in the side channel, as shown in Figure ***30***. The flow must change direction approximately 90° after passing over the weir. The weir can be designed according to the same principles discussed for overflow spillways.

The flow over the weir is collected in the side channel and discharged into a narrow chute or tunnel. Since the flow changes direction approximately 90 degrees after passing over the weir, the weir crest length may be selected largely independent of the chute width or tunnel diameter. Almost the entire velocity head produced by the drop over the weir is dissipated in turbulence in the side channel, and new velocities must be developed parallel to the weir. The side channel must be deep enough to provide the necessary head to overcome friction and produce the necessary velocity. Usually the channel cross section increases in area in the downstream direction in proportion to the increase in discharge.

Flow from the chute or tunnel is usually discharged as a free jet above tailwater level. A saucer shaped flip bucket has been developed which has proven to be quite effective for tunnel discharge. The saucer shape is intended to disperse the jet by spreading it laterally as well as throwing it horizontally away from the tunnel portal.

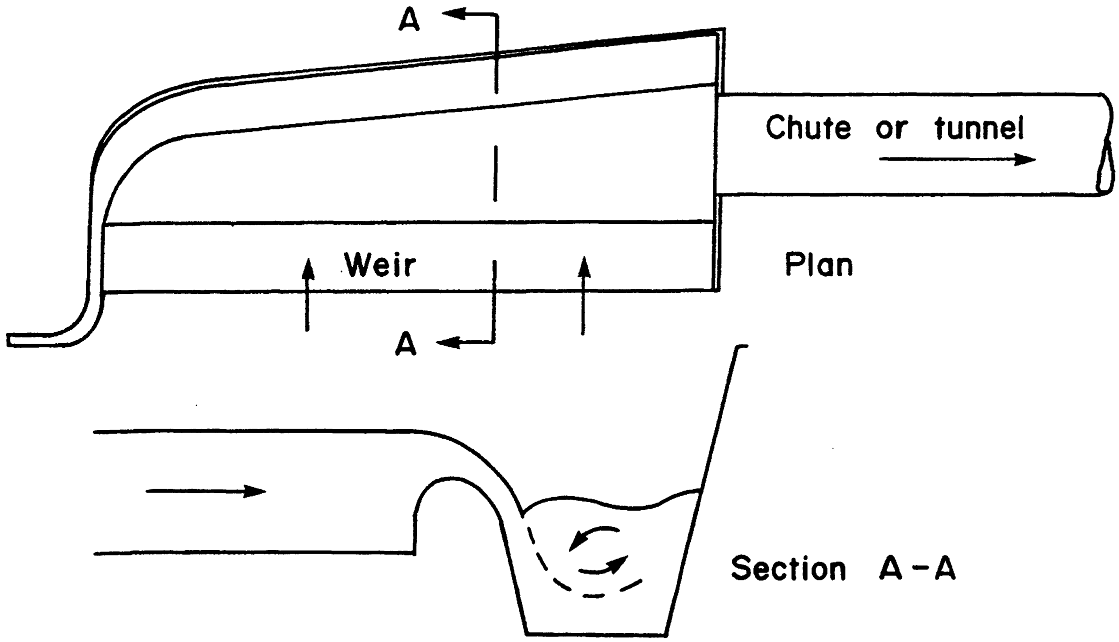


Figure 30. Side Channel Spillway

The side channel spillway and energy dissipator is a specialized structure with limited application. It is advisable to model test all designs for important dams.

32. Shaft Spillway

The shaft spillway is a separate spillway which may be used as an alternative to a side channel spillway. It consists of a circular concrete weir which discharges flow into an inclined or vertical shaft, as shown in Figure 31. The shaft is connected to a horizontal tunnel by a long radius bend. This type of spillway is often economical where a tunnel has been necessary for diversion purposes, and the same tunnel can ultimately be used for the spillway.

The circular weir is usually a high coefficient streamlined weir. The diameter of the weir at the crest is necessarily greater than the diameter of the shaft and tunnel in order to accommodate the shape of the nappe. Furthermore, since the nappe must cling to the weir in order to discharge down the vertical sides of the shaft, subatmospheric pressures on the weir crest cannot be avoided. The crest length can be taken as πD minus the width obstructed by piers, if any. The coefficient will not be quite as high as for a straight weir of the same shape because of the convergence of the flow.

It is not customary to design a high head shaft spillway for full flow operation. The control point for the discharge may switch back and forth from the crest to some point inside the shaft, producing unsteady flow and large fluctuating dynamic loads on the tunnel, accompanied by air gulping, noise, vibration and excessive spray from compressed air at the outlet. Deflectors and air vents near the inlet can be used to insure free surface flow throughout the tunnel, as shown in Figure *31*. A high drop in combination with a steep slope can produce a very high velocity (up to 50 m/s has been observed on some spillways), so special attention must be given to prevention of damage from cavitation.

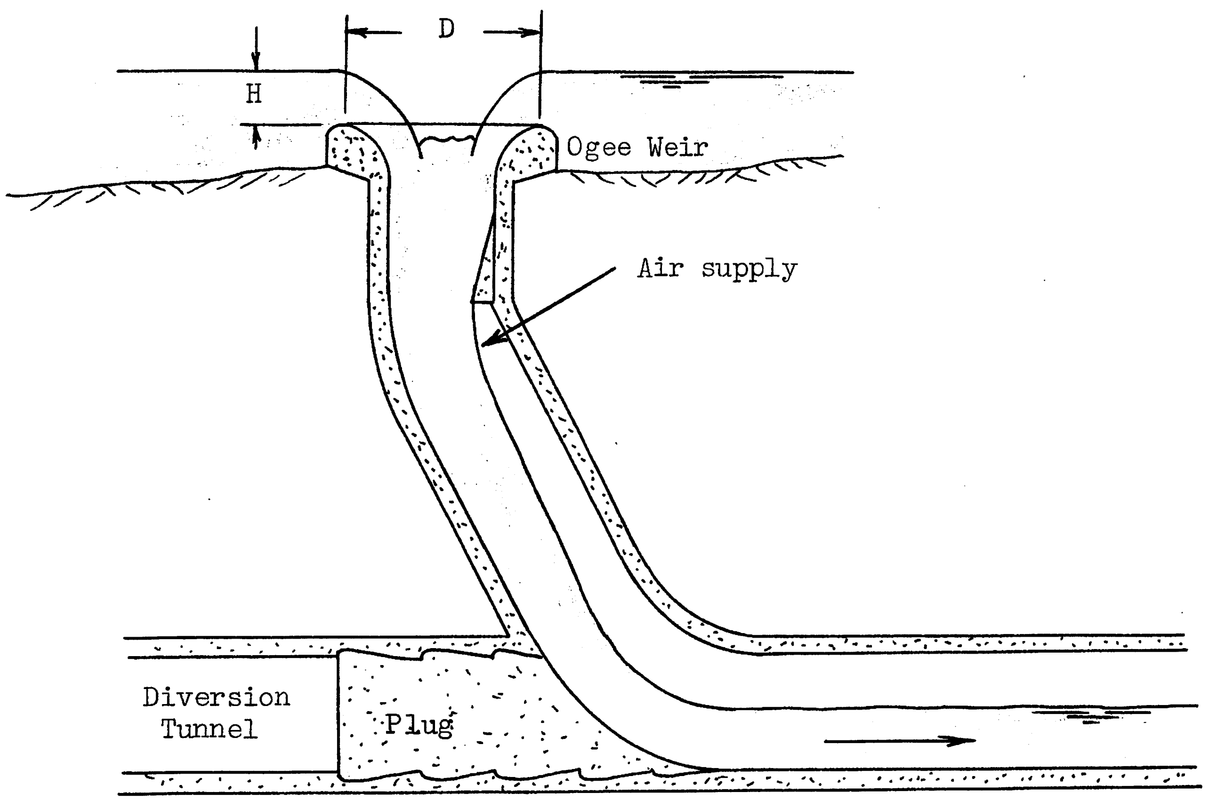


Figure 31. Shaft Spillway

G. RECTANGULAR BOX INLET SPILLWAY

33. <u>Description</u>

This spillway is an adaptation of the side channel spillway to smaller less important structures. The form is governed to some extent by simplicity of design and construction. All surfaces are plane, so that construction in reinforced concrete is relatively simple. A schematic layout is shown in Figure *32*, and an actual operating spillway in Figure *33*.

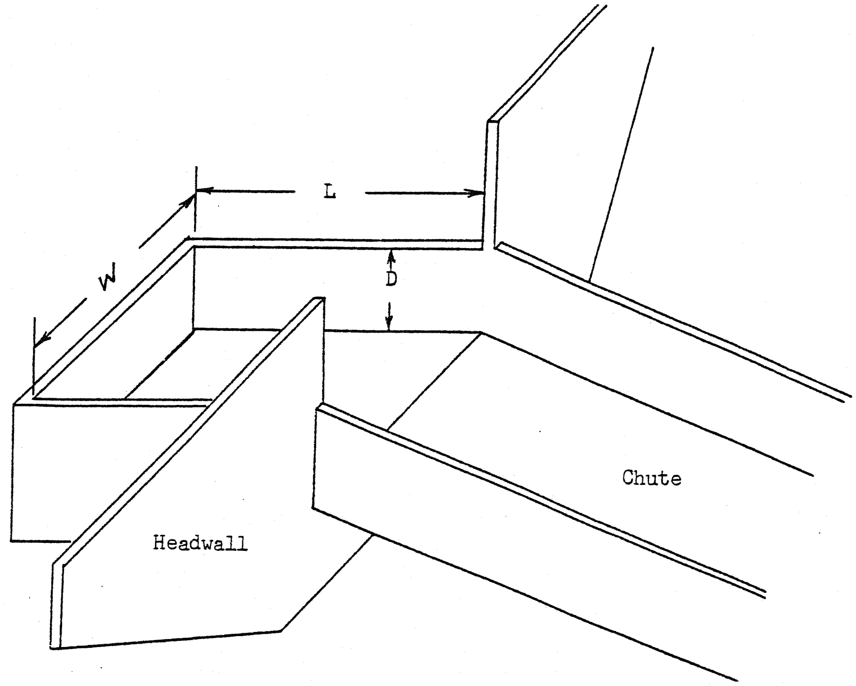


Figure 32. Rectangular Box Inlet Spillway

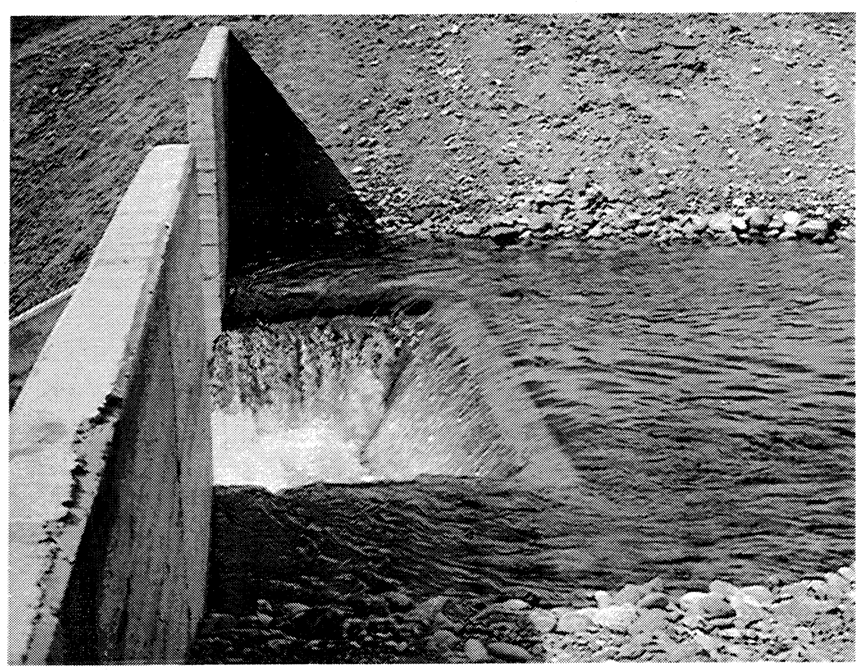


Figure 33. Box Inlet Spillway in Operation

Flow is allowed to spill over three sides of a rectangular box which acts as a three sided weir. The fourth side is open and discharges the flow into a chute. The structure is uncontrolled and discharges automatically as soon as the reservoir rises above the crest elevation of the box. It is particularly adapted to smaller reservoirs where the permissible head rise is limited. The crest length required for an uncontrolled spillway increases greatly if the available head is limited. If a single straight crest is used across the chute, the chute would be excessively wide and uneconomical. The rectangular box inlet allows the crest length and chute width to be selected independent of one another. The crest of the inlet is frequently placed at the same elevation as the bed of the spillway approach channel, but the bed may be lowered if it is necessary to further reduce the approach channel velocity. The chute may terminate in a conventional hydraulic jump stilling basin.

It is important that uplift be considered in the design of the box inlet. When the reservoir is at the crest of the spillway, the weight of the empty box will not balance the uplift force underneath the floor unless the uplift pressure is reduced to less than reservoir pressure. This can be achieved by a cutoff around the perimeter of the box, and a drainage system under the floor inside the cutoff.

34. Crest Length

The crest length is taken as the inside dimension of the three sides of the box. The discharge equation is

$$Q = C(W + 2L)\, H^{3/2} \qquad [39]$$

in which H is the total head, including velocity head. There is no upper limit to the value of L as long as the box is deep enough to carry the flow away. Theoretically there is no lower limit either, but ordinarily the design would not be used unless $L/W \geq 0.5$.

The bottom width of the approach channel must be at least equal to the crest length (W + 2L), otherwise the side slopes will interfere with the flow coming over the sides of the inlet and reduce the discharge capacity. For example, if the bottom width was equal to W, flow over the sides would be essentially eliminated. Model tests have shown that the weir coefficient C is a function of H/W and L/W, and varies over a practical range of 1.80 to 1.96. The coefficient will increase as H/W increases, and decrease as L/W increases. The reason for the decrease is that the effective head on the sides does not include the velocity head of approach, so as W becomes a smaller part of the total crest length, the coefficient must be reduced to compensate. With an approach channel of adequate width, and the channel bed flush with the crest of the box inlet, the coefficient is given by

$$C = 1.96\,[1 - 0.056\sqrt{(L/W - 0.5)} - (0.06 - 0.1\,H/W)^2] \qquad [40]$$

This value of C applies within the limitation $0.1 < H/W < 0.6$ and $L/W > 0.5$. However, given Q, W, and H, the required value of L can be solved for design purposes using the basic coefficient as 1.88. (This is exact for $L/W = 1.0$ and $H/W = 0.6$).

35. Depth of Box

If the box is too shallow it will not be capable of discharging the indicated flow without excessively submerging the nappe and increasing the head. In this case the space discharge control for the inlet would be at the headwall. This can be avoided by making the box deep enough to avoid excessive submergence. At the design condition the energy loss which occurs in the box inlet due to the mixing as the flow drops in, is about 0.25 H. The remaining energy of flow must be sufficient to pass the flow out of the open end of the box at critical depth d_c. For critical flow the total energy is 1.5 d_c, so the energy equation written between the reservoir and the headwall, with the floor of the box inlet as datum, becomes

[41] $$H + D = 1.5\, d_c + h_L$$

Since $d_c = (q^2/g)^{1/3}$, $q = Q/W$, and $h_L = 0.25$ H, then

[42] $$D \geqq 0.701\, (Q/W)^{2/3} - 0.75H$$

Designs made on the foregoing basis will be satisfactory for discharges up to about 30 m^3/s without the need of a model test.

H. DROP INLET PIPE SPILLWAY

36. Description

This spillway is an adaptation of the shaft spillway to smaller less important structures. The shaft spillway has a circular ogee weir crest and a smooth long radius elbow. The complicated construction associated with such three dimensional curves cannot be justified on smaller projects. A simpler design, herein referred to as the drop inlet pipe spillway, has been developed from model tests. It is suitable for discharges up to 30 m^3/s, velocities up to 10 m/s, and embankment heights up to 10 m. Because of the smaller heads involved, and hence lower velocities, the drop inlet can be designed to flow full at maximum flood.

The drop inlet pipe spillway may be used where the valley walls are not suitable for chute spillway construction. The pipe, which may be a corrugated metal or a reinforced concrete pipe, can be placed in a shallow excavation anywhere under the embankment. A formed reinforced concrete drop inlet structure, square in plan, is simply placed at the inlet of the pipe. The lip elevation of the drop inlet is at the reservoir full supply level. All construction is in the open, in contrast to the case for the shaft spillway. At the outlet a stilling device is required. The most common type of outlet structure for this situation is a transition and stilling basin, as discussed in Chapter 3.

A section view of the drop inlet with significant dimensions is shown in Figure *34*. Some of the features of the drop inlet are the curved lip at the crest, the sloping fin in the bottom, the 45 degree bevel on the lip at the pipe entrance, and the footing. The curved lip is used to stabilize the flow coming over the drop inlet by allowing the flow to cling to the sides of the inlet. The fin is necessary to redirect the side flow jets in an axial direction and prevent spiral flow in the pipe. The bevelled pipe entrance reduces the head loss. The footing is required to prevent flotation of the drop inlet due to uplift pressures.

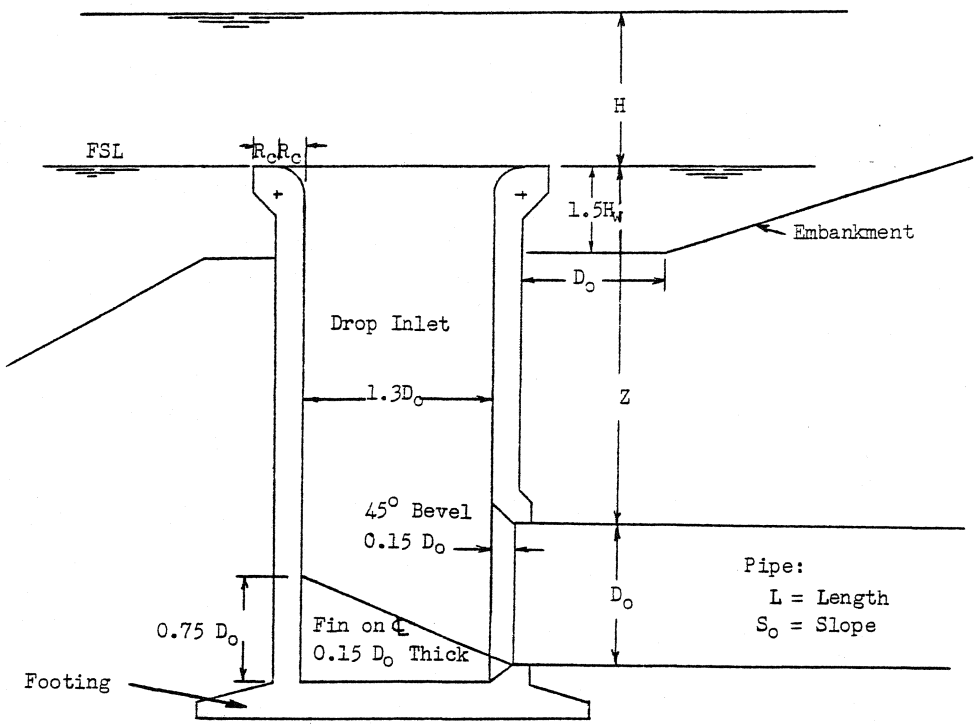


Figure 34. Definition Sketch of Drop Inlet Pipe Spillway

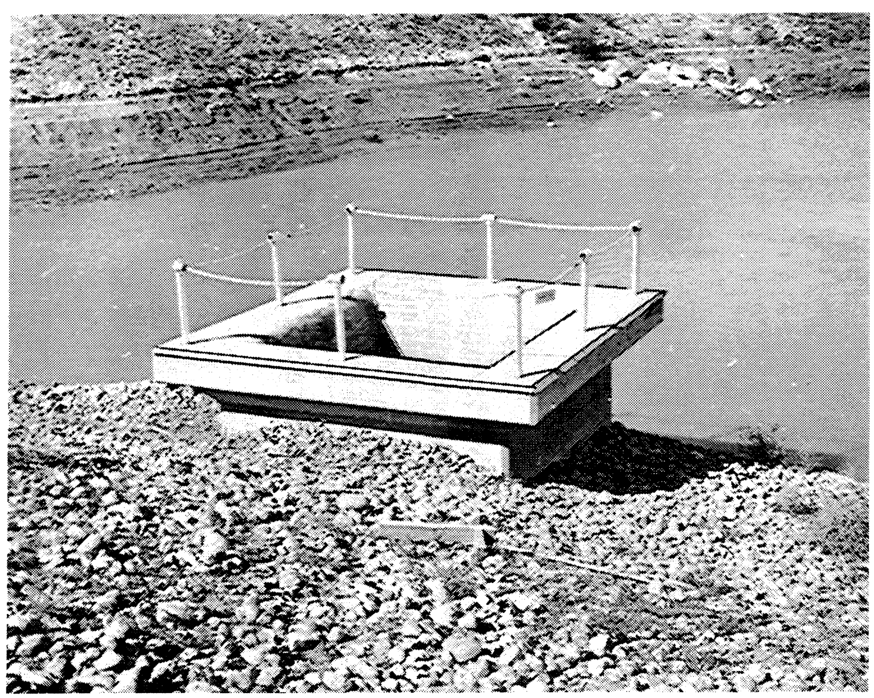


Figure 35. Crest of a Drop Inlet Pipe Spillway

This is most critical when the discharge is zero and the drop inlet is empty. The footing should be designed to mobilize part of the weight of the embankment above it. Figure 35 shows a prototype example of a drop inlet.

37. Weir Flow

As the reservoir rises flow will spill over the four sides of the drop inlet. The discharge is dependent on the head H on the lip, and may be calculated by a conventional weir equation. The inside perimeter of the drop inlet ($5.2\ D_O$) may be taken as the crest length for the weir. Hence,

[43] $$Q = C\ 5.2\ D_O\ H^{3/2}$$

The coefficient C depends upon both the head and the crest curve radius R_C, in accordance with

[44] $$C = 2.20\ (1 + 0.5\ R_C/D_O + 0.04\ H/R_C)$$

The value of C in [44] would appear to be unrealistically high when compared with the coefficient for an ogee weir. The reason is that the true effective crest length is considerably greater than $5.2\ D_O$, and therefore the true coefficient is much smaller. For example, at low heads the effective crest would correspond more closely to the length around the start of the crest curve rather than the inside perimeter of the inlet. In the strict sense, both the effective crest length and the coefficient vary with the head, but it is more convenient for design purposes to use a constant crest length and put all the variation in the coefficient, as indicated in [44].

In [44], the second term in the bracket, $0.5\ R_C/D_O$, accounts for the increase in the coefficient due to the increase in effective crest length as the radius of the curve increases. The third term, $0.04\ H/R_C$, accounts for the increase in the coefficient due to the development of negative pressure under the nappe, which clings to the inside of the drop inlet after passing over the curved lip at the crest. Negative pressures must develop because any curve which is tangent of a vertical surface will be steeper than a natural nappe trajectory. The negative pressure, and the coefficient, will increase for either an increase in head or a decrease in curve radius, thus accounting for the form of the last term.

If the radius of the curve is too short the pressures will drop below the allowable limits and the nappe may separate from the sides of the inlet. The lowest pressure on the lip may be calculated from

[45] $$P/\gamma = -0.8\ H_W^2/R_C$$

in which H_W is the maximum weir head which can occur, that is, it is the value of H at the flood out point. Although H will exceed H_W at the design flood, the drop inlet will be flowing full throughout and pressures on the lip will become positive. It is usually desirable in hydraulic structures to limit the minimum pressure to about -2m. The expression for the minimum allowable radius then becomes

[46] $R_c \geq 0.4\, H_w^2$

It is recommended that a radius of not less than 0.2 D_o be used for preliminary design, and that this be subsequently increased, if necessary, in accordance with the requirement of [46].

38. Pipe Flow

The flow spilling into the drop inlet creates a pool in the riser. The depth of the pool is such that it provides the required head to pass the discharge coming over the weir through the pipe. As long as the level of this pool is below the lip of the drop inlet, the discharge is determined by the weir head alone.

As the discharge increases the pool depth will also increase, until eventually the pool will submerge the nappe and the system will flow full throughout. The discharge must then be computed from a consideration of the head losses in the system and the total differential head h. Weir flow as such ceases to exist. This flow condition is referred to as pipe flow.

For weir flow Q varies as $H^{3/2}$ and for pipe flow Q varies as $h^{1/2}$. Hence, there is an abrupt change in the discharge capacity curve at the point of transition. This point is called the flood out point and is indicated in Figure *36*. The location of the flood out point on the discharge capacity curve may be found quite simply by computing both the weir flow and the pipe flow capacity curves and determining the point of intersection.

If the outlet is unsubmerged and the pipe flows full at a Froude number greater than 1.8, the effective position of the hydraulic grade line at the outlet may be assumed at the centerline of the pipe. This is usually the case and the differential head is measured from the centerline of the pipe at the outlet to the reservoir, as in

[47] $h = H + z + S_oL + D_o/2$

in which D_o, S_o, and L are the pipe diameter, slope, and length, and z is the height of the riser. This same head is equal to the summation of the losses plus the outlet velocity head, as in

[48] $h = h_e + h_f + V^2/2g$

The entrance loss through the drop inlet, h_e, is approximately 0.3 $V^2/2g$, and the pipe friction loss, h_f, may be computed from the Darcy-Weisbach equation, from which

[49] $h = (0.3 + f\, L/D_o + 1.0)\, V^2/2g$

Solving [49] for V, and noting that Q = VA, there results

[50] $Q = 0.785\, D_o^2 \sqrt{2gh/(1.3 + f\, L/D_o)}$

The discharge through the system will be given by [43] or [50] whichever is smaller.

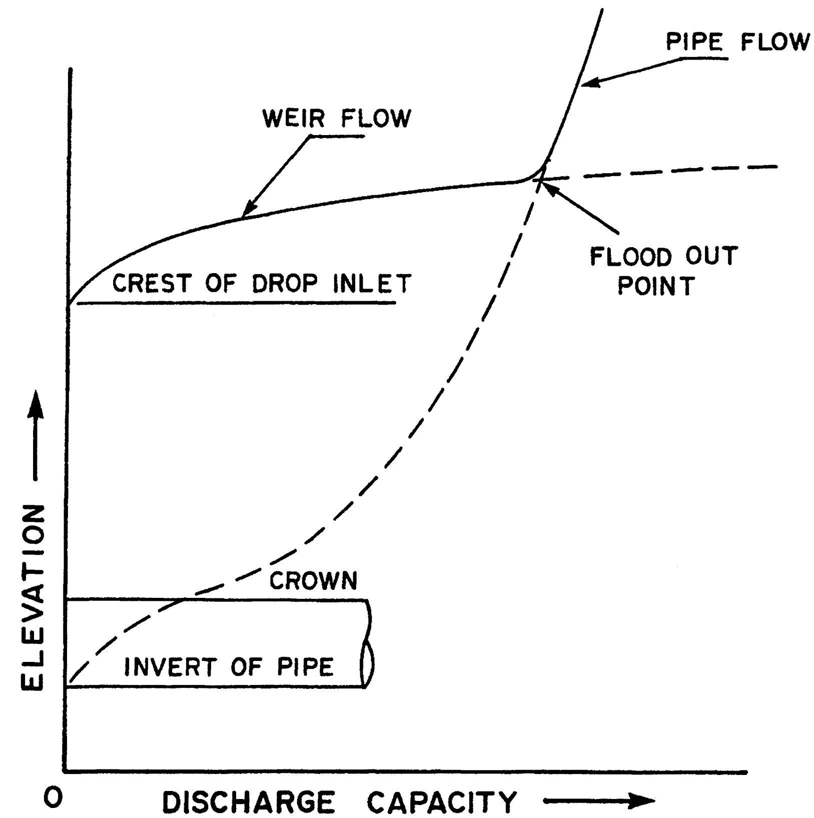


Figure 36. Discharge Capacity Curve for a Drop Inlet Pipe Spillway

A feature of any closed conduit type of spillway, including the drop inlet pipe spillway, is that the reservoir will rise very rapidly if the discharge continues to increase after the flood out point is reached. In order to reduce the risk of overtopping the embankment, it is necessary to have an emergency overflow channel to release this excess discharge. This may take the form of an inexpensive shallow earth cut around one end of the dam, set at an elevation to start operating soon after the drop inlet floods out. A fuse plug dam may be placed in the excavation and designed to fail when overtopped. Overtopping should occur infrequently, perhaps once each 50 years, depending on the design frequency selected. It is to be expected that there will be considerable erosion in the channel when it is called upon to operate.

39. Slug Flow

Slug flow is the terminology applied to that condition of flow wherein large quantities of air are intermittently drawn into and carried away by the pipe. This phenomenon is peculiar to the case of a steep pipe (a pipe for which the slope S_0 is greater than the slope of the hydraulic gradient), and will only occur during the weir flow phase of the discharge. The pipe alternately flows full and partly full, so that the effective head alternately increases and decreases, and the flow discharges in bursts or slugs (i.e., unsteady). When the pipe flows full, the effective head, measured to the outlet end of the pipe, is more than required to discharge the weir flow coming over the lip of the drop inlet. The level of the pool in the riser of the drop inlet then drops in order to supply this temporary excess discharge capacity. When the pool drops below the level of the crown of the pipe, air is admitted, breaking the syphonic action. The effective head abruptly decreases so that the pipe capacity is less than the weir discharge. The pool level in the drop inlet then begins to rise, submerging the pipe inlet, and causing the pipe to flow full again. The cycle is then repeated.

Slug flow is accompanied by noise, vibration, and vertical surging in the inlet, and unsteady flow at the outlet. It should be avoided on all except smaller unimportant works, and will not occur if the pipe slope is equal to or less than one-quarter of the friction slope corresponding to the discharge at the flood out point. This limiting slope may be calculated by Manning from

[51] $$S_o \leq V^2 n^2/(4R^{4/3})$$

or by Darcy-Weisbach

[52] $$S_o \leq fV^2/(8gD_o)$$

Ordinarily the pipe slope is made as steep as possible in order to minimize the height required for the drop inlet. Some slope is always desirable to insure that the pipe will be free draining.

A further condition which can result in unsteady flow is a premature sealing off of the drop inlet. Tests show that at $H = 0.7\ D_o$ the nappes from opposing sides of the inlet will meet and seal off the inlet whether the natural flood out point has been reached or not. If it is found that the drop inlet will not flood out before $H = 0.7\ D_o$, the need for a larger drop inlet is indicated (i.e., $> 1.3D_o$). This condition could arise if the vertical height of the drop inlet is large.

40. Design Procedure

A friction factor (or Manning's n value) is assumed for the pipe. Given the design discharge and the corresponding differential head h, [50] must be solved for the diameter D_o, after which the friction factor can be checked, and [50] reworked if necessary. It remains to check the limitations for R_c, H_w and S_o. With $R_c = 0.2\ D_o$, [44] may be solved for the weir coefficient. The discharge capacity curve is calculated and the head H_w at flood out is determined. The adequacy of the lip radius is checked by [46] and increased if necessary. The limitation $H_w \leq 0.7\ D_o$ is checked. Finally the maximum limiting slope is determined.

I. AUXILIARY SPILLWAY

41 General

An auxiliary spillway is a lower cost earth bypass channel intended to pass the excess discharge should the required outflow exceed the design capacity of the service spillway. It has no crest section with gates, no chute, and no stilling basin. It is used primarily with fill dams, particularly earth fill dams, where overtopping would result in loss of the dam and reservoir. On major projects, failure of the dam would usually result in catastrophic downstream flood damage, including loss of life, and would be categorized by the Canadian Dam Association as "high consequence". High consequence dams must be designed to discharge the probable maximum flood (PMF) without overtopping the dam. Design for the PMF is considered as necessary to reduce the risk to a practical minimum value acceptable for the project. Smaller design discharges (e.g., 0.5 PMF or 0.1% flood) may be acceptable for lower consequence dams.

Apart from downstream damage, the loss of the dam itself may represent a large economic loss. However, the loss of the reservoir may also result in severe economic loss. For example, if the reservoir is used for water supply for irrigation, or hydro power production, or cooling water for a thermal power plant, or in a mineral extraction process, the loss of this supply for a period of several years could have a major impact on the economy.

In the mid – 1900's it was common to refer to the bypass channel as an emergency spillway. This term is considered undesirable from a public perception point of view, and has been replaced by auxiliary spillway. This is quite appropriate for present day designs because these spillways now receive careful attention and are "engineered" designs.

Erosion of the bypass channel will occur and is expected during each use, and would be most severe for the PMF. The channel must be designed so that the erosion does not extend into the reservoir or present any risk to the dam itself. However, repeated use of the auxiliary spillway is not expected. In most cases the service spillway would be designed for the 0.1% flow (the 1000 year flood), so operation of the auxiliary spillway would be an extremely rare event. For example, for a 200 year useful physical life of a project, the chance of the outflow discharge exceeding the service spillway capacity would be only one chance in five in the 200 year period.

42 Channel Layout

The inlet to the bypass channel is located somewhere on the reservoir perimeter upstream from the dam. The channel may discharge over a broad flat area that slopes toward the river, or over a natural drainage course that leads back to the river. A typical layout is shown in Figure 37. In some cases site conditions may not be suitable for construction of an auxiliary spillway, such as high ground on the uplands adjacent to the dam. Where an auxiliary spillway cannot be used, the project design flood must be handled by the service spillway alone, as was the case for Gardiner Dam spillway.

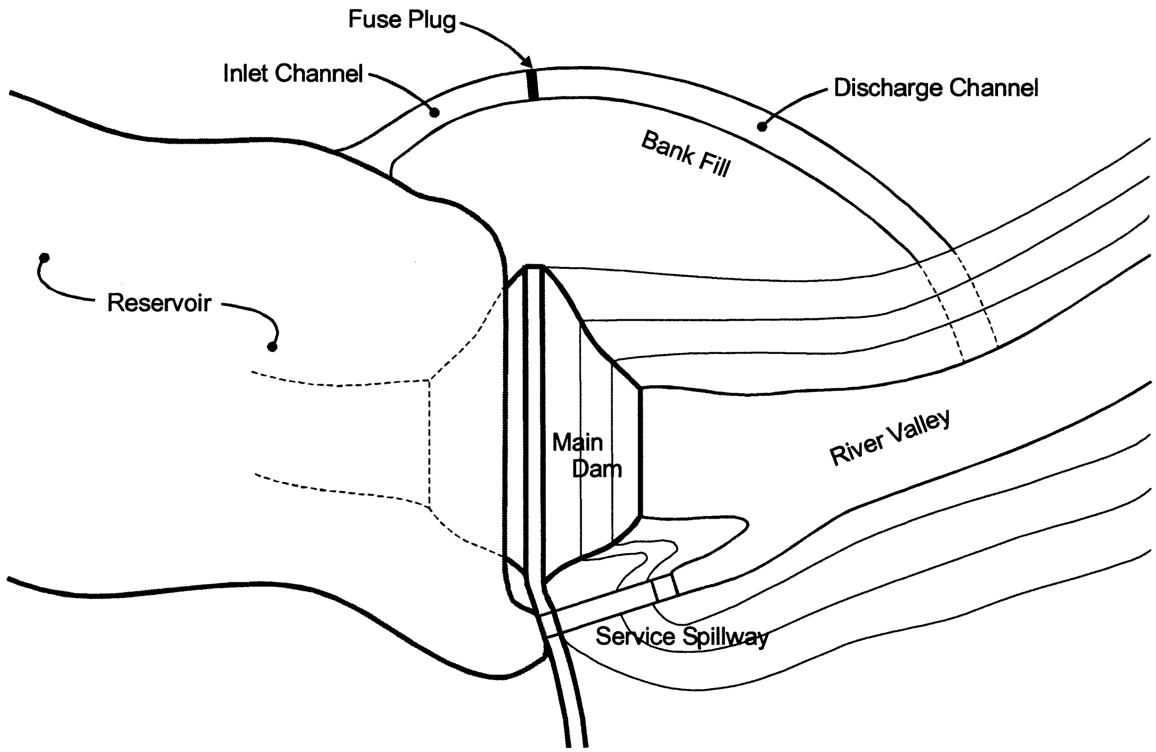


Figure 37. Layout for Auxiliary Spillway and Channel

A key feature of the auxiliary spillway is the fuse plug located at the control section. The fuse plug is a small dam designed to fail rapidly when overtopped by flood water, thus activating the discharge capacity of the auxiliary spillway. The fuse plug must be located where it will not be subjected to direct wave action from the reservoir because large riprap cannot be placed on the upstream face without compromising the need for rapid failure and washout. Typically, the size of the riprap on the face of the fuse plug would be 200 mm, which would be satisfactory for a 0.4 m wave, but the riprap on the face of the main dam, for example to resist a 2 m wave produced by a thousand year wind, would have to be about 1000 mm. Wave action in the inlet channel is naturally inhibited by the narrow width and shallow depth, and it is also beneficial to have a bend in the channel between the inlet and the fuse plug, as shown in Figure 37. The inlet channel should have zero slope, or even a small upslope, so the backwater upstream from the control section will result in somewhat lower velocities in this reach of channel.

The outlet channel should rejoin the river well downstream from the main dam. Erosion at the steep drop at the end of the channel will commence as soon as the spillway is activated. If the bank material is cohesionless there will be rapid loss of material, and a steep erosion channel will develop upstream from the drop. In cohesive material, the erosion response time will be much longer, and a vertical headcut will usually develop and work its way upstream, forming a gully in the discharge channel. If there is hard glacial till or a rock outcrop at the end of the channel, this may become a control point, reducing the progress of upstream erosion.

A point to be considered when an auxiliary spillway is used is the increase in tailwater level that will occur at the service spillway stilling basin when both the auxiliary spillway and the service spillway are operating. The excess tailwater will move the jump up the slope preceding the basin, as indicated earlier in Figures 22, 23 and 24, and will have implications regarding uplift, floor slab thickness, and sidewall height.

The most uncertain and most difficult aspect of the auxiliary spillway design is prediction of the rate of sediment loss from the channel. Often, the stratigraphy and sub-surface soil conditions along the length and breadth of the channel vary widely and are not fully defined by whatever foundation exploration that is undertaken.

Predictive relationships have been developed to estimate headcut advance in cohesive materials, but it is not possible to correctly account for variations in soil properties or predict the route along which the gully will develop. For example, due to the natural tendency for the discharge to concentrate in areas of lower erosion resistance where scour develops more rapidly, the headcut will usually develop to a width that is independent of the width of the discharge channel. Theoretically the discharge intensity would be reduced by widening the channel, but in practice this reduction will not necessarily occur. A reasonably long discharge channel downstream from the control section is the best insurance against headcut development all the way to the reservoir.

Erosion will also develop in the discharge channel upstream from the headcut. Channel slopes as little as 1 or 2% are usually enough to produce supercritical flow and erosive velocities. For a 3 m head at the control section, where it may be assumed that the flow passes at critical depth after the fuse plug has failed, the unit discharge would be 8.8 m^2/m and the velocity would be 4.4 m/s. This velocity would be highly erosive, particularly in cohesionless material, in which case the need for an erosion check at the downstream end of the control section would be indicated. This is discussed in Section 43.

It is necessary that the right bank of the discharge channel shown in Figure 37 be high enough to contain the discharge and prevent the possibility of a breakthrough which would allow the discharge to flow down the bank near the downstream side of the dam. Since it is not the practice to riprap either the bed or banks of the discharge channel, it is recommended that waste excavation material be placed in the area between the right bank and the dam. A large quantity of fill placed in this area will act as sacrificial material and minimize the risk of a breakout.

43 Fuse Plug Design

Typical details for a fuse plug are shown on Figure 38.

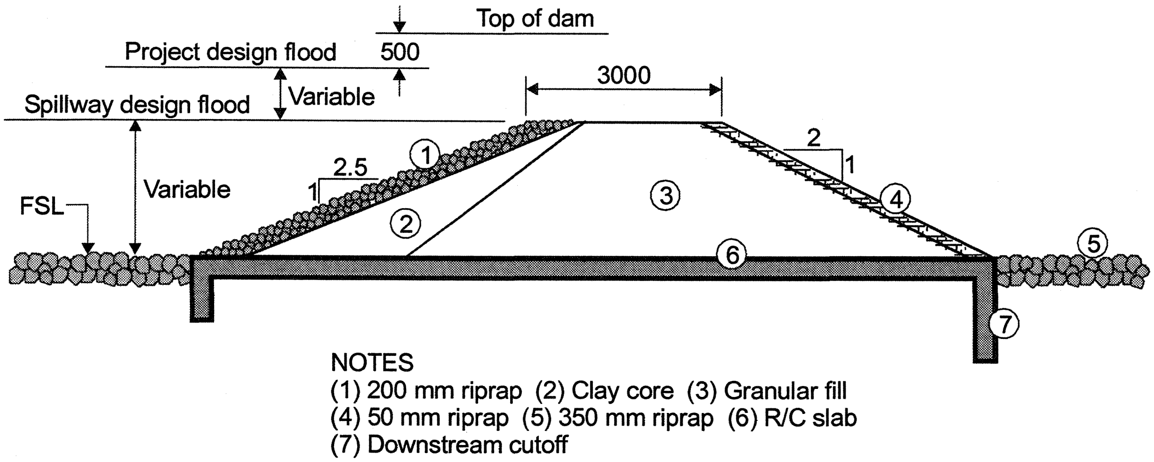


Figure 38. Fuse Plug Dam Cross-Section Details

When possible, the base of the fuse plug should be at or near the normal full supply operating level (FSL). In this way the fuse plug is not continuously exposed to the impounded water in the reservoir. Flood levels above FSL but below the overtopping level of the fuse plug may occur frequently. For these situations it is necessary to have riprap on the upstream face to resist small wave action, such as 200 mm riprap in a 400 mm layer. An inclined clay core (compacted select impervious material) is also required to minimize seepage and avoid the risk of a piping failure. The horizontal thickness of the core at the base of the fuse plug should be about equal to the height of the fuse plug. A gravel bedding layer (not shown) should be placed between the riprap and the clay core.

The body of the fuse plug consists of clean coarse granular material (sand and pea size gravel from 1 mm to 20 mm). This material will not retain water, and without nutrients, will not support the establishment of vegetation. A fuse plug design consisting of a solid clay core, which can become consolidated and heavily vegetated (circa 1950's designs) can be highly erosion resistant and could take days to wash out, which is unacceptable.

The downstream face of the fuse plug should have 50 mm riprap in a 100 mm layer. This layer is intended to prevent erosion of the downstream face during rainfall events. The face slope of 2:1 is just flatter than the angle of repose, which would be close to 1.5:1. It can be shown by rock slope stability analysis (Section 15, Chapter VIII) that 50 mm stone would have to be placed on a 6:1 slope to be stable with a 0.2 m overtopping head.

A design of this nature is expected to fail rapidly (within one hour) under the influence of an initial overtopping head of 0.2 to 0.3 m. As the erosion proceeds, the head increases and the failure process accelerates. The clay core actually does not have to be eroded since it breaks up and collapses due to loss of support underneath as the granular material is washed away.

The crest of the fuse plug should be at the elevation of the design flood level for the service spillway. A top width of 3 m (minimum) is required to accommodate service vehicles or other traffic. Some agencies do not require freeboard above the project design flood level. It is considered that design for the PMF itself is sufficiently conservative that it is too severe an imposition on the design to superimpose additional factors of safety. However, a nominal allowance of 0.5 m is often specified.

In the absence of an erosion resistant base such as a dense till, hardpan or rock outcrop, it is recommended that the base of the fuse plug be placed on a reinforced concrete slab, referred to as a hard sill. This slab is intended to prevent erosion at the ground level at the location of the fuse plug after the fuse plug has been washed out. This approach was used for the first stage fuse plug at Dickson Dam on the Red Deer River in Alberta, shown in Figure 39. The slab has also been placed on the side slopes at the ends of the fuse plug.

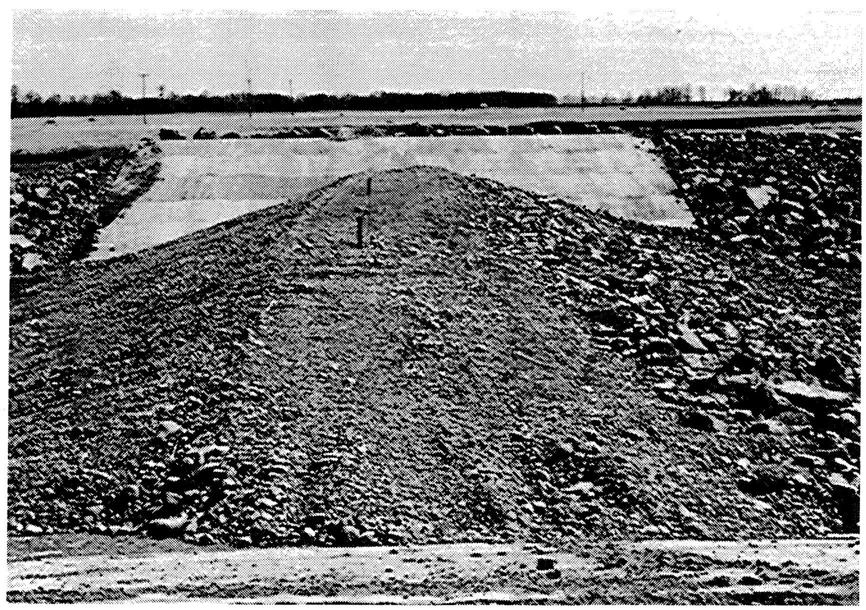


Figure 39. Fuse Plug Dam, Dickson Dam, Alberta (upstream face on the right)

Riprap at the downstream end of the slab is intended to resist erosion during the initial phase of the failure when water is running down the 2:1 downstream slope, and also to inhibit erosion of the bed and banks on the immediate downstream side of the structure. Riprap at the upstream and downstream ends of the slab may be placed at a nominal length of 3 times the height of the fuse plug. The downstream cutoff is intended to prevent undermining of the slab and delay any tendency for the developing scour in the downstream channel to progress upstream. On some designs where significant sheet erosion may be expected in the channel just downstream from the location of the fuse plug (such as for a cohesionless channel bed), an inclined slab may be run at a slope several metres below the initial bed level. At the Twin Valley Dam on the Little Bow River, where significant erosion would be expected in the cohesionless surficial soils in the discharge channel, a roller compacted stepped concrete slab, placed in overlapping horizontal layers at a 2:1 slope, was placed at the downstream end of the control section.

44 Multi-Stage Fuse Plugs

Once the fuse plug has breached, the discharge in the auxiliary channel is uncontrolled and will continue until the reservoir level drops to the base level at the control section. If the design discharge for the auxiliary spillway is of the same order of magnitude as for the service spillway, a single fuse plug may be used. In this case, if the inflow hydrograph discharge starts to decrease shortly after the fuse plug has breached, an excess discharge could occur, equal to the sum of the auxiliary channel discharge and service spillway discharge, such that the outflow exceeds the inflow. This can be avoided by temporarily reducing the service spillway discharge by partially closing the gates.

There are situations where the design discharge for the auxiliary spillway may be much greater than the capacity of the service spillway. In this case it may be desirable to use two or three fuse plugs (multi-stage), each set to breach at a different reservoir elevation. The fuse plugs will be aligned across the channel at the position of the control section, and will discharge into the same discharge channel, but they will be separated by a dividing wall extending vertically up from the sill, or by a heavily riprapped earth berm.

The first fuse plug to be breached will have a crest length which will allow a discharge comparable to the service spillway discharge. If the reservoir continues to rise even with both the service spillway and the auxiliary channel operating, the second fuse plug will breach, allowing a further surge in the discharge. A second stage would be set to breach at an elevation at least 0.3 m higher than the first stage.

The use of the multi-stage approach avoids the possibility of having a peak outflow much larger than the peak inflow, which could occur if one long fuse plug breached just before the inflow hydrograph discharge started to decrease. A three-stage fuse plug would probably represent an extreme case, but a two-stage fuse plug is not uncommon. One advantage of the multi-stage design is that the probability of breach of an additional stage is even more remote than the probability of breach of the first stage. Also, the post-flood maintenance program, requiring fuse plug replacement, is simpler when there is only a partial breach.

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PROBLEMS

1. The ogee crest for an overflow spillway is designed for a 9 m head. Calculate the unit discharge when the head is only 5.5 m.

2. A unit discharge of 14 $m^3/s/m$ enters a 45° free flip bucket with a velocity of 36.5 m/s. Calculate the required bucket radius and the theoretical horizontal throw of the jet. Assume 50% bulking due to the air entrainment.

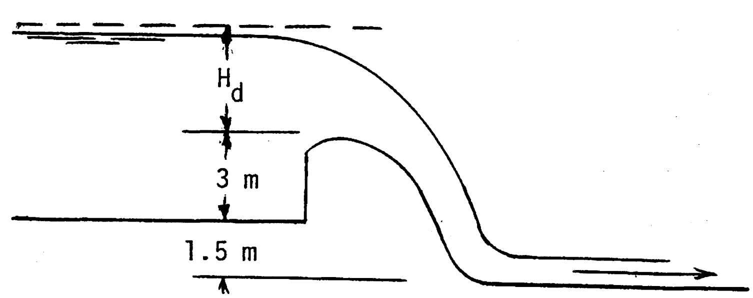
3. Explain the difference in the hydraulic action of a submerged roller bucket and a flip bucket.

4. Calculate the flow depth (neglecting air entrainment) at a level 70 m below the crest of the overflow section for a concrete gravity dam when the spillway head is 6 m, given that the crest shape was designed for a 5.5 m head.

5. What are the advantages and disadvantages of a controlled crest for a spillway?

6. A vertical face ogee weir 1.5 m high is operating at a design head of 3 m. Calculate the depth of flow immediately downstream from the weir, given no change in floor elevation.

7. Given q = 11.5 $m^3/s/m$, calculate the depth d_1 at the base of the ogee weir for the conditions illustrated.



8. Calculate the length and drop of a two-thirds gravity curve for a velocity of 18.3 m/s, going from a level slab to a chute sloping at 3 horizontal to 1 vertical.

9. Give 3 reasons why it is necessary to calculate velocities and depths down the chute of a spillway. In this calculation, what single item contributes most to possible errors in the result?

10. Calculate the head required on a 1 m high 45° sloping face ogee weir for a parallel sided chute spillway designed for 88 m^3/s. Assume a spillway width according to Equation 29.

11. Explain why the size of a hydraulic jump can be reduced by the addition of floor baffles in a stilling basin.

12. Explain, with the aid of a sketch, the function of an end sill on a stilling basin.

13. The flow entering a stilling basin is 0.6 m deep and has a velocity of 18.3 m/s. Using the momentum equation, compute the minimum tailwater depth required to keep a hydraulic jump in the basin if the force on the floor baffles is $\rho V_1^2 \, d_1/4$ newtons per metre of width.

14. Calculate the stilling basin details for a chute spillway designed for 280 m^3/s if the velocity at the bottom of the chute is 15 m/s.

15. Explain how uplift pressures may be produced by a hydraulic jump. How is this problem handled in the design of a spillway?

16 Establish the principal hydraulic dimensions for a chute spillway for1400 m^3/s, given the reservoir elevation at full supply as 95.0, the allowable reservoir rise during flood as 1.5 m, the elevation of the tailwater during flood as 78.0, and the head lost to friction in the chute as 3 m.

17. A rectangular box inlet spillway has L = W = 3 m and is to be operated under a maximum head of 1.8 m. Calculate the design discharge and the required depth of the box.

18. A drop inlet pipe spillway has a 1.5 m diameter pipe 90 m long, for which the friction factor $f = 0.02$ and the slope $S_o = 0.01$. The lip of the drop inlet has a radius of 0.2 D_o and is 6 m above the crown of the pipe at the inlet. Calculate the discharge through the structure when the reservoir level is above the lip by an amount (a) 0.6 m, and (b) 3 m.

19. Explain the occurrence of slug flow in a drop inlet pipe spillway.

20. If the maximum weir head before flood out on a drop inlet pipe spillway is expected to be 1.5 m would you recommend a lip radius of 0.3 m?

21. A 10.5 m high earth dam is to be constructed on a small stream to retain a 7.5 m deep reservoir at full supply level. The dam will have a 3:1 upstream slope, 4.5 m top width, and 2:1 downstream slope. The maximum allowable reservoir rise above FSL during flood is 1.5 m, at which head a design discharge of 14 m^3/s must be passed through a spillway. Calculate the hydraulic dimensions for a suitable drop inlet pipe spillway using the smallest possible diameter of cast-in-place concrete pipe with $f = 0.02$. Assume the conduit invert at the outlet is at the bed elevation of the channel.

22. Would you expect any air entrainment on a steep faced overflow dam 50 m high when operating with a 10 m head on the crest? Assume $C = 2.2$ for the ogee crest and that the slope of the spillway face is 0.7 horizontal to 1 vertical.

23. Calculate the bulked depth ($d_a + d_w$), due to air entrainment, on a long chute spillway with a 20% slope. Use a unit discharge of 2.5 m^2/s and assume terminal velocity is reached based on a Manning's $n = 0.012$.

24. A spillway chute 80 m wide has a 20% slope and a stilling basin with blocks at the end of the slope. A B-jump occurs when the structure is operating with v_1=22 m/s, d_1=0.8 m, and D_2=9.35 m. How far upstream from the end of the chute will the toe of the jump be located, and what will be the total potential uplift force on the chute and basin. (ANS. 17.8 m and 116,000 kN).

CHAPTER III

OUTLET WORKS

A. GENERAL

1. Purpose

An outlet works is a structure or combination of structures which provide for the controlled release of water from a reservoir. The outlet is invariably located at a lower elevation than the spillway crest, thereby permitting regulation of the reservoir even when the pool is below this level. Whereas the spillway is primarily a protective device, the outlet works is often the bread and butter structure for the project. In many cases the outlet works may even cost more than the spillway.

Outlets are built for irrigation, water supply, flood control, and power. A small outlet built to release water to meet the minimum downstream requirement, or riparian flow, is often called a riparian outlet. These are common on small storage dams scattered throughout the prairies. Larger outlets, called flood control outlets, may be used to draw down the reservoir in advance of a flood. These could also be used for emergency drawdown of the reservoir in the event of trouble with the embankment or spillway. The regulated release of water for irrigation purposes may be provided for by an irrigation outlet. While the riparian outlet or flood control outlet will discharge at the river level, an irrigation outlet will normally discharge at a higher level into an offtaking canal. A water passage terminating at the turbine of a power house is referred to as a power outlet. In some concrete dams a series of outlet tubes will be placed at different elevations to assist in the control of sedimentation in the reservoir. These are called sluices.

Where a conduit or tunnel is used with an earth dam, rock fill dam or arch dam, it is common practice to construct the conduit or tunnel first and use it for river diversion during construction of the dam proper.

2. Components

The components of an outlet works vary somewhat according to the purpose of the structure and type of dam. Generally, however, an outlet works must have an inlet, water passage, means of control, and an outlet structure. The water passage may be through, under, or around the dam.

The "through" type of water passage is restricted to concrete dams, and consists simply of formed openings left in the mass concrete. The "under" type consists of a cut and cover conduit as used with earth or rock fill dams. The "around" type consists of a tunnel driven around one end of the dam in the abutment. Tunnels are used for all types of dams, but particularly for thin arch dams, or large earth or rock fill dams. The three basic types of outlet works are illustrated in Figure 1.

In concrete gravity dams the inlet can be incorporated in the upstream face of the dam. Flow regulation is accomplished by gates operated from a gallery running over the tubes near the upstream face of the dam. There is no need for a special outlet structure, as the tubes are located to discharge at the spillway face of the overflow section. In this way the energy dissipating device used for the spillway will also serve for the outlet. This is quite permissible, since there is no need to operate the outlet when the spillway is discharging. Normally a separate outlet structure is required at the end of a conduit or tunnel to dissipate the excess energy of flow. The one exception to this is for the power outlet. In this case the energy is extracted by the turbine and the flow is discharged at low velocity from the draft tube.

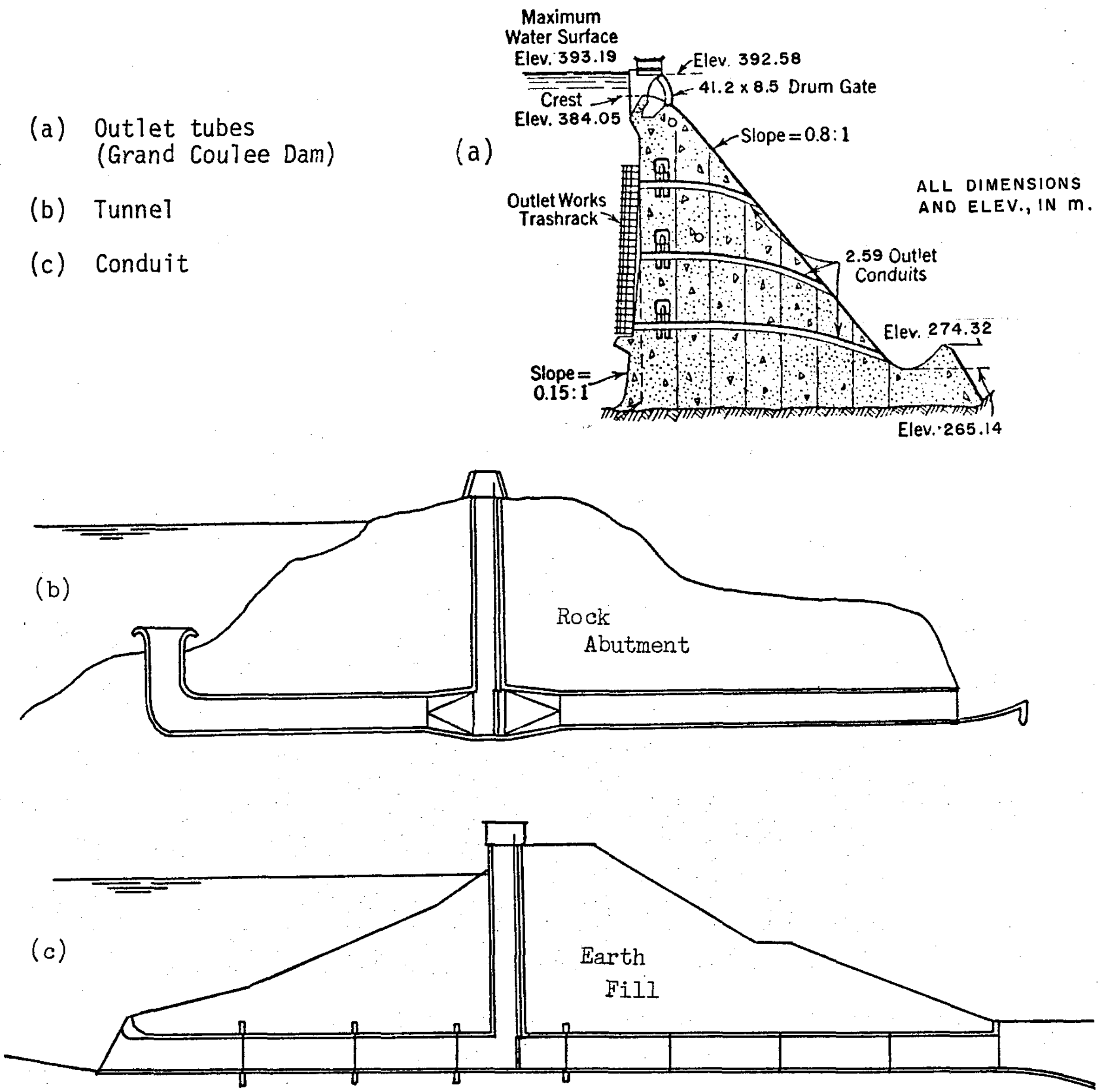


Figure 1. Types of Outlet Works

B. INLET STRUCTURE

3. Bellmouth Inlet

On small low head conduits, a simple earth retaining headwall with a square lip is often used at the inlet, for which it is customary to allow an inlet loss of 0.5 $V^2/2g$. This loss could be reduced to 0.05 $V^2/2g$ by using a streamlined bellmouth inlet. However, the corresponding increase in capacity may only be 5% or 10%, and this may not be enough to justify the complicated construction associated with the bellmouth inlet.

On high head outlets a streamlined inlet is a necessity if cavitation is to be avoided. Low pressures occur at the inlet if a square edged inlet is used because of the local high velocity in the region of the contracted jet. The jet will contract to 0.61 times the area of the inlet opening. The velocity head produced by the jet contraction will be greater than the velocity head in the conduit by a factor $(1/0.61)^2 = 2.7$. If the reservoir is above the centre-line of the pipe at the inlet by a height H, then the pressure head at the crown will be

[1] $$P/\gamma = H - D_o/2 - 2.7\ V^2/2g$$

in which D_o is the pipe diameter and V is the average conduit velocity. In most cases it will be found that the pressure obtained by [1] will be subatmospheric.

A properly designed bellmouth inlet will eliminate the jet contraction and thereby eliminate the negative pressure. The ideal bellmouth is shaped to fit the natural shape of a jet contracting from a sharp edged orifice, just as the ogee weir is made to fit the nappe from a sharp crested weir. To locate the origin for the bellmouth curve for a circular pipe, let D be the diameter of a sharp edged circular orifice which will produce a jet equal to the pipe diameter D_o. Then

[2] $$\pi D_o^2/4 = C_c\ \pi D^2/4$$

With $C_c = 0.61$, it will be found that

[3] $$D = 1.28\ D_o$$

In design, a value of 1.30 D_o is normally used, so the origin for the bellmouth is located 0.15 Do outside the pipe, as shown in Figure 2(a). Although the natural position of the vena contracta for a free jet occurs about 0.5 D_o downstream from the plane of the inlet, a shorter length, 0.25 Do, is satisfactory for the bellmouth curve. This shape can be closely approximated by a quarter ellipse with a half major axis of 0.25 D_o and a half minor axis of 0.15 D_o, as shown in Figure 2(a), and is frequently used for the inlet at the vertical upstream face of a concrete dam.

Frequently the inlet cross section will be square, even though the conduit may be round, because a bellmouth is easier to build for a square inlet. It is a simple matter to change the section with a square to round transition using a length of two diameters. If the invert of the inlet is flush with the bed of the approach channel, streamlining is confined to three sides. In this case a quarter ellipse with half major and half minor axis of 0.40 B_o and 0.25 B_o, respectively, may be used for each of the three sides.

In some cases the entire streamlining may be confined to the roof alone, as shown in Figure 2 (b). This could occur if there are several conduits side by side with only a thin dividing wall between, or if the sidewalls extend upstream. In this case, lengths for the half major and half minor axis should be 1.0 B_o and 0.6 B_o, corresponding to the jet shape produced by a sharp edged sluice gate. The 0.6 B_o value is indicated by the fact that $B_o/(B_o + 0.6\ B_o) = 0.625$, which compares favorably with the sluice gate contraction coefficient of 0.611. It will be conservative to allow 0.1 $V^2/2g$ for an inlet loss for any of these bellmouth designs.

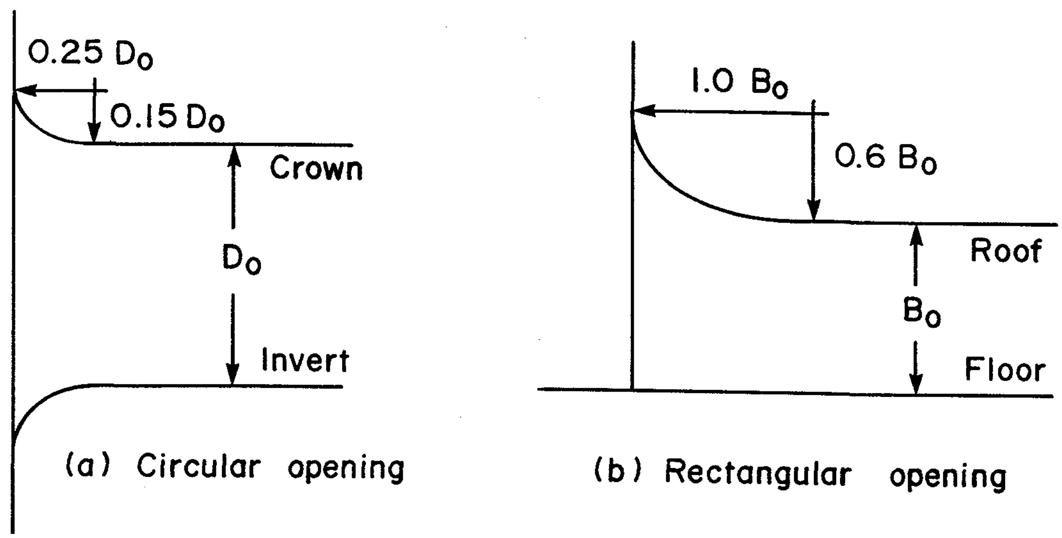


Figure 2. Bellmouth Inlet

C. WATER PASSAGE

4. <u>Tunnels versus Conduits</u>

Either tunnels or cut-and-cover conduits may be used for the water passage of an outlet works for an earth or rock fill dam. The tunnel is driven through virgin rock in one of the dam abutments, whereas the conduit may be placed almost anywhere under the fill. Usually the conduit is laid in a shallow excavation (cut) and backfilled up to the crest of the dam (cover).

The advantages of tunnels are:

(1) The tunnel located in an abutment will be in an area which will not be subjected to a large increase in load due to construction of the dam, hence there will be little settlement or movement of the tunnel, and a continuous unjointed lining can be used.

(2) The rock surrounding the tunnel usually can be depended upon to carry some of the load, so relatively thin linings may be used.

(3) Voids and fissures behind the tunnel lining can be pressure grouted from the inside. In this way a good watertight bond is secured and the risk of a piping failure is eliminated.

The service record of tunnels is excellent and they are invariably used on major dams. On smaller dams where a greater risk can be justified, conduits are often used to take advantage of their lower construction costs. Costs for tunnels can easily be twice as great as for conduits.

Cut-and-cover conduits may be subjected to considerable movement as the foundation consolidates under the weight of the dam. The barrel must be jointed transversely at frequent intervals to allow the conduit to follow the movement. The risk of a piping failure due to seepage along the outside of the barrel is much greater than for tunnels. Usually it is advisable to use collars around each joint at 10 to 15 m intervals along the conduit. These collars extend into the fill about 1 metre so as to intercept any seepage path which may develop along the barrel. The conduit wall thickness must be designed to carry the load due to the full height of overburden, and each joint must be waterstopped.

When considerable deformation of the foundation is expected, large joint openings initially may be left between each conduit section. The concrete collars around the perimeter of the pipe at each joint overlap the ends of the sections and are designed to allow opening, closing or rotation at each joint. Once the movements have stabilized, the openings can be permanently concreted. Usually the conduit can be used for diversion during construction, even with large joint openings, because under the low head conditions which occur for diversion flows, the velocities will be low. However, these openings should be concreted and waterstopped before the conduit is operated at full discharge with the reservoir at FSL. This procedure was followed at Alameda Dam in Saskatchewsan, where foundation settlements along the low level outlet at the centre of the dam reached 600 mm. Initially, 1 m joint spaces were left at 20 m intervals along the whole length of the conduit. The Alameda conduit was a 3.9 m high horseshoe shaped section, placed under a maximum fill height of 31.5 m (from the invert to the crest of the dam).

The advantage of conduits is the lower cost and relatively simple construction procedure. All components may be constructed entirely in the open using cast in place concrete. Pre-cast concrete or corrugated metal pipe has even been used on smaller dams for lower head outlets.

5. <u>Control</u>

Regulation of the gate for a conduit or tunnel is from a control house or hoist house located above the reservoir at the top of a vertical control tower, well, or shaft. A control tower is situated at the inlet end of a conduit or tunnel and an access bridge is required to get to the control house. This is not favoured in northern latitudes because of the difficult design problem associated with possible movement of the reservoir ice sheet which surrounds the tower. A control well is situated near mid-length of a conduit at approximately the centre of the fill, as shown in Figure 1(c). A control shaft is sunk to intersect a tunnel, again near mid-length of the tunnel, as shown in Figure 1(b). The mid-length location has the advantage that an access bridge is not required, and the structure does not have to be designed to resist ice load. However, the upstream half of the conduit or tunnel is inaccessible for regular inspection. In an emergency a bulkhead may be placed over the inlet by divers, and the length upstream from the gate may be drained.

The control gate is never located at the downstream end of the conduit, since the entire conduit would be pressurized at reservoir pressure when the gate is closed.

The discharge is regulated by vertically raising or lowering the gate. The gate may be a plain sliding gate, or operate on wheels or rollers. Except in small sizes, a rectangular gate is simpler and more economical than a circular gate, and therefore a round to rectangular transition is often used to connect the conduit or tunnel to the control.

When the gate is closed the water level in the control well or shaft will be at the same level as the reservoir. This level will drop as the gate is opened. In a sense the gatewell is like an oversized piezometer, the level in which is determined by the pressure head in the conduit at the base. It follows that the lowest level will occur when the losses between the reservoir and the gate are greatest, corresponding to maximum discharge. Observations on models have shown, as expected, that the level in the gatewell corresponds closely to the reservoir level minus the sum of the velocity head and the losses between the reservoir and the gatewell.

The head loss for flow through the gatewell with the gate wide open may be expressed as

[4] $$h_g = K_g\, V^2/2g$$

in which K_g is the gatewell loss coefficient. Experiments have shown that this loss coefficient depends upon the size of the well relative to the pipe, expressed as W/D_o, and the depth of water standing in the well, expressed as h_w/D_o. In these expressions W represents the plan view width dimension of a square in plan gatewell, and h_w represents the depth of water in the gatewell relative to the invert of the conduit. The magnitude of h_w may be determined from analysis of the losses in the upstream part of the conduit, as explained in the previous paragraph. When this depth is shallow, there is violent agitation of the water above the jet flow through the bottom of the well from the upstream pipe to the downstream pipe. As h_w increases the pool in the well becomes more quiescent and the loss coefficient decreases. Once h_w/D_o exceeds a value of 2 the loss becomes independent of h_w/D_o. The term W/D_o is a measure of the boundary sidewall discontinuity at the position of the gatewell. The smallest value for W/D_o is unity, and this gives the smoothest flow through the bottom of the well. A large value of W/D_o may be used to accommodate the gate frame and gate seats outside the diameter of the conduit. The lateral offset of the sidewalls of the well permits some diffusion of the jet flow as it passes through the well and the loss is increased. The effects of h_w and W on K_g are shown in Table 1.

TABLE 1
GATEWELL LOSS COEFFICIENT K_g

		Value of W/D_o		
		1.0	1.25	1.50
Value of h_w/D_o	1.2	0.10	0.14	0.16
	1.6	0.09	0.13	0.15
	> 2.0	0.08	0.12	0.14

Special problems and gate types for outlet works are discussed in more detail in Chapter 4, Gates and Valves.

6. Bends

Horizontal bends are usually unnecessary in conduits because a satisfactory layout can be obtained with a straight conduit.

Horizontal bends in tunnels may be necessary to secure good alignment of the approach to the inlet and discharge from the outlet. A total bend angle of 45° or less is usually sufficient to accommodate the alignment. These bends can be spread over most of the tunnel length and consequently may be very gradual. With the ratio of bend radius/diameter greater than 20 the bend loss may be neglected. It is usually desirable to have a length of straight tangent of 10 diameters or more preceding the outlet structure.

Vertical bends are sometimes required if the inlet must be placed at a higher elevation than the invert of the water passage. The bend is usually a full 90° and is used to connect the horizontal water passage to a vertical riser or shaft, which extends through the fill or abutment material to the reservoir above. This design is common for inlets where it is desirable to keep the inlet above the expected silt level. Unlike horizontal bends, vertical bends must be accommodated in a limited space and with a bend radius as short as possible. The permissible radius, however, is limited by the minimum allowable negative pressure. Low pressures will occur on the inside of the bend due to the production of higher than average velocities along this surface. The inner radius should be large enough so that the pressures will not drop below -2 m of water. This will insure that flow separation will not occur, the head loss will be small, and the risk of cavitation will be slight. A relationship between the inner radius R_i and the pressure head may be derived as follows:

Consider flow in a square conduit with a side dimension of B_o as shown in Figure 3. The unit discharge will be

[5] $$q = Q/B_o$$

For irrotational flow, the velocity v across the bend on a 45 degree line will vary inversely as the radius, as in the case of a free spiral vortex, giving

[6] $$v = K/R$$

in which K is a constant of proportionality and R is the radius to the point in question. The assumption of irrotational flow will be valid as long as the bend is within a few diameters of the inlet. This is usually the case for vertical bends.

From the continuity equation

$$dq = v\, dR$$

or

$$q = \int_{R_i}^{R_o} (K/R)\, dR$$

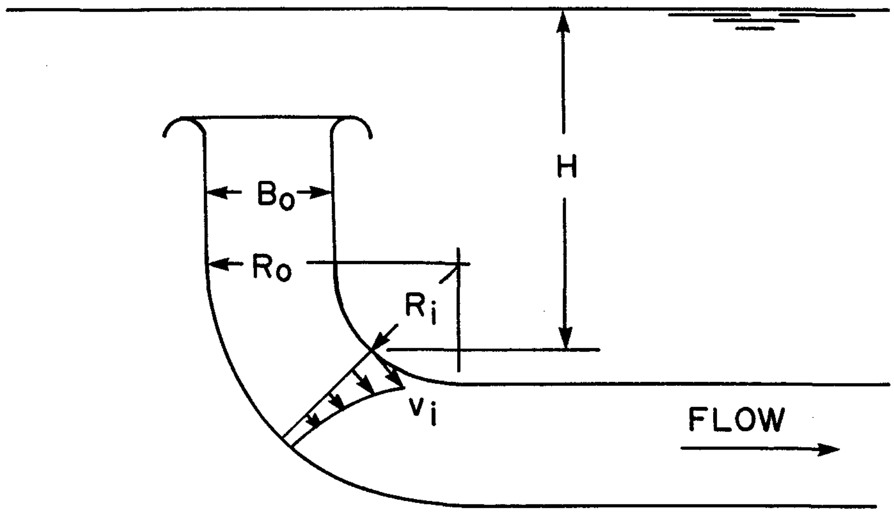


Figure 3. Flow Around a Vertical Bend

or

[7] $q = K \ln (R_o/R_i)$

Given q, R_i and R_o, K may be solved from [7]. The maximum velocity v_i, which occurs on the inside of the bend, may then be solved using [6] written as

[8] $v_i = K/R_i$

The pressure head at the inside of the bend may then be determined from a consideration of energy principles, as

[9] $P/\gamma = H - h_L - v_i^2/2g$

in which H represents the depth of submergence below the reservoir and h_L the inlet head loss. It will be conservative to allow 0.1 $V^2/2g$ for this loss for streamlined inlets.

If the pressure head given by [9] is lower than -2 m, consideration should be given to using a larger radius. In any case R_i should not be less than 0.5 B_o.

The foregoing method can also be applied to determine the minimum bend radius for a conduit or tunnel with a circular cross section. It is only necessary to change the definition for unit discharge. The discharge per unit width naturally varies across a circular section, but the critical area for bend analysis is the unit width on the vertical diameter. The flow through this strip, expressed in $m^3/s/m$, is

[10] $q = V D_o$

in which V is the mean velocity as given by Q/A. Hence

[11] $q = 4Q/(\pi D_o)$

This value for q must be used to solve for K in [7], after which [8] and [9] may be applied as before. This approach has been verified on a model of the vertical bend with circular cross-section for the power tunnels at Gardiner Dam in Saskatchewan.

The cross-section for cast-in-place cut-and-cover conduits is usually square to take advantage of simpler formwork. It is also easier to prepare good bedding conditions for a conduit which has a flat floor slab. If the conduit has a vertical riser at the inlet it is permissible to omit the outside curve on the vertical bend, as shown in Figure 4. The omission of the outside curve not only simplifies the problem of layout and construction, but the flat bottom also serves as a convenient base or footing for the riser.

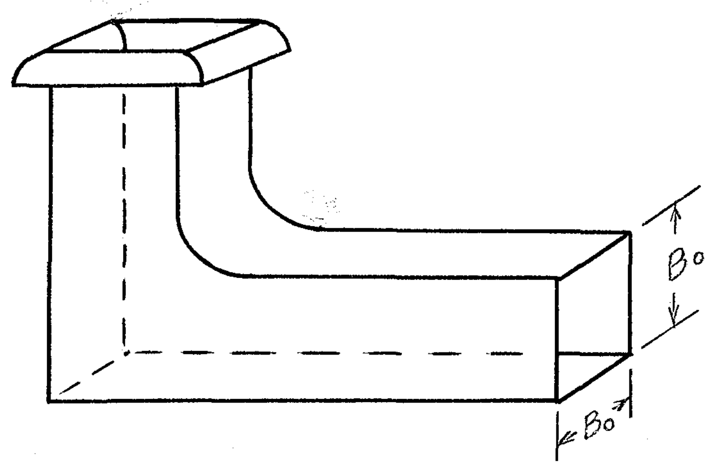


Figure 4. Bend Detail for a Square Conduit

Figure 5 shows the pressure distribution, observed on a model, around the inside of a 90° vertical bend with an inside radius $R_i = 0.5\ B_o$. It is seen that the pressure distribution is the same whether the outside curve of the bend is included or omitted. Further, the observed minimum pressure at piezometer 3 agrees with the theory previously developed. As the bend is symmetrical about piezometer 3 it might be surmised that the pressure distribution should also be symmetrical. However, the skewed plot is a reflection of gravity effects in that the pressure head at piezometer 5 must be greater than at piezometer 1 by the magnitude of the radius R_i.

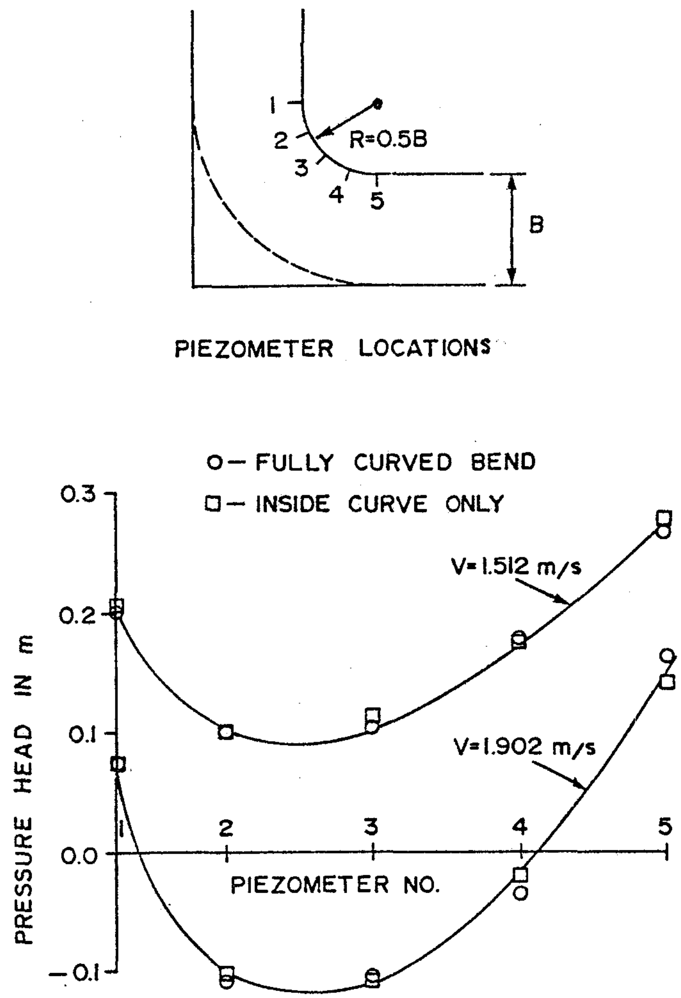


Figure 5. Pressure Distribution on Inside of Bend

In spite of the absence of separation the head losses at the bend are significant and must be considered. For the bend shown in Figure 5 a value of $K_b = 0.3$ was determined by Smith when the outside curve was included and $K_b = 0.5$ with the outside curve omitted. The slightly higher value for the square outside corner is due to the formation of a small separation zone and weak eddy in the corner. If a square corner is used on the inside of the bend as well, forming a 90° miter bend ($R_i = 0$), the loss coefficient increases dramatically, to about 1.3. This large loss is due to separation at the inside corner, followed by a severe contraction of the flow immediately downstream from the bend and the formation of a large strong eddy between the contracted jet and the roof of the conduit. The contraction is even more pronounced than at a sharp-edged projecting inlet. Observations from models show that the jet depth at the vena contracta is only 0.47 B_o.

If a larger radius is needed to avoid excessive negative pressure, the bend loss will decrease. This effect is shown in Table 2, after Miller. These values are slightly smaller than those determined by Smith, as discussed in the previous paragraph.

TABLE 2
BEND LOSS COEFFICIENT K_b

Smooth Conduits, 90° Bend, Large Reynolds No.

R_i/D_o or R_i/B_o	K_b	
	Round Conduit	Square Conduit
0.0	1.10	1.20
0.5	0.25	0.24
1.0	0.18	0.16
1.5	0.16	0.15
2.0	0.15	0.14

7. <u>Outlet Hydraulic Grade Line</u>

The effective position of the hydraulic grade line at the outlet end of the conduit influences the discharge capacity curve for the outlet works, and also has a bearing on the design of the outlet structure. If the outlet is submerged by tailwater, the hydraulic grade line (HGL) will be at the elevation of the water surface standing over the submerged jet. Generally, however, it is desirable to have free flow at the outlet. This is particularly true if a spreading floor transition is used with the outlet structure, because submergence interferes with the proper spreading of the jet.

For a free outlet it has been assumed by some designers that the HGL is at the crown of the pipe, and by some that it is at the centre of the pipe. In fact, it may be neither. If the pipe velocity is high so that the jet discharges with essentially parallel streamlines, then effectively the jet is at atmospheric pressure, and the effective position of the HGL may be taken at the centre of pipe. At lower velocities there is a pronounced convergence of streamlines at the outlet portal due to the downward inclination of the surface streamline. This produces internal pressure in the jet and is reflected in an increase in the elevation of the HGL. This elevation cannot be determined by a piezometer reading at the invert of the outlet because the pressure is not hydrostatically distributed; however, a line drawn through upstream piezometer readings may be extended to intersect the plane of the outlet, as shown in Figure 6, and the intersection at depth y above the invert of the pipe represents the effective position of the HGL.

The effective position of the HGL has been determined from models for both round and square pipe placed horizontally or with a mild slope, and flowing full at the outlet. The results are shown in Table 3, which gives the variation of y/B_o or y/D_o versus the Froude number F_o. The Froude number is calculated for the square pipe from

[12] $$F_o = V/\sqrt{gB_o}$$

and for the round pipe from

[13] $$F_o = V/\sqrt{gD_o}$$

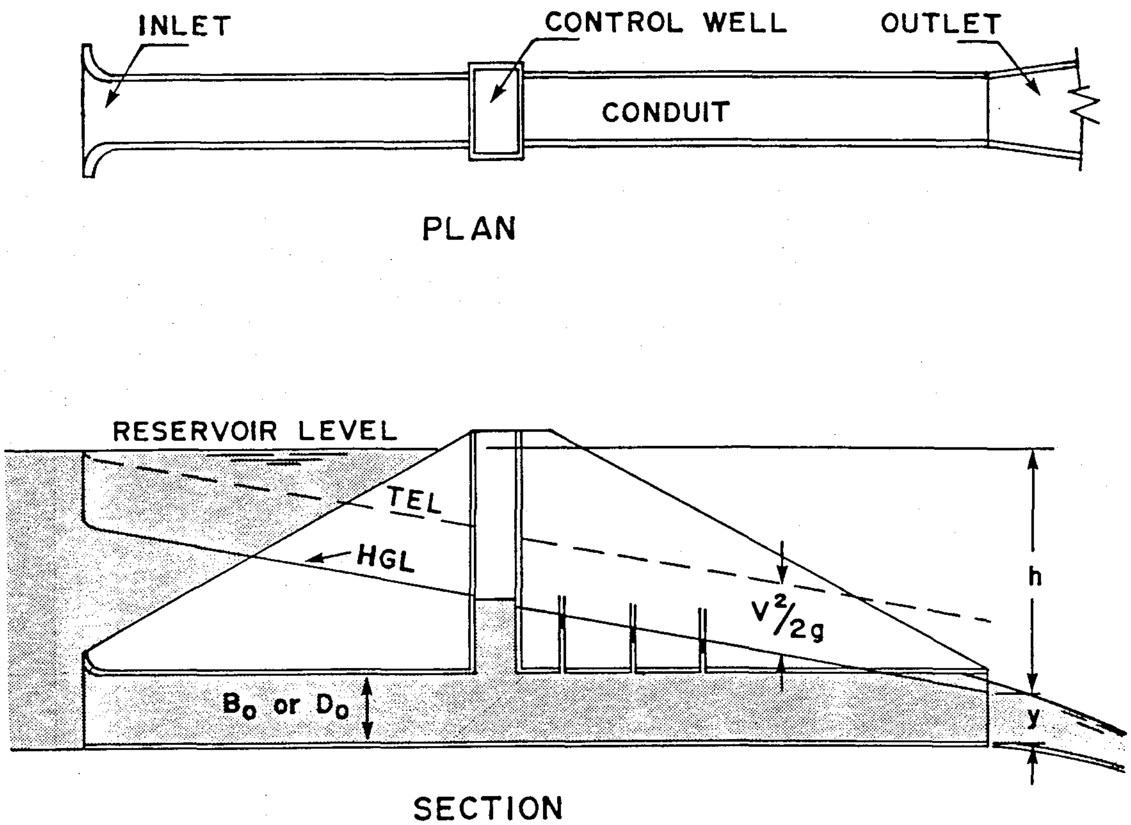


Figure 6. Position of Hydraulic Grade Line

TABLE 3
EFFECTIVE POSITION OF THE HYDRAULIC GRADE LINE

	F_o	1.0	1.2	1.4	1.6	1.8	20	2.2	2.4
(1)	y/D_o	0.75	0.70	0.60	0.53	0.51	0.50	0.50	0.50
(2)	Y/B_o	0.80	0.75	0.67	0.60	0.55	0.52	0.51	0.50
(3)	y/B_o	0.92	0.91	0.89	0.87	0.85	0.83	0.81	0.80

(1) Circular pipe, free or supported jet
(2) Square pipe, free unrestrained jet
(3) Square pipe, partial jet support (i.e. outlet structure)

It has been found that the value of y/D_o is little affected by the presence of a level floor or vertical sidewalls at the conduit outlet. Apparently, since there is only a line of contact between the jet and the boundary at the plane of the outlet, the jet is not effectively supported and additional pressure is not developed. However, for a square pipe, there is a significant difference in y/B_o for a free unrestrained jet and a jet supported at the outlet by the walls and floor of an outlet structure. This must be obvious from the fact that y/B_o would be unity for a fully supported jet. It is somewhat less than unity for the partial support obtained with diverging sidewalls.

8. Discharge Capacity

The full gate discharge capacity of an outlet works may be computed from a knowledge of the available head and head losses in a similar manner to the method described for the drop inlet pipe spillway. The available differential head h is utilized by producing velocity head and overcoming losses. The head h is measured from the reservoir surface to the tailwater level, if the outlet is submerged, or to the effective position of the hydraulic grade line at the outlet, if it is free.

Summing the losses

[14] $$h = h_e + h_b + h_g + h_f + V^2/2g$$

in which h_e is the inlet loss, h_b is the bend loss (if any), h_g is the junction loss at the control shaft, and h_f is the friction loss. The friction loss h_f may be determined from Darcy-Weisbach as

[15] $$h_f = f\,(L/D_o)\,V^2/2g$$

or from Manning as

[16] $$h_f = V^2 n^2 L/R^{4/3}$$

For a straight conduit and assuming $h_e = h_g = 0.1\ V^2/2g$, [14] may be reduced to

[17] $$h = (1.2 + fL/D)\,V^2/2g$$

or if the friction loss is calculated by Manning

[18] $$h = (1.2 + 19.6\,n^2 L/R^{4/3})\,V^2/2g$$

Example 1:

Calculate the upstream reservoir elevation required to discharge 22.5 m^3/s through a level 2 m diameter concrete conduit 130 m long. The conduit has a bellmouth inlet followed by a vertical bend with an inside radius of 2 m, a control well with W/D_o = 1.2, Manning's n = 0.012, and the invert elevation at the outlet is 100.0. The control well is 80 m upstream from the outlet.

Pipe flow velocity $V = Q/A = 22.5/0.785 \times 2^2 = 7.165$ m/s

Froude number $F_o = V/\sqrt{gD_o} = 7.165/\sqrt{g \cdot 2.0} = 1.618$

Value of y/D_o (Table 3) = 0.528

Friction loss $h_f = V^2n^2L/R^{4/3} = 7.165^2 \times 0.012^2 \times 130/0.5^{4/3} = 2.422$ m

From this it may be calculated that the water depth in the gatewell is 0.528 x 2 + (80/130)2.422 = 2.55 m, hence $h_w/D_o = 2.55/2 = 1.28$, and from Table 1 $K_g = 0.13$.

The entrance loss coefficient K_e may be taken as 0.05 and the bend loss coefficient K_b as 0.18 (from Table 2).

$$\text{Reservoir elevation} = 100.0 + y + V^2/2g + h_e + h_b + h_g + h_f$$

$$= 100.0 + (0.528 \times 2) + (1.0 + 0.05 + 0.18 + 0.13) \times 7.165^2/2g + 2.422$$

$$= 107.04 \text{ m}$$

D. OUTLET STRUCTURE

9. Spreading Transition

At the outlet end of the conduit the flow occurs as a highly concentrated, fast moving jet which has considerable potential for causing damage. There are several cases on record where such a jet with its associated eddies has caused erosion of the downstream toe of the dam, undermined the outlet, and formed a wide deep scour hole in the discharge channel. In order to prevent this, some form of structure is required to dissipate the energy of the jet before the flow is released to the downstream channel.

The most effective means of dissipating the excess energy is in a stilling basin designed for a hydraulic jump. However, it is desirable from both an economic and a hydraulic point of view to make the basin considerably wider than the width of the conduit, and therefore it is necessary to use a transition to spread the concentrated jet from the conduit into a relatively uniform shallow sheet preceding the jump.

A suitable design is shown in Figure 7. The rate of sidewall divergence is designated as R. The wall flare extends through the stilling basin so three widths may be defined: B_o is the width at the start of the transition; B_1 is the width at the end of the transition (or start of the stilling basin); and B_2 is the width at the end of the stilling basin. The width B_o is equal to the width of the conduit, which, in case of a circular conduit, is the diameter.

The transition length L_t, by simple geometry, is given by

[19] $$L_t = (B_1 - B_o)/2R$$

The total length from the outlet portal of the conduit to the start of the stilling basin is $L_t + B_o/2$, where the $B_o/2$ represents the tangent length for a short simple curve which is used to connect the parallel sides of the conduit to the flaring sidewalls of the transition.

Although in theory any value can be selected for the width B_1, economy dictates that it should be kept to a minimum. The reason for this is that L_t and B_1 increase together, and therefore a wider structure must also be longer, greatly increasing the cost. A satisfactory design will result if the width is calculated from

[20] $$B_1 = 1.1\sqrt{Q}$$

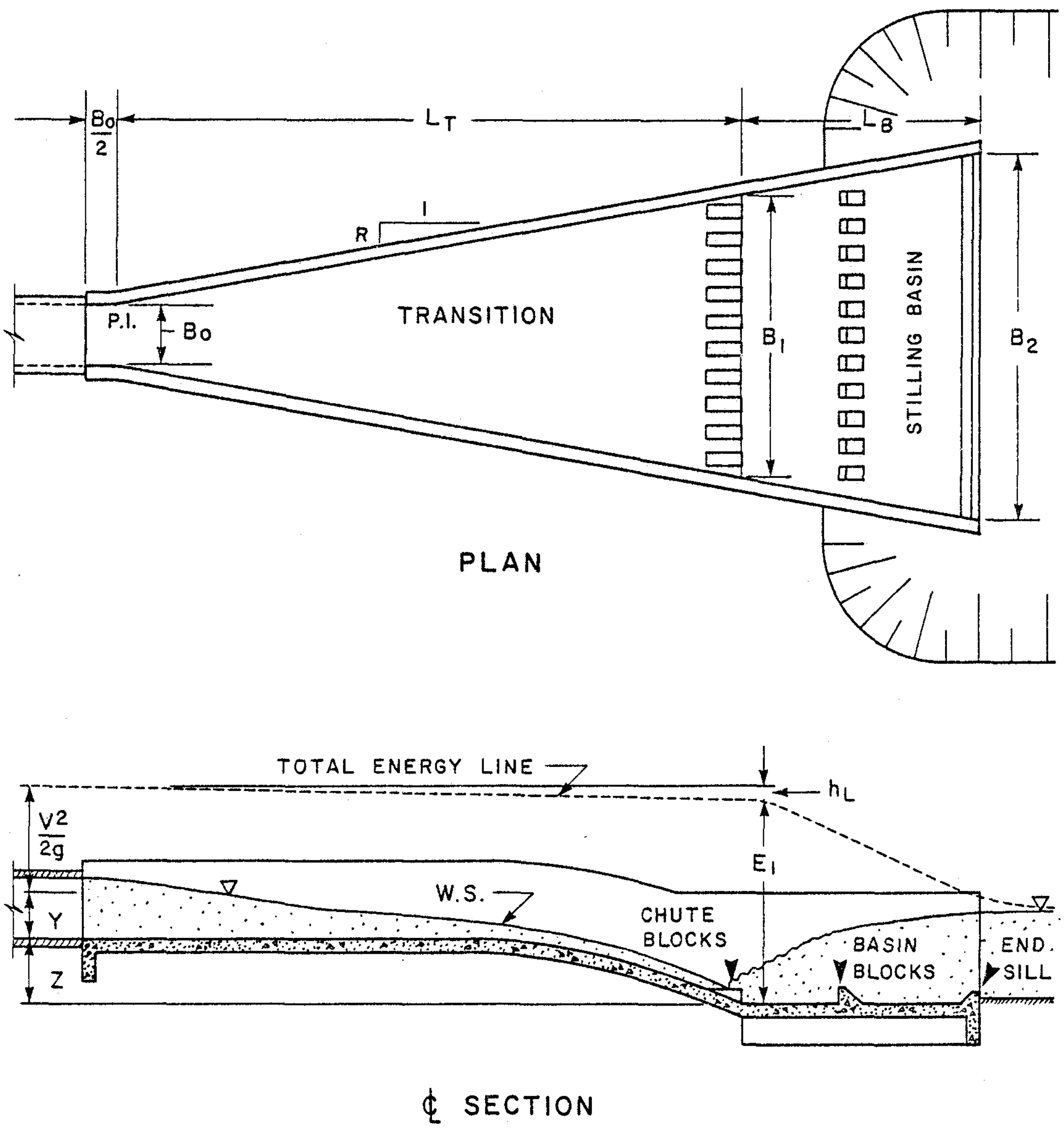


Figure 7. Definition Sketch for Outlet Structure

Rouse, Bhoota and Hsu have shown that expansion of supercritcal flow may be characterized by the equation

[21] $$z/B_o = [x/(B_oF_o)]^{3/2} + 1$$

in which B_o and F_o represent the initial width and Froude number for the flow, and z is the width between sidewalls at a distance x from the start of the expansion. Although the x and z coordinates define a curve, the equation can be adapated to a straight walled transition providing the rate of flare R is small (less than 0.2). Substituting $z = B_1$, $x = L_t$ (with L_t defined as in [19]), and introducing a coefficient C, then [21] can be written as

[22] $$R = C(B_1/B_o - 1)^{1/3}/2F_o$$

This equation is the same as [21] if C is unity. However, C must be considerably smaller than unity in order to get a sufficiently uniform depth distribution to produce a stable hydraulic jump.

Model tests by Smith have shown that for the straight walled transition and stilling basin the coefficient C shows some dependency on the Froude number. This function has been established as

[23] $$C = 2F_o/(4.5 + 2F_o)$$

and hence [22] can be written in terms of the permissible rate of sidewall flare in the form

[24] $$R = (B_1/B_o - 1)^{1/3}/(4.5 + 2F_o)$$

For a square conduit F_o is taken as $V/\sqrt{gB_o}$, and for a round conduit as $V/\sqrt{gD_o}$. This rate of flare will produce a reasonably uniform distribution of flow at the start of the jump, and is sufficiently gradual to permit continued expansion of the flow at this rate through the stilling basin as well.

10. Hydraulic Jump Depth

In the stilling basin the jump occurs in a diverging section, as illustrated in Figure 8. In applying the momentum equation, therefore, it is necessary to consider the gross section rather than a unit width, and the longitudinal component of the force exerted by the sidewalls on the flow must be included. Neglecting the forces due to the floor blocks, the force balance becomes

[25] $$P_2 - P_1 - 2F_wR/\sqrt{R^2 + 1} = Q\rho\,(v_1 - v_2)$$

or

[26] $$B_2\gamma d_2^2/2 - B_1\gamma d_1^2/2 - 2F_w\,R/\sqrt{R^2 + 1} = Q^2\rho(1/B_1d_1 - 1/B_2d_2) \quad (24b)$$

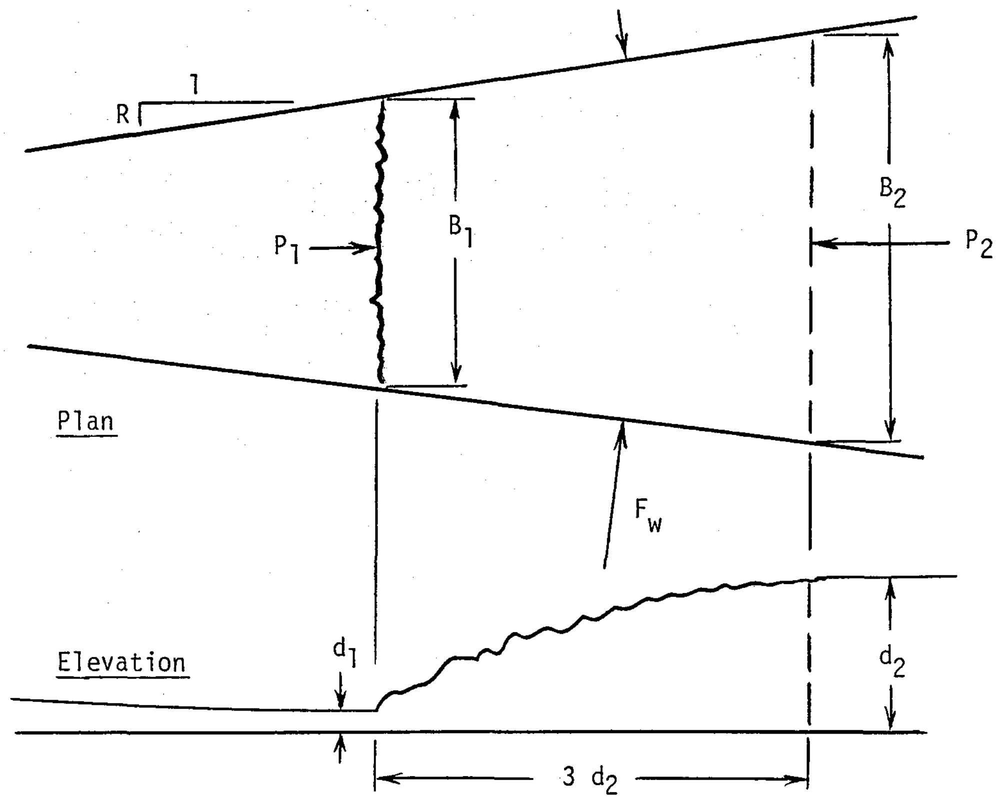


Figure 8. Definition Sketch for Diverging Jump

When R = 0, $B_1 = B_2$ and [26] reduces to the conventional jump equation for a parallel sided basin, viz

[27] $$d_2 = (\sqrt{8F_1^2 + 1} - 1)d_1/2$$

For a basin length of $3d_2$ the area of wetted sidewall adjacent to the jump may be taken as $3d_2(d_1 + d_2)/2$, and hence

[28] $$F_w = \gamma\bar{y}\, 3d_2\, (d_1 + d_2)/2$$

in which $\bar{y}$ is the depth to the centre of gravity of the trapezoid. The value $\bar{y}$ may be calculated from $\Sigma(Ay)/\Sigma A$ from which

[29] $$\bar{y} = \{d_1^2/2 + (d_2 - d_1)\,[d_1 + (d_2 - d_1)/3]/2\} \,/\, [d_1 + (d_2 - d_1)/2]$$

Finally, widths B_1 and B_2 are related by

[30] $$B_2 = B_1 + 2R(3d_2)$$

Given Q, B_1, d_1 and R, [26], [28], [29] and [30] can be reduced to a single equation containing d_2 as the only unknown. Needless to say the equation is cumbersome and the solution for d_2, which must be solved by trial, is laborious. Fortunately for the type of design being considered and the range of R values encountered, d_2 is very nearly 90% of the value given by [27] for the conventional jump, and hence

[31] $$d_2 = 0.45\,(\sqrt{8F_1^2 + 1} - 1)\,d_1$$

may be used for design. Although [31] is not exact for every case, it is quite satisfactory for design because the basin blocks will hold the jump in the basin with substantially reduced tailwater depth. In fact, the sweep-out depth is approximately 80% of the calculated value given in [31].

11. Stilling Basin Details

A stilling basin length of L = 3 d_2 should be used for the outlet structure. Chute blocks, basin blocks, and end sill are standard appurtenances. Blocks are particularly useful for this type of design because the flow distribution at the start of the jump may not be completely uniform. However, the force induced by a block depends upon the velocity approaching it, so a greater force is exerted where it is most needed. Specifications for the blocks are the same as for chute spillway basins (Section 27, Chapter 2).

In order to stabilize the hydraulic jump, it is essential that there by a definite slope on the floor of the transition at the entrance to the stilling basin. If the floor is flat and the tailwater excessive, the jump will be pushed upstream into the transition and may attempt to form at a position where the jet depth is not distributed uniformly enough to produce a stable jump. Basin flow may degenerate into a strong central jet with side eddies which circulate deep into the transition. A floor slope flatter than IV:4H should not be used. Steeper slopes are entirely satisfactory, although for practical reasons it is unlikely that IV:2H will be exceeded. A slope of IV:3H is a good compromise and should be used when suitable. In order to accommodate this slope into the basin, the pipe invert should be set high enough to permit a drop height of at least 1/2 D_o down to the stilling basin floor.

The jet should be fully supported throughout the transition in order to assist spreading and preclude the possibility of separation from the floor. A two-thirds gravity curve can be used to connect the level portion to the sloping portion of the transition.

The stilling basin floor must be set below the available tailwater level at design flow by an amount d_2, as calculated by [31]. This depth is dependent upon d_1 and v_1 at the start of the basin. These values in turn may be computed from the known unit discharge $q = Q/B_1$, and specific energy E_1. The specific energy at the start of the basin is given by

[32] $$E_1 = y + V^2/2g + z - h_L$$

in which y is the height of the effective position of the hydraulic grade line above the invert of the conduit, V is the conduit velocity, z is the drop in elevation from the conduit invert to the stilling basin floor, and h_L is the head loss in the transition.

The head loss in the transition is due to both boundary friction and shock wave loss. The flow occurs as rapidly varied non-uniform flow, for which no exact method of loss calculation exists. However, a small error in assessing this loss has very little effect on the computed jump height because d_2 varies approximately as the fourth root of the available specific energy. Further, a margin of safety is provided by the inclusion of floor blocks in the stilling basin. A satisfactory design can be made if the loss is taken as $h_L = 0.15\ V^2/2g$. The basin floor elevation and the value of z is dependent upon the jump height, which is dependent upon E_1, which is dependent upon z. Therefore, a trial value for z must be used for preliminary calculations. If it is found that due to high tailwater the final value for z is not enough to provide a drop into the basin of at least 1/2 D_o for jump stabilization, then either the conduit invert elevation at the outlet must be raised, or the floor of the transition must be raised. In the latter case the floor takes the form of a raised hump between the conduit outlet and the stilling basin.

A model of the straight walled spreading transition and stilling basin operating at design discharge is shown in Figure 9.

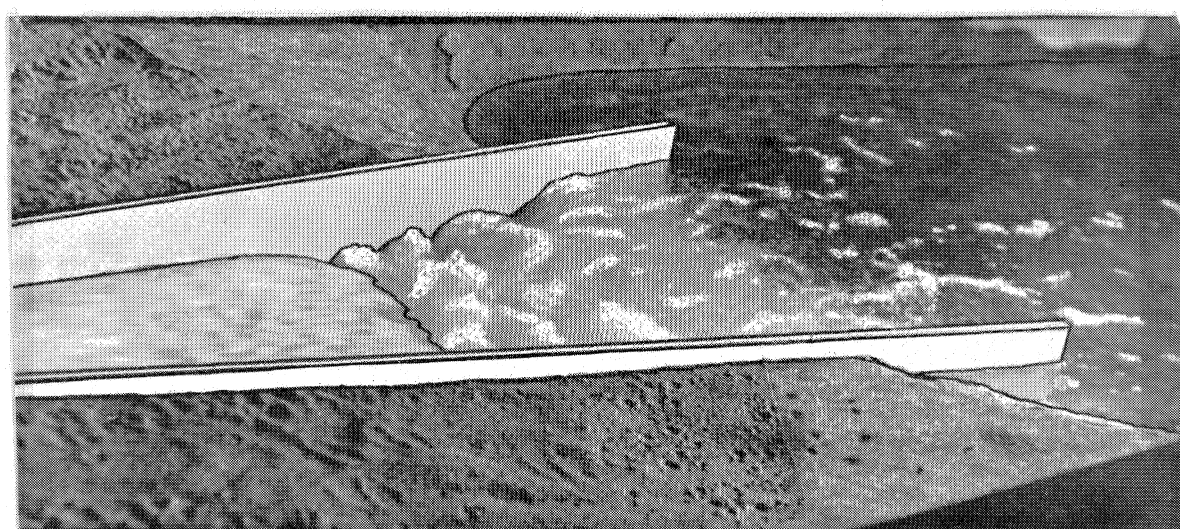


Figure 9. Model of Outlet Structure with Straight Diverging Walls

<u>Example 2:</u>

Establish the hydraulic dimensions for an outlet structure to receive 28 m^3/s from a 1.7 m square conduit. The drop in elevation from the conduit invert to the floor of the stilling basin will be 1.4 m.

Basin entry width $B_1 = 1.1\sqrt{Q} = 1.1\sqrt{28} = 5.8$ m

Conduit velocity $V = Q/A = 28/1.7^2 = 9.69$ m/s

Pipe Froude number $F_o = V/\sqrt{gB_o} = 9.69/\sqrt{g\ 1.7} = 2.37$

Value of $y/B_o = 0.80$ (from Table 3)

Specific energy at basin $E_1 \quad = z + y + V^2/2g - h_L$

$= 1.4 + (0.8 \times 1.7) + (0.85 \times 9.69^2/2g)$

$= 6.83$ m

Unit discharge at basin $q = Q/B_1 = 28/5.8 = 4.83\ m^3/s/m$

For given E_1 and q, $v_1 = 11.19$ m/s, $d_1 = 0.432$ m, and $F_1 = 5.44$

Design jump depth $d_2 = 0.45(\sqrt{8F_1^2 + 1} - 1)d_1 = 2.80$ m

Basin length $L_b = 3d_2 = 3 \times 2.8 = 8.40$ m

Rate of wall flare R $\quad = (B_1/B_o - 1)^{1/3}/(4.5 + 2F_o)$

$= (5.8/1.7 - 1)^{1/3}/(4.5 + 2 \times 2.37)$

$= 0.145$

Transition length $\quad = (B_1 - B_o)/2R + B_o/2$

$= (5.8 - 1.7)/2 \times 0.145 + 0.85$

$= 15$ m

Basin exit width $B_2 = B_1 + 2RL_b = 5.8 + 2 \times 0.145 \times 8.4 = 8.24$ m

Basin exit velocity $= Q/B_2d_2 = 28/(8.24 \times 2.80) = 1.21$ m/s

Other details to be established are wall heights, basin block dimensions, and coordinates for the 2/3 gravity curve.

Although not required for the design, it is of interest to check the force balance for the stilling basin using the calculated dimensions, and neglecting the floor blocks.

Upstream pressure force $P_1 = \gamma d_1^2\ B_o/2 = 9.81 \times 0.432^2 \times 5.8/2 = 5.31$ kN

Downstream pressure force $P_2 = \gamma d_2^2\ B_2/2 = 9.81 \times 2.8^2 \times 8.24/2 = 316.87$ kN

Wall area $(d_1 + d_2) L_b/2 = (0.432 + 2.80) \times 8.40/2 = 13.57 \, m^2$

Value of $\bar{y}$ (from [29]) = 0.95 m

Wall force $2F_w R/\sqrt{R^2 + 1} = 2 \times 9.81 \times 0.95 \times 13.57 \times 0.145\sqrt{0.1452 + 1} = 36.29$ kN

Upstream momentum flux $Q\rho V_1 = 28 \times 1 \times 11.19 = 313.32$ kN

Downstream momentum flux $Q\rho V_2 = 28 \times 1 \times 1.21 = 33.88$ kN

$\Sigma F = 316.87 - 5.31 - 36.29 = 275.75$ kN

$Q\rho(V_1 - V_2) = 313.32 - 33.88 = 279.44$ kN

Since very closely $\Sigma F = Q\rho(V_1 - V_2)$, the design, using $d_2 = 2.80$ m, is verified. The floor blocks will be capable of producing about 111 kN of force acting upstream, so an absolute minimum tailwater depth of approximately 2.3 m would still create a jump.

E. TWO STAGE STILLING BASIN

12. Purpose

The two stage stilling basin is a design sometimes used for high head outlets. The distinguishing feature of the design is that essentially two hydraulic jumps occur in series in the same structure. The tailwater depth $d_{2(1)}$ for the first basin is created by a fixed weir near mid-length of the basin. Tailwater for the second basin is due to the natural tailwater available in the downstream channel at the design discharge. The floor of the structure must be placed $d_{2(2)}$ below the natural tailwater elevation. Since most of the energy of flow is dissipated in the first jump, the required tailwater depth to produce a jump in the second basin is considerably decreased, with the result that the entire stilling basin floor may be placed at a higher elevation than for a single basin.

Although the two stage basin will be more than twice the length of a single basin, there will be certain cases where the benefits of a higher floor elevation will more than offset the cost of the extra length. For example, the deeper excavation for a single stage basin may involve a difficult dewatering problem or costly rock excavation. In clay or shale foundations problems associated with foundation rebound may develop, and the stability of the slopes adjacent to the stilling basin excavation may be critical. This may require slope flattening, greatly increasing the volume of excavation. These problems are minimized by reducing the depth of the cut. The depth of the excavation is reduced by $d_{2(1)} - d_{2(2)}$ if a two stage basin is used.

The advantage of the two stage basin increases as the entry Froude number at the first basin increases. There is little benefit to be gained unless the Froude number is greater than 8, hence the two stage basin is usually only considered for high head structures. The two stage basin used for the Oahe Dam tunnel outlets on the Missouri River permitted the basin floor to be raised 7 m above the level that would have been required for a single basin.

Although not discussed in Chapter 2, the two stage stilling basin may be adapted to spillway stilling basins as well. Such a basin was used for the Mangla Dam chute spillway in Pakistan, designed for 25,500 m^3/s and a 100 m drop. Usually a spillway width can be increased to reduce the jump height, if necessary, so the two stage basin is not as common for spillways as for outlet works.

13. Theory

The definition sketch for a two-stage basin is shown in Figure 10. Floor baffles are not used in the first basin. Generally baffles should not be used in any structure which is expected to produce high jet velocities a large percentage of the time. The baffles may be destroyed by cavitation and the safety of the structure jeopardized. This argument does not apply to spillways because the design discharge occurs with very low frequency. An outlet works, on the other hand, may be expected to operate at or near the design discharge almost every year. If it is a high head outlet which produces velocities in excess of 25 m/s, then it is inadvisable to depend on baffles for the proper performance on th structure. Baffles may be used in the second basin because the jet velocities will be reduced to less than half the velocity entering the first basin.

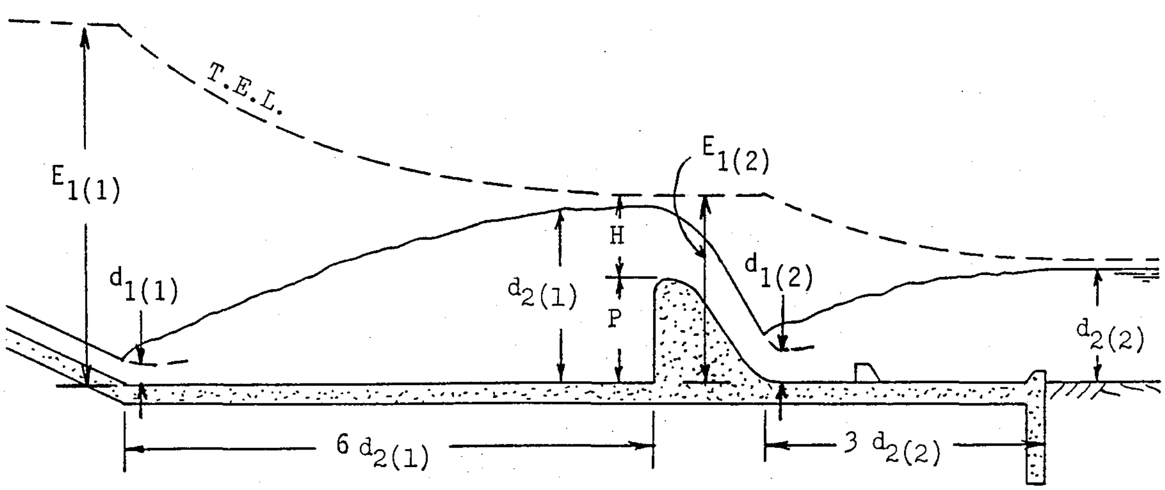


Figure 10. Centre-line Section of a Two-Stage Stilling Basin

For supercritical flow the ratio of jet depth to specific energy can be expressed in terms of the Froude number, by noting that $v_1^2/2g = d_1F_1^2/2$, and substituting this value of $v_1^2/2g$ in the specific energy equation $E_1 = d_1 + v_1^2/2g$, giving

[33] $d_1/E_1 = 1/(1 + F_1^2/2)$

Also the ratio of the specific energy at the end of a jump to the original specific energy E_2/E_1 may be expressed as

[34] $E_2/E_1 = [2(d_2/d_1) + (d_1^2F_1^2/d_2^2)]/(2 + F_1^2)$

The ratio of d_2/d_1 is given by the jump equation

[35] $d_2/d_1 = (\sqrt{8F_1^2 + 1} - 1)/2$

The head loss in the jump, $h_L = E_1 - E_2$, may be expressed non-dimensionally as

[36] $h_L/E_1 = 1 - E_2/E_1$

Equations [33] to [36] apply to any hydraulic jump, and therefore apply to both the first and second jumps when appropriate values are used for the input variables.

Neglecting head losses for flow over the weir, then the energy at the end of the first jump will be the same as the energy at the start of the second jump. Expressed symbolically

[37] $E_{2(1)} = E_{1(2)}$

From this it is possible to solve $d_{1(2)}$, $v_{1(2)}$, $F_{1(2)}$ and $d_{2(2)}$ for the second basin, and the ratio of the second jump height to first jump height $d_{2(2)}/d_{1(2)}$ may be calculated. Table 4 gives values for each of the ratios in [33] and [34] in terms of the Froude number.

For the first row in the table in which $F_1 = 1$, the flow is critical with the depth equal to two-thirds of the specific energy. There is no jump and no energy loss. The values h_L/E_1 show that for Froude numbers of 8 or greater the energy loss in the jump is large, and the last column in the table shows that the size of the second jump will be substantially reduced.

TABLE 4
EFFECT OF FROUDE NUMBER ON JUMP PARAMETERS

F_1	d_1/E_1	d_2/d_1	E_2/E_1	h_L/E_1	$d_{2(2)}/d_{2(1)}$
1	0.6667	1.00	1.00	0.000	1.000
4	0.1111	5.18	0.609	0.391	0.797
8	0.0303	10.82	0.336	0.664	0.688
10	0.0196	13.65	0.272	0.728	0.652
12	0.0137	16.48	0.228	0.772	0.625
14	0.0101	19.31	0.198	0.802	0.604
16	0.0078	22.13	0.174	0.826	0.588
18	0.0061	24.96	0.155	0.845	0.571

Since the two-stage basin will be about twice as long as a single basin, it will not be economical unless there is a large reduction in jump height in the second basin. This is why this design is not usually considered for Froude numbers below 8.

The weir height P must be selected to produce the required depth $d_{2(1)}$ for the first jump, in accordance with

[38] $$P + H = d_{2(1)} + v_{2(1)}^2/2g$$

In turn, the head will be related to the discharge through the weir equation

[39] $$H = (q/C)^{2/3}$$

As shown in Table 3 of Chapter 2, C is a function of H/P. However, for entry Froude numbers of 8 or greater the ratio of H/P will always be less than unity, and for an ogee weir designed with a pressure free profile C will fall in a narrow range. Design values are given in Table 5, taken from the U.S. Bureau of Reclamation Boulder Canyon Project Reports. As H and P are unknown initially, trial and error would normally be required to solve for C. This can be avoided by noting that there is also a unique relationship between H/P and the entry Froude number, which is known. Values for this relationship are also given in Table 5.

TABLE 5
WEIR COEFFICIENT AT DESIGN DISCHARGE

H/P	0.5	0.6	0.7	0.8	0.9
C	2.174	2.166	2.159	2.154	2.150
F_1	20.0	14.5	11.2	9.1	7.7

Coordinates for a pressure free profile for the vertical face weir for certain H/P values are given in Table 6. Coordinates for other values of H/P may be determined by interpolation. The origin for these coordinates is located H_s below the total head line, where H_s and H are related by the equation

[40] $$H_s = H + Y_{max}$$

and Y_{max} occurs at a value of $X/H_s = 0.25$ in Table 6.

14. Hydraulics

The hydraulic characteristics of a free jump or flow over a weir are well understood separately. However, if the jump is forced by a weir in close proximity, the performance of each is affected by the presence of the other. For example, flow near the end of a jump is expanding, whereas flow approaching a weir is contracting. If the jump is close to the weir, the flow contraction produced by the weir may occur before expansion of the jet under the jump is complete. In this case the velocity distribution, pressure distribution and energy content of the flow will all be different from normal values for the jump or weir alone. It may be expected that the design equations in Section 13 would require some modification to account for the difference in conditions.

TABLE 6
LOWER NAPPE COORDINATES
(From Boulder Canyon Project Reports)

Numbers in Table are Values of Y/Hs

	H/P			
X/H_s	0.44	0.59	0.74	0.89
0.00	0.0000	0.0000	0.0000	0.0000
0.05	0.0540	0.0525	0.0510	0.0490
0.10	0.0810	0.0785	0.0760	0.0735
0.15	0.0960	0.0930	0.0900	0.0868
0.20	0.1030	0.0998	0.0965	0.0922
0.25	0.1050	0.1012	0.0975	0.0928
0.30	0.1020	0.0980	0.0940	0.0892
0.35	0.0965	0.0918	0.0870	0.0820
0.40	0.0870	0.0820	0.0770	0.0720
0.45	0.0745	0.0697	0.0650	0.0595
0.50	0.060	0.055	0.050	0.044
0.60	0.022	0.017	0.012	0.006
0.70	-0.028	-0.033	-0.038	-0.044
0.80	-0.084	-0.090	-0.095	-0.102
0.90	-0.130	-0.156	-0.162	-0.168
1.00	-0.224	-0.230	-0.236	-0.242
1.20	-0.402	-0.407	-0.412	-0.417
1.40	-0.614	-0.618	-0.623	-0.628
1.60	-0.860	-0.864	-0.867	-0.870
1.80	-1.140	-1.144	-1.147	-1.152

It is evident that the position of the jump in the first basin will be dependent upon the weir height. If the weir is high the jump will form well upstream, and if the weir is low the jump will form very close to the weir. Conditions at the weir and in the second basin are improved if the first jump is completed well upstream, but this would require a long and expensive first basin. Therefore, the first basin should be as short as possible, while maintaining satisfactory performance in the remainder of the structure.

Model tests by Smith have shown that if the jump in the first basin forms 10 $d_{2(1)}$ upstream from the weir, the pressure distribution on the face of the weir and on top of the crest is the same as for reservoir conditions (i.e. no jump) upstream from the weir. With the jump at 8 $d_{2(1)}$ pressures on the face of the weir are about the same. At 6 $d_{2(1)}$ pressures on the face of the weir are slightly reduced, giving the first indication that the jump is affecting the weir and vice versa. The reduced pressure at the weir arises from the fact that the jump expansion is not complete, so the water depth at the weir is less than the theoretical $d_{2(1)}$. The nappe surface is rough but otherwise the weir flow appears normal.

At 5 $d_{2(1)}$ pressures on the weir face are locally increased, showing that the impact force of the jet is more than compensating for the decreased flow depth. The jet is deflected vertically upward at the weir face, and pressures over the top of the crest reduce significantly. The nappe becomes very rough, with sporadic separation from the weir. Flow enters the second basin in surges. At 4 $d_{2(1)}$ very high pressures are induced at the face of the weir, a boil forms over the weir crest, separation occurs frequently, and flow is unsteady.

It is evident from the foregoing description that the weir height must be designed to force the jump at least 6 $d_{2(1)}$upstream, and the first basin should be 6 $d_{2(1)}$ in length. A model of such a basin operating at design flow is shown in Figure 11.

15. Design

Observations on a hydraulic model have shown that if the weir height is selected according to [38] and [39], with the weir coefficient taken from Table 3, the height will not be sufficient to force the jump at 6 $d_{2(1)}$ upstream from the weir. It appears that since the jump is not fully completed before the flow reaches the position of the weir, the energy level is slightly greater than the theoretical energy $d_{2(1)} + v_{2(1)}^2/2g$. The weir crest must be slightly elevated to satisfy the head requirements and force the jump upstream at 6 $d_{2(1)}$. The calculated weir height should be increased by 3% to meet this requirement. Of course at any discharge less than the design discharge the weir height will be more than needed, and the first basin will be fully utilized. If the discharge exceeds the design discharge, the jump will move closer to the weir. In fact a 2% increase in discharge will shorten the jump to about 5 $d_{2(1)}$, and a 4% increase will shorten the jump to 4 $d_{2(1)}$. There is no need for this to occur on a gated outlet works because the discharge can be controlled. On a spillway there is always the chance that the design flood can be exceeded, however remote the possibility. This risk is always present regardless of the type of design used.

The second basin should be equipped with floor baffles and an end sill. Floor baffles are particularly beneficial when the entering flow is not completely steady, as they intercept the jet and reduce the wave action which would otherwise be produced by surge. For the two stage basin the floor blocks should be located 1.2 $d_{2(2)}$ downstream from the base of the weir, have a height of 1.5 $d_{2(1)}$, a width of 1.0 $d_{2(1)}$ and a spacing of 1.5 $d_{2(1)}$ between blocks. The end sill height may be set at 0.10 $d_{2(2)}$.

Figure 11. Model of Two-Stage Basin at Design Flow

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PROBLEMS

1. What is an outlet works, and what functions may it serve at a storage dam?

2. Why is it advisable to use collars on a cut-and-cover conduit?

3. Compute the pressure head on the inner radius at the mid-point of a 90° vertical bend near the inlet of an outlet works with a 3 m diameter riser and tunnel. The inner radius is 4.5 m, the discharge is 84 m^3/s, and the reservoir is above the invert of the tunnel by 2.2 times the tunnel velocity head. The inlet head loss is 0.3 m.

4. A flow of 90 m^3/s must pass around a 90° vertical bend in a 3 m diameter tunnel. The mid-point of the bend on the inner radius will be 10 m below the reservoir surface, and the head loss up to this point will be 0.05 $V^2/2g$. Would you recommend an $R_i = 3m$?

5. A reservoir outlet consists of a 1.2 m by 1.2 m square conduit which is 60 m long with an invert elevation of 0.0 m and a friction factor of 0.02. The outlet has a central control well and an abrupt square edged inlet. With the reservoir at elevation 12.0 m, by what percent could the discharge capacity be increased if a bellmouth inlet is used instead?

6. Why is it necessary to have a diverging transition preceding the stilling basin at the outlet end of an outlet works?

7. Determine the significant hydraulic dimensions for an outlet structure for a 1.3 m diameter pipe designed for a discharge of 8.5 m^3/s. The pipe invert will be located 1 m above the stilling basin floor.

8. Given the natural tailwater elevation = 100.0 m, v_1 = 30 m/s and d_1 = 0.3 m as the basin entry conditions for a high head outlet, calculate the required weir height P for a two stage basin. How much lower would the floor have to be placed if a single basin is used instead?

9. Why is it necessary to construct the weir at the end of the first basin of a two-stage stilling basin slightly higher than suggested by theory?

CHAPTER IV

GATES AND VALVES

A. INTRODUCTION

1. Spillway Gates

Spillway gates are used to control the spillway discharge when the crest of the spillway is situated below the normal operating level of the reservoir. Most major spillways are gated because the extra head obtainable with a lower crest will allow passage of the flood with a narrower and more economical structure. Considerable manipulation of the outflow hydrograph is possible with a gated spillway for all except the largest floods. At the spillway design discharge, the gates will normally be wide open and the outflow hydrograph uncontrolled.

Spillway gates are mounted between vertical piers on the spillway crest, and therefore the gated opening is always square or rectangular. For some gate types there are slots or recesses in the piers into which the gate projects. The entire water load on the gates must be transmitted to the piers, and this is a very important load to consider in pier design.

It is not practical or economical to design the gates and piers to resist ice thrust, so some means of preventing ice buildup at the gates is required. A compressed-air bubbler system is most often used. The system consists of a submerged horizontal air pipe running parallel to the upstream face of the gates, and an air supply from a compressor located at one abutment. The air pipe is imbedded in the spillway crest just upstream from the gate seat, and is equipped with nozzles spaced at approximately 1.5 m. Compressor pressures are quite low. Air is pumped into the pipe as required, usually for several hours a day in the coldest weather. The curtain of rising air circulates the warmer water from the lower depths and thereby maintains an ice free gap in front of the spillway. In addition, electric heaters may be required to insure that gate seals, seats, wheels, or other operating components do not become ice bound due to leakage through the gate. A portable steam hose backup may also be available on site for emergency situations.

Sliding, vertical-lift, radial, drum, rolling, and ring gates are considered in more detail in Part B of this chapter.

2. Gates for Outlet Works

Regardless of the purpose, outlets are located below the normal reservoir operating surface, so naturally all outlets must be gated. Sliding, vertical-lift, and radial gates may also be used for outlet works. Circular gates can be used, particularly in smaller sizes, but rectangular gates are preferred for large outlets. Whereas the width of rectangular spillway gates may be several times the height, the height of rectangular outlet gates usually exceeds the width. Narrow gates are necessary in order to reduce the size of the control shaft in which the gate operates. When rectangular gates are used with a circular conduit, transition sections must be used upstream and downstream from the gated section.

Design problems on outlet gates are considerably more difficult than for spillway gates. These are discussed in detail in Part C of this chapter.

3. Valves for Outlets

At heads greater than 25 m, difficulties are often encountered when operating outlet gates at partial opening, particularly for very small openings. Some gates must be operated either completely open or completedly closed. This may not be a problem where there are a large number of outlets because the discharge from a single gate wide open may still be a small percentage of the total capacity of the outlet works. In many cases, however, there may be only one outlet, and closer regulation of the discharge may be required than can be obtained within the operating limitations of an outlet gate. In case of an arch dam, for example, it may not be possible to use an outlet gate at all because of space limitations. A valve may be used to advantage in these situations.

Outlet valves are normally installed at the downstream end of the conduit, and usually discharge to atmosphere. Since the conduit upstream is pressurized at full reservoir head when the valve is closed, it is necessary to use a steel lining inside concrete conduits. The lining must extend for a considerable length preceding the valve, at least to a position upstream from the centre-line of the dam, and be designed to resist the full internal water pressure. In some cases, a smaller diameter free standing penstock may be constructed inside the concrete conduit.

Outlet valves are used for flow regulation for flood control, irrigation, and water supply. The needle, tube, hollow jet, and Howell-Bunger valve are considered in Part D of this chapter.

B. SPILLWAY GATES

4. Plain Sliding Gate

The plain siding gate is the simplest of all gate types. In the smaller sizes, it may be completely cast in steel, consisting of a flat plate with vertical and horizontal ribs on the back to stiffen the gate. In the larger sizes, instead of cast ribs, a built up structural steel framework is used to carry the load, and this is covered with a steel skin-plate for water tightness.

A babbitt metal seat may be used for the water seal at the bottom of the gate. This is a soft copper tin alloy which takes on the exact shape of the gate lip when the gate is closed, insuring water tightness. The babbitt metal is poured in a recess in the spillway crest. The plain sliding gate has a continuous bearing in compression along both sides, so it gives the simplest and most effective water seal of any gate type. The lifting force F is given by

[1] $$F = W + N \tan \phi$$

in which W is the weight of the gate, $N = \gamma h A$ = the static water load perpendicular to the gate, and $\tan \phi = \mu$ = the coefficient of starting friction. The hoist capacity should be designed for a lifting force at least 50% greater than given by [1]. This allowance, or safety factor, is necessary to allow for possible misalignment, deterioration of the seating surfaces, or difficulty with debris.

Design values for the friction coefficient for well finished clean surfaces are 0.6 for steel on steel, and 0.45 for bronze on bronze or bronze on steel. Bronze running surfaces are more expensive, but the hoist capacity is reduced. Bronze is less affected by water, and more dependable.

Because of the large friction force, the sliding gate must be forced closed as well as open. The value of N tan ϕ is almost always greater than W, even for bronze seating surfaces, so a hoist with positive action for both opening and closing is required. The smaller gates have a screw type hoist with a steel threaded rod, called the gate stem. The lower end of the stem is attached to the top of the gate and the upper end passes through the hoist pedestal. The stem must have adequate guides to prevent buckling when the gate is being closed.

Larger sliding gates may use a rack-and-pinion type hoist to increase the mechanical advantage. The rack may be a wide flange section with gear teeth in one flange, and is the gate stem. The lifting force may be geared down by a succession of spur gears so that a mechanical advantage as great as 100 may be obtained at the crank. The gates may be power operated using a hydraulic cylinder. Slide gates are slow operating. The rate of gate travel may be as little as 5 or 10 cm per minute.

5. Self Closing Vertical-Lift Gates

These gates travel vertically on wheels or rollers and when released will close under their own weight, hence the designation self closing. The construction of the body of the gate is similar to the slide gate, consisting of a skin plate on a structural steel frame. The principal load-carrying members in the frame are horizontal.

A flexible steel cable or link chain may be used for opening the gate. The cable is reeled in on a cylindrical drum. The drum may be power operated, or in the smaller sizes, manually operated by a crank at the end. These gates have a relatively short operating time as hoist speeds may be up to 1 m/min.

Sealing is the most difficult design problem for this type of gate. There are no gate seats along the sides as in the case of the slide gate, as the sides of gate body are not in direct contact with the gate frame. It is necessary to use flexible seals, consisting of flat strips of rubber belting or hard rubber J-seals. These are bolted to the sides of the gate and held in contact with the water seal surface on the piers by the water pressure. Usually a flat steel plate, called a rubbing plate, is used for the seating surface for the seals on the pier sides. The seal friction is high, as the coefficient of friction for rubber on steel is 1.1; however, the surface area of the gate seal exposed to water pressure is usually small. There is often a small leakage at the gate corners where it is difficult to get a continuous water seal.

One of the first self closing gates was the Stoney gate, named after the inventor. This gate travels on hard steel rollers which are independent of both the gate and the track. As the roller travel is only half the gate travel, the required vertical extent of the rollers is twice the vertical height of the gate. The roller position is maintained by a light steel framework. Rolling resistance is only about 1% of the normal load, and hence there is a substantial reduction in operating load as compared to the slide gate. Today the Stoney gate is not used, as some trouble has been experienced with the independent rollers.

The caterpillar or roller train gate, sometimes called the Broome gate, is a more satisfactory type of gate. Each side of the gate is equipped with a continuous chain of caterpillar rollers which run on a vertical seat in the slot in the piers. The caterpillar rollers are attached to the gate. The hoist capacity for the Stoney and caterpillar gate is usually designed for the gate weight + 5% of the normal load + side seal friction + safety factor, as in

$$HC = 1.5\ (W + \mu_s N_s + 0.05\ N) \quad [2]$$

in which μ_s is the coefficient of side seal friction and N_s is the water load pressing the side seals onto the rubbing plate.

Fixed-wheel gates are similar to the Stoney gate except that they move on wheels attached to the gate. Fixed wheel gates are one of the most common types of spillway gates in use. The wheels move on a track in the pier recess. On high head gates the spacing of the load carrying horizontal members naturally is smaller near the bottom of the gate, and likewise, the wheel spacings must be reduced near the bottom. The permissible wheel spacing limits the size of these gates. A rule of thumb for design is that the span times the head at the bottom of the gate must be less than 190.

The hoist capacity for a fixed wheel gate may be calculated from

$$HC = 1.5\ [W + \mu_s N_s + N(\mu_a r/R + \mu_r/R)] \quad [3]$$

in which R is the wheel radius, r is the axle radius for the wheel, μ_a is the starting coefficient of axle friction (about 0.3 for plain axles, 0.01 for ball bearings) and μ_r is the starting coefficient of rolling resistance (about 0.001 m for a steel wheel on a steel rail). Hoist speeds of 0.6 to 0.9 m/min can be obtained. A cross section of a fixed wheel gate is shown in Figure 1, and details of typical track and side seals are shown in Figure 2. The fixed wheel gates used for Squaw Rapids spillway in Saskatchewan are shown in Figure 3. These gates are 9.45 m high and 12.2 m wide.

<u>Example 1:</u>

Calculate the power required to raise at 0.6 m/min a 12 m by 10 m high fixed wheel gate. The gate has 0.3 m wheels with 5 cm axles, 5 cm side seals, and a mass of 50000 kg.

Assuming the reservoir is at the top of the gate, the side seal friction will be 2 x 1.1 (9.81 x 10/2 x 0.05 x 10) = 53.96 kN.

The normal water force on the gate is

$$N = 9.81 \times 10/2 \times 12 \times 10 = 5886 \text{ kN}$$

and the lifting force

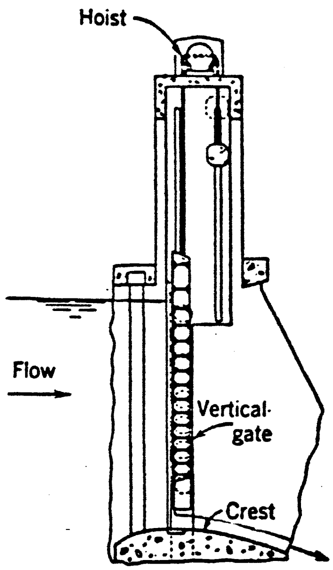

Figure 1. Fixed Wheel Gate

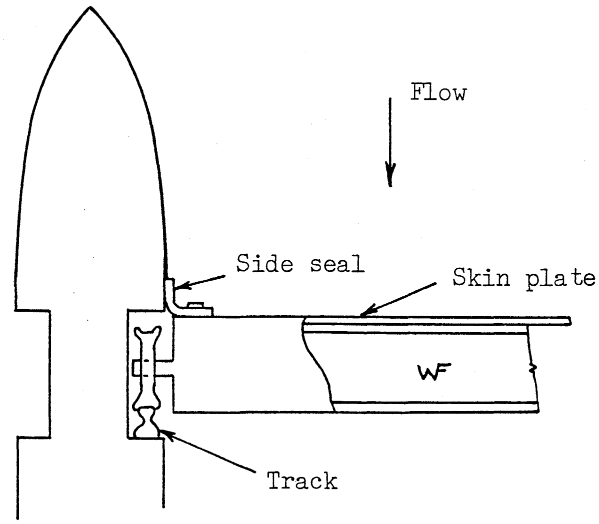

Figure 2. Gate Track and Side Seal

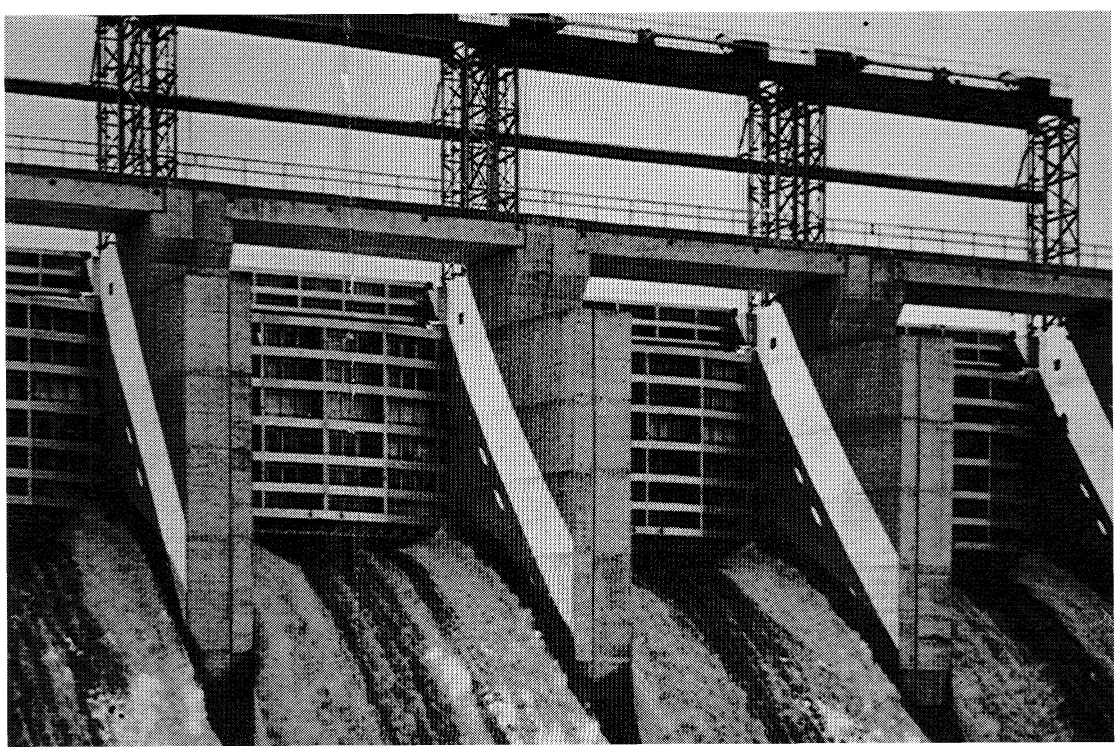

Figure 3. Fixed Wheel Gates, Squaw Rapids Spillway, Saskatchewan

$$
\begin{aligned}
F &= (9.81 \times 50000) + 54 + 5886(0.3 \times 0.05/0.3 + 0.001/0.3) \\
&= 490.5 + 54 + 313.9 \\
&= 858.4 \text{ kN}
\end{aligned}
$$

The power becomes 858.4 x 0.6/60 = 8.58 kW

This should be slightly conservative, because the actual running axle friction and rolling resistance will be less than 313.9 kN. However, the design hoist power should be set at approximately 13 kW in order to provide a safety factor in operation.

6. Radial Gate

The radial gate is the other most common type of spillway gate in use. The gate consists of a skin plate which is a segment of a circle, reinforced as necessary and supported by a structural steel frame. The arms of the gate are connected to a trunnion which is at the centre of curvature of the skin plate. A typical cross-section of the gate is shown in Figure 4. Since water pressure acts normal to a submerged surface, the resultant water force always acts through the trunnion pin. This is true for any gate position, open or closed. As pin friction is both small and at a mechanical disadvantage, the weight of the gate plus side seal friction is the principal force resisting opening. The radial gates for Gardiner Dam spillway in Saskatchewan are shown in Figure 5. These gates are 8.8 m high and 12.1 m wide.

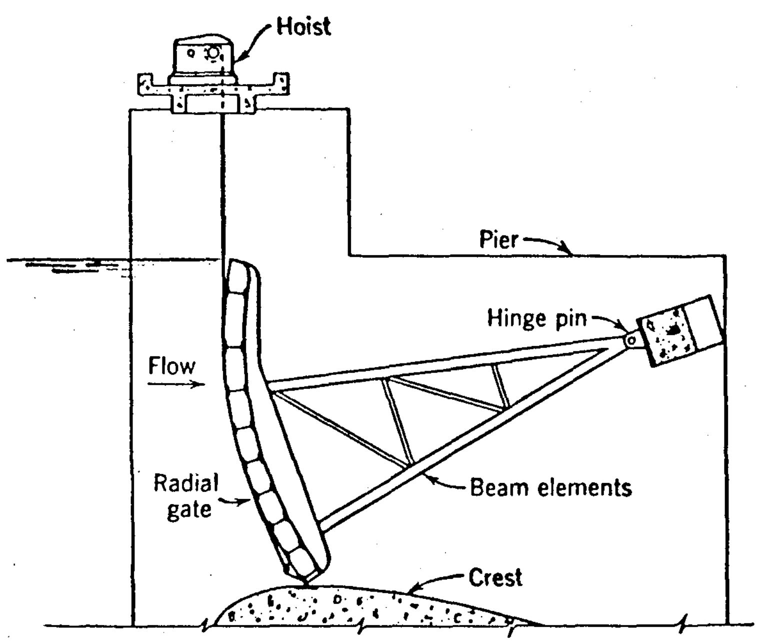

Figure 4. Spillway Radial Gate (Closed)

The bottom seal for a radial gate may be by a direct contact between the gate lip and a babbitt metal seat, or a hard rubber J-seal may be bolted to the lip at the front of the gate. Compression of the seal by the gate weight is sufficient to prevent leakage. Side

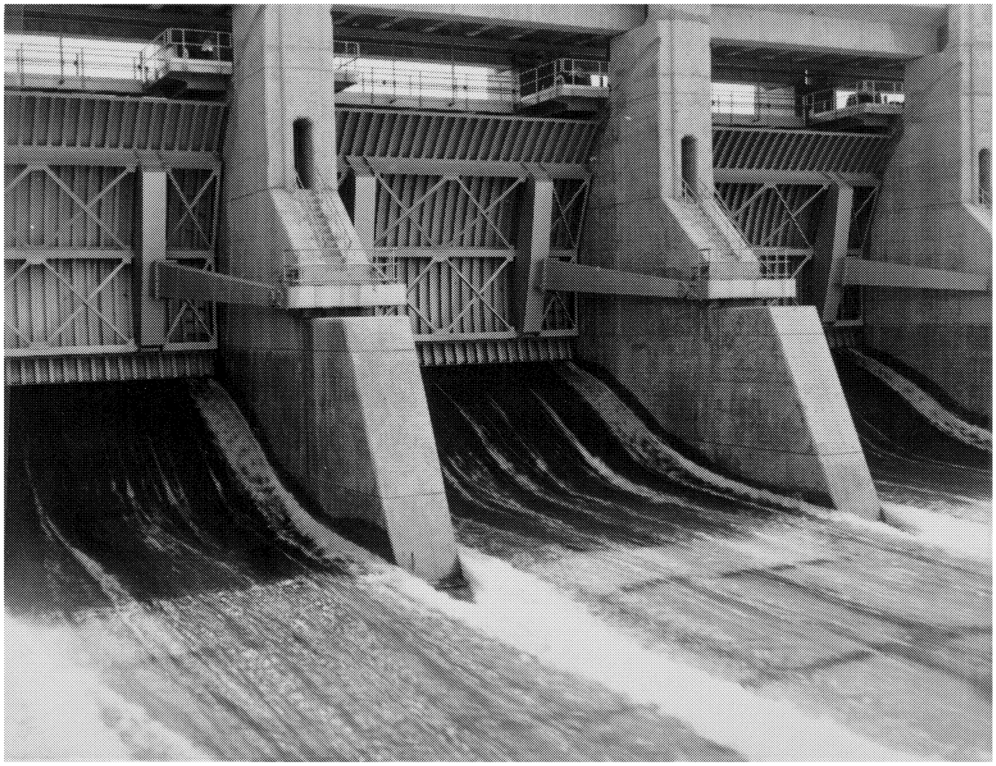

Figure 5. Radial Gates, Gardiner Dam Spillway, Saskatchewan

seals may be rubber belting or J-seals. These are attached to the skin plate and pressed by water pressure onto the steel rubbing plates which are flush with the sides of the pier.

The radial gate does not require any slots or recesses in the piers. This is a distinct advantage for operation at higher heads where slots would greatly disturb the high velocity flow shooting under the gate. On wheeled gates cavitation produced by gate slots has in some cases caused damage to the piers downstream from the slots.

In order to permit inspection or repair of a gate without lowering the reservoir, provision is usually made to shut off the spillway bay by placing stoplogs or a bulk-head in pier slots located upstream from the gate. However, these slots, being in a low velocity area, do not significantly disrupt the smooth passage of the flow.

The hoist capacity, including 50% factor of safety, may be calculated by

[4] $$HC = 1.5\,(W\,R_c/R + \mu_p Pr/R + \mu_s N_s)$$

in which W is the total weight of the gate and frame, R_c is the radius to the centre of gravity of the gate and gate frame, R is the radius to the face of the skin plate, μ_p is the coefficient of friction for the pin (0.3 for plain pins, 0.01 for ball bearings), r is the radius of the pins, P is the resultant force through the pins, μ_s is the coefficient of friction for the side seals (1.1 for rubber), and N_s is the normal force on the side seals.

Since W acts quite near the face of the gate the term W R_c/R due to gate weight is significantly greater than the sum of the two remaining terms due to pin friction and side seal friction. Accordingly the gate is self closing when the hoist cables are unwound. Two cables are used, one at each end, to prevent the gate from twisting and possibly jamming in a partially open position. The cables are connected to the bottom of the gate. This makes it possible to raise the top of the gate well above the level of the hoist itself.

The radial gate may be adapted to automatic float controlled operation. In this case the term W R_c/R in [4] must be largely balanced off by a counterweight. The counterweight, usually concrete, may be hung from the lift cables which run over pulleys, or it may be placed downstream from the hinge on an extension of the arms connecting the gate face to the trunnion.

Due to ease of operation, absence of gate slots, small hoist capacity in relation to size of gate, and relatively low cost, the radial gate is common for chute spillways. The principal disadvantage is that special pin anchorage is necessary, since the entire water load on the gate is transmitted to the trunnion pins. In the slide or wheeled gates the water load is distributed over the full height of the gate at each side, near the front of the piers. In the radial gate there is a large concentrated load at the pins near the back of the piers. This load must be transmitted into the whole pier by special pier reinforcement. For example, on a 15 m by 8 m high radial gate the pin loading would be 2.35 MN. A 0.35 m long pin would have to be almost 0.3 m in diameter to keep bearing stresses within allowable limits (24 MPa for bronze on bronze).

The discharge capacity of a radial gate is a function of the gate opening and reservoir head. This is illustrated on Figure 6, which shows the discharge capacity curve, as determined from a hydraulic model, for Gardiner Dam on the South Saskatchewan River. The lower curve gives the head-discharge relationship for the ogee weir crest alone, such as would occur with the gates open far enough to not interfere with the flow over the weir. To meet this requirement the gate opening would have to be equal to at least three-quarters of the head. This would allow the top streamline to clear the bottom lip of the gate. With the gates partially closed the reservoir will be retained against the face of the gate, and the flow is classified as underflow rather than overflow. Although the flow is similar to flow under a sluice gate, it is not easily calculated. The coefficient of discharge depends upon the curvature of the weir crest, the radius of the radial gate, the position of the gate seat, the elevation of the gate pin, and the height of the gate opening. For this reason it is usually necessary to determine the discharge capacity by a model test.

Since in theory radial gates are never fully open except at design discharge, most discharges will occur as underflow. As a result, ice or other floating debris will be trapped in the reservoir. Log booms may be used to keep debris away from the spillway if necessary.

7. <u>Drum Gate</u>

The drum gate consists of a hollow watertight vessel, triangular in cross-section, and hinged at the upstream end. It is fabricated as a structural steel frame with steel skin plates. The skin plates must completely enclose the gate to make a bouyant vessel. The gates fit in a large recess in the crest of the spillway, as shown in Figure 7. The appearance of a pair of drum gates from the downstream side are shown in Figure 8.

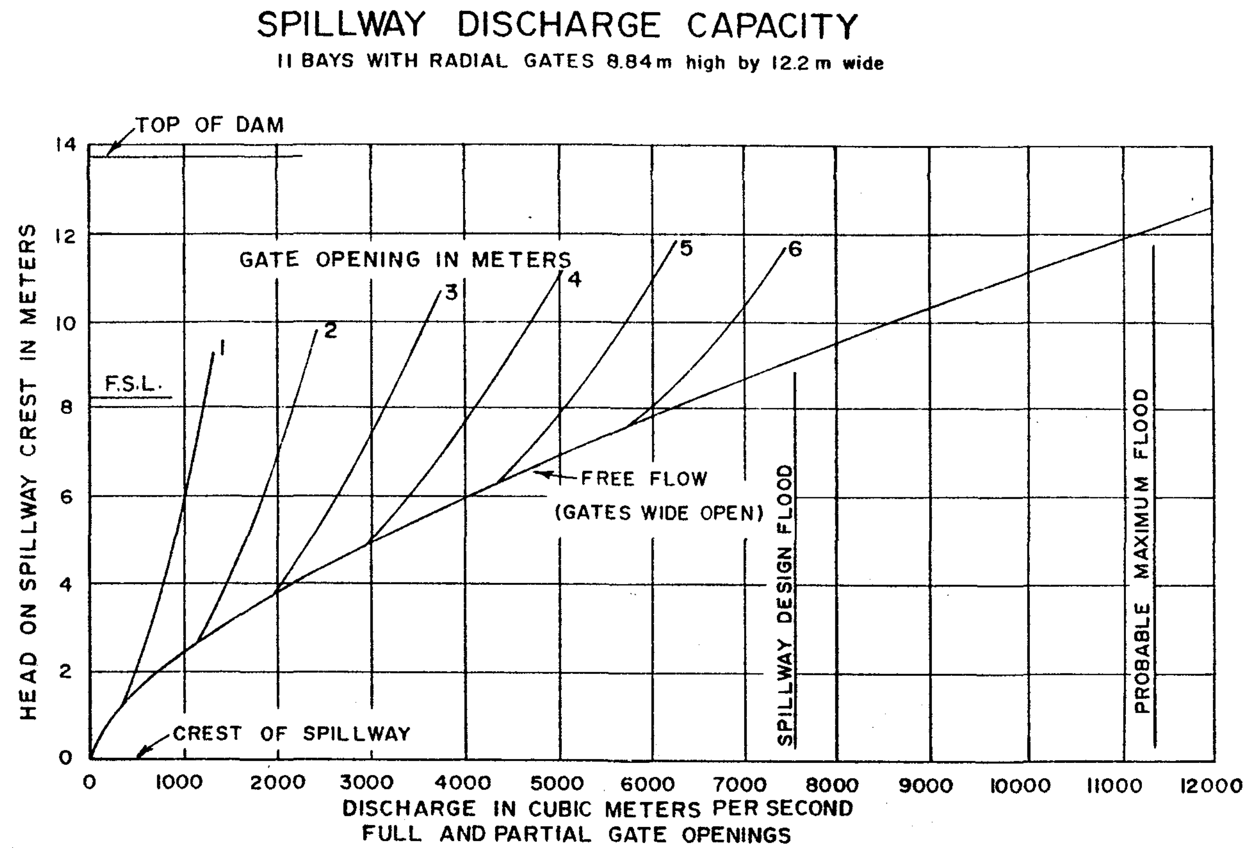

Figure 6. Spillway Discharge Capacity Curve, Gardiner Dam

The interesting feature of the drum gate is that it is opened and closed by water pressure from the reservoir. The gate can be held in any position, and it can be controlled automatically from a float well connected to the reservoir. When the gate is wide open, the gate is completely enclosed in the crest recess and the shape of the upstream skin plate matches the natural trajectory of the flow over the crest of the spillway.

Since the drum gate is operated by water pressure, hoists are not required. A further advantage is that there is free surface overflow for all discharges, large or small. The free overflow attainable with the drum gate is advantageous where it is desirable to discharge ice flows or floating debris which would otherwise accumulate on the reservoir.

The water load on a drum gate is not transmitted to the piers, but is carried over the whole length of the gate by the continuous hinge. Accordingly, the span is not limited by gate loading, nor by hoist capacity, as there are no hoists. This gate has been built in sizes up to a 41 m span, as used at Grand Coulee Dam on the Columbia River.

Despite these advantages the drum gate is usually restricted to high concrete gravity dams where the large gate chamber recess can be accommodated in the crest. In chute spillways where the H/P ratio is greater than unity, the depth of the crest would not be sufficient to accommodate the drum gate in the open position.

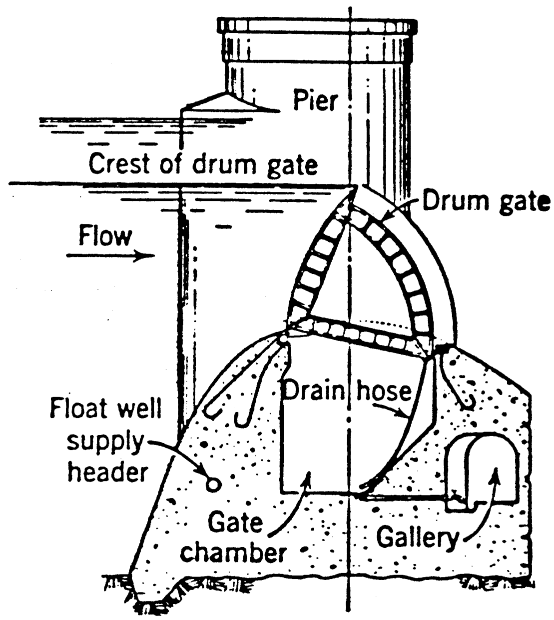

Figure 7. Drum Gate (Closed)

Figure 8. Drum Gate, Rock Creek Dam, California

8. Rolling Gate

The cross-section for a rolling gate is shown in Figure 9. This gate consists of a structural steel frame and steel skin plate to form a hollow steel cylinder, as shown for Imperial Dam on Figure 10. The diameter of the cylinder is somewhat less than the damming height, the difference being made up with an apron on the bottom. The apron has a cusp to give a fixed spring point when the gate is operated in sluice fashion. The gate is operated by rolling it up an inclined track. It can be rolled up to be completely clear of maximum flood stage. In some cases it has been designed to be rolled down slightly below FSL to allow a free overflow for skimming action should this be desired.

The gates are installed horizontally between piers, which must have deep wide recesses in order to accommodate the full diameter of the cylinder. Because of the great bending strength of the cylinder, the rolling gate can be used for large spans. This is useful where a large unobstructed opening is desired. The gate is a European development, and has been used up to a 45 m span.

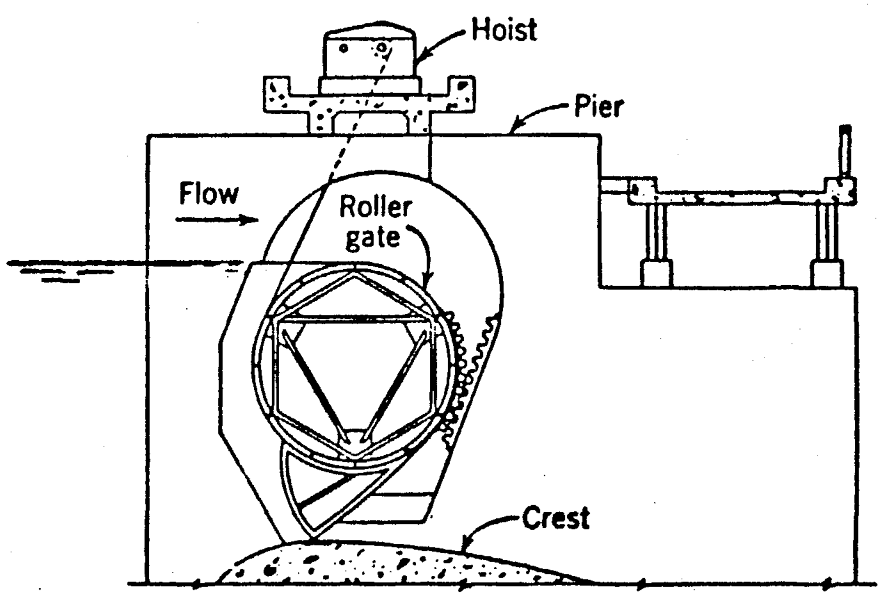

Figure 9. Rolling Gate (Closed)

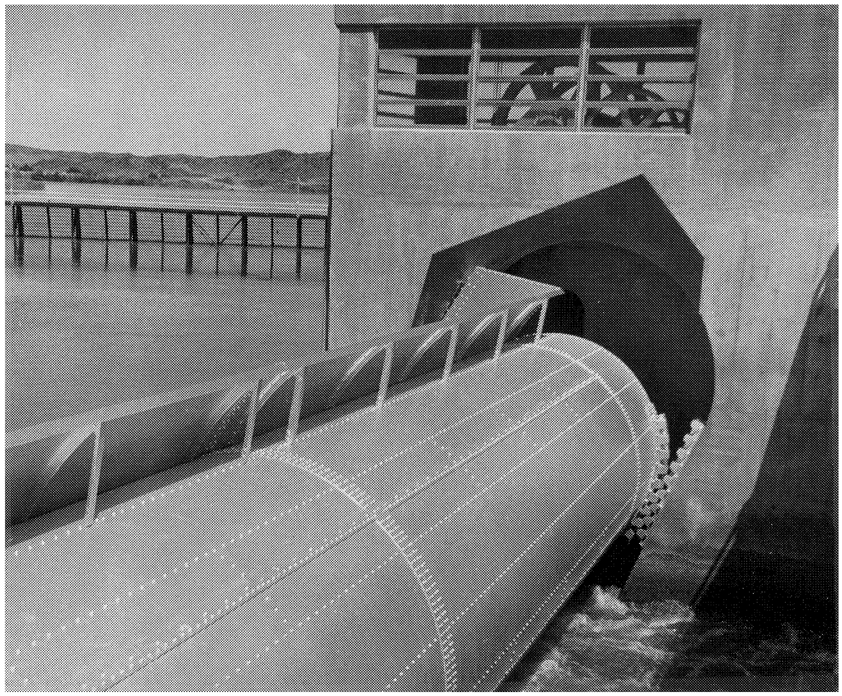

Figure 10. Rolling Gate, Imperial Dam, California

9. Ring Gate

The ring gate is shown in Figure 11. This is an adaptation of the drum gate idea to the circular crest of a shaft spillway. The gate consists of a hollow buoyant annular drum which is circular in plan. The gate fits in a recess in the circular crest of the spillway and is operated by water pressure. When the gate is fully lowered into the crest recess, the upper surface matches the crest and gives a smooth uninterrupted flow line. Ring gates have been constructed up to 18 m in diameter.

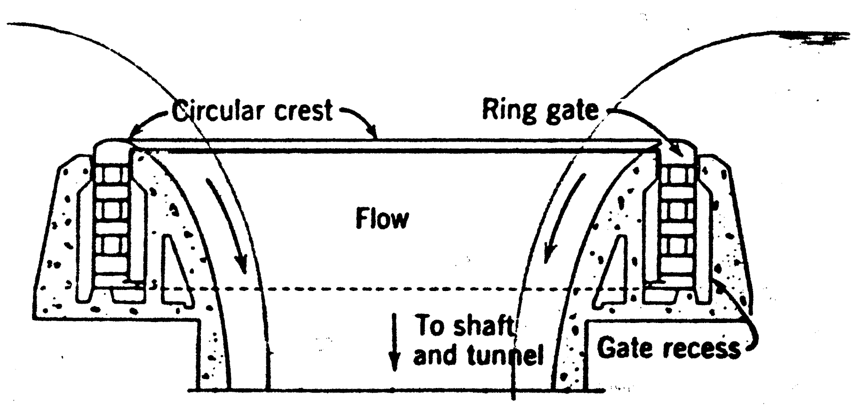

Figure 11. Ring Gate (Open)

C. OUTLET GATES

10. Gate Types

The gates used for outlet works, sluice openings, or power penstocks are of the same general type as the underflow gates used for spillways. In this case, however, circular gates may be used as well as square and rectangular shapes. Slide gates, wheeled gates, and radial gates have all been used on outlets.

Outlet gates may be divided into 2 categories: (1) submerged face gates; and (2) enclosed gates with bonnet.

The submerged face gates are not sealed off in any way from the reservoir, which means that reservoir pressure can act on the top of the gate as well as on the front. This occurs for any gate which is mounted on the upstream face of the dam, or for a gate which is mounted in a wet control well. The water depth is the same in a wet well as the reservoir depth when the gate is closed.

Enclosed gates may be used inside concrete gravity dams. They are located below a gate operating gallery, and have a bonnet covering them which is capable of resisting full reservoir pressure.

11. Gate Problems

Gate design for an outlet is considerably more difficult than for a spillway because outlet gates are submerged and in a confined space, and must operate under much greater heads. In contrast, spillway gates are easily accessible for inspection and can be simply dewatered by placing stoplogs in pier slots upstream. Spillway heads are usually less than 10 m, whereas for outlet gates the heads may be several times this value. The much larger head has far reaching consequences in design.

The design problems for any gate are sealing, lifting, cavitation, vibration at partial opening, and accessibility. These problems are more difficult for higher heads, and often require special designs for their solution, as shown in Table 1.

12. Sealing

Sealing is more difficult for outlet gates because of higher pressures and because a continuous seal is required around the entire perimeter of the gate. Obviously the slide gate offers the best solution to the sealing problem. However, the high water pressures lead to heavier gates and greater friction forces, thus limiting the practical size of the gates.

In wheeled spillway gates the side seals are attached to, and travel with, the gate. It is difficult to do this for the top seal on wheeled outlet gates. A ring seal has been developed, consisting of a rubber O-ring mounted in a recess which runs around the perimeter of the gate opening. When the gate is closed the ring is pressurized and makes a watertight contact with a seat on the face of the gate.

TABLE 1
DESIGN PROBLEMS FOR HIGH HEAD OUTLET GATES

Design Problem	Reason for Increased Difficulty as Compared to Spillway Gates	Consequences in Design
Sealing	- higher pressure - top seal required	- special pressure seals or special gates
Lifting	- larger normal water force - vertical water force	- smaller gates or bigger hoists - special lip design
Cavitation	- higher velocities	- special slot design - special gate design
Vibration	- larger air demand	- air vents required
Accessibility	-gates are submerged and in a confined space	- guard gates required

A method of sealing used for the caterpillar gate involves the use of an inclined seat. The gate seat and frame seat are in full contact only when the gate is closed. Sliding or wedging of these two inclined surfaces must be avoided so it is necessary to provide some means of adjustment.

Recently radial gates have been adapted to outlet conduits because of their easy opening and closing features and fast operation. The big disadvantage of radial gates in an outlet is the space required. Usually the gate arms are longer than the gate height, thus a large chamber is required to contain the gate. Sealing has been accomplished using solid rubber seals and an eccentric trunnion which pushes the gate ahead against the seals when it is closed.

13. Lifting

Three factors make the lifting force greater for an outlet gate than for a spillway gate of the same size. First, since the normal water force is greater, the beam sections of the gate must be deeper, increasing the weight. Second, the frictional resistance, either sliding or rolling, increases as the water load increases. Finally, a vertical water force, known as hydraulic downpull, must be overcome in lifting the submerged type of outlet gates. The effect of these factors is that outlet gates must usually be made much smaller than spillway gates.

Hydraulic downpull is the downward hydraulic force due to unbalanced vertical water pressure on top of submerged face gates, when in the open or partially open position. This is a very significant force in outlet gate design, and unless given special attention, it can amount to many times the weight of the gate. This force does not occur with spillway gates, as the pressure is atmospheric on the top and bottom of the gate. With outlet gates the pressure on top of the submerged gate may equal the full reservoir pressure, whereas the pressure on the bottom of the gate may be atmospheric, or in some cases less than atmospheric. It will be atmospheric for free flow downstream, and it may be less than atmospheric if the outlet is full and pressures on the gate bottom are reduced by high local velocities. This differential pressure may produce a large vertical force because of the great thickness and top area required for the gate.

The hoist capacity must include hydraulic downpull, so consideration must be given to partial relief of this force. This can be done with a special lip design at the bottom of the gate, such as shown in Figure 12. This design will produce at least some pressure on the gate bottom in spite of high underflow velocities.

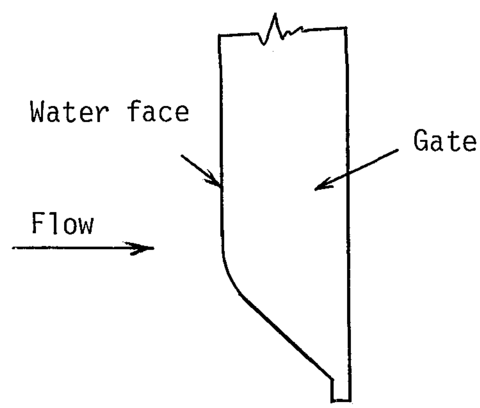

Figure 12. Gate Lip to Reduce Downpull

14. Cavitation

Cavitation potential is a function of flow velocity. One source of cavitation is around the gate slots. A rectangular square cornered slot, which is the simplest, is often used for spillways as a matter of convenience. However, flow separation at the exterior corners has been found to be a source of cavitation at high velocities. Cavitation damage occurs on the side walls of the conduit downstream from the slot. A recommended slot developed by the United States Waterways Experiment Station to give streamline relatively cavitation free flow past the slot is shown in Figure 13.

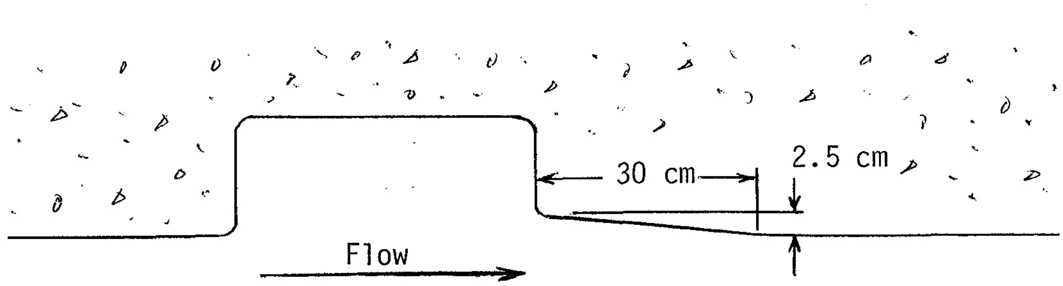

Figure 13. Cavitation Free Gate Slot

At heads of 75 m and up, cavitation damage has been experienced on gate slots even with special slot design. In this case a special gate design is required, such as the ring-follower gate. This is an enclosed slide gate with a bonnet, adapted to circular outlets up to about 2.5 m in diameter. The gate leaf is more than twice the height of the opening. The top half of the leaf is solid, the bottom half of the leaf has a circular opening the exact size of the conduit. When the gate is open there is a perfectly smooth water passage through the opening in the gate leaf. The slots in which the gate slides are not exposed. This gate must be used fully open or fully closed, as it is not adapted for operation at partial gate openings. Discharge regulation, if required, may be obtained by using a large number of gates of smaller size.

15. Vibration and Air Demand

Outlet gates must have air vents. An air vent is a duct or pipe connecting the conduit immediately downstream from the gate to atmosphere. This is necessary to replenish the air which is removed from the flow in the outlet by the ejector action of the the flow in the outlet when operating at partial gate opening. If this air is not supplied, pressures drop and cavitation potential is increased. Severe vibration of the gate may result. This vibration is due to the transient nature of the pressures, and is called gate chatter.

It is evident that the air flow is zero when the gate is closed or when the gate is fully open. In the latter case the air vent behaves as an open piezometer, with water standing in the vent at the elevation of the hydraulic grade line. Between closed and full open, air is drawn out of the conduit for various reasons, as follows:

(1) Air drawn out of the space between the water surface and the crown of the pipe, due to the surface drag effect of the high velocity water;

(2) Air ejected or entrained by spray action around slots or seals;

(3) Air entrained in the water flow due to turbulence;

(4) Air evacuated by the hydraulic jump, should one occur in the outlet;

(5) Air required to compensate for reduction in discharge during rapid gate closure on a flowing full pipe.

Campbell and Guyton have proposed the empirical equation

[5] $$\beta = 0.04\,(\mathbf{F} - 1)^{0.85}$$

in which β is the ratio of air flow to water flow (Q_A/Q_W), and **F** is the Froude number for the flow just downstream from the gate. Equation [5] is the equation of an envelope curve for prototype tests conducted on five large dams by the United States Corps of Engineers, and is conservative. It may be assumed for purposes of calculation that the maximum air demand occurs with the conduit flowing half full.

In sizing the air vent the air velocity should not be allowed to exceed 45 m/s, and the pressure drop in the air vent should be limited to 2 m of water.

Example 2:

Compute the air vent size for a 3 m x 3 m outlet to operate under a 60 m head.

The water flow velocity

$$V = \sqrt{2gh} = \sqrt{2 \times 9.81 \times 60} = 34.31 \text{ m/s}$$

For half full flow the depth will be 1.5 m, for which the Froude number

$$\mathbf{F} = 34.31/\sqrt{1.5\, g} = 8.94$$

and

$$\beta = 0.04(\mathbf{F}\text{-}1)^{0.85} = 0.04 \times 7.94^{0.85} = 0.233.$$

Since $Q_W = VA = 34.31 \times 1.5 \times 3 = 154.4 \text{ m}^3/\text{s}$, then

$$Q_A = 0.233 \times 154.4 = 35.97 \text{ m}^3/\text{s}.$$

For 45 m/s velocity, the required air duct area is $35.97/45 = 0.8 \text{ m}^2$.

A square duct 0.9 m x 0.9 m will meet this requirement. The pressure drop across the duct may be calculated closely assuming a friction factor of 0.02 and transition losses of 1.5 $V^2/2g$. If the air vent is vertical it must be 60 m long to reach atmosphere, hence the pressure drop will be

$$\Delta P/\gamma = (0.02 \times 60/0.9 + 1.5)\, 45^2/2g = 292.4 \text{ m of air.}$$

The allowable pressure drop is 2 m of water, which is equivalent to 2 x 9810/11.8 = 1663 m of air, and greatly exceeds the required pressure drop. The air expansion due to pressure drop can be neglected as it would be almost negligible for this small pressure change, and any error is on the safe side.

16. Guard Gates

Because of the possibilities of trouble at outlet gates, and because of their relative inaccessibility, it is common practice to provide for more than one means of shutting off the flow. This practice is desirable as a second line of defense in the event that trouble is experienced with the service gate. One means of doing this is by the inclusion of a second gate, called a guard gate or emergency gate. The guard gate is usually located just upstream from the service gate.

Difficulties arising from track wear, gate oscilliation, residual strain, or cavitation damage are not anticipated for the guard gate because it is operated less frequently than the service gate. For ordinary inspection and repair of the service gate, it is not even necessary to raise or lower the guard gate under a differential head, so it is expected to remain in good condition. The guard gate is normally left in the open position, even when the service gate is closed.

D. VALVES

17. Needle Valve

The components of a needle valve are shown in Figure 14. The valve body consists of a bulb shaped steel outer jacket which is bolted to the end of the penstock. The diameter is about 50% greater than the diameter of the penstock. The valve proper consists of a fixed part and a sliding part. Both parts are pointed or needle shaped at the ends. The diameter of the valve body is slightly greater than the diameter of the penstock.

The fixed part of the valve has the needle pointing upstream, and is anchored to the valve body by fixed vanes. The vanes are thin and lined up in the direction of flow to minimize losses. The sliding part has the needle pointing downstream. It may be moved horizontally so as to regulate the size of the annular opening at the end of the valve. When the sliding part of the valve is fully extended (to the right in Figure 14) the valve body makes contact with the circular seat and at the end of the valve body and the flow is shut off.

The needle valve produces a solid circular jet under all conditions of valve opening. It is frequently used on arch dams where the discharge must be directed into a narrow canyon downstream. It has also been used submerged below tailwater. The disadvantage of the needle valve is the probability of cavitation pitting around the needle when operated at small openings. Also, the valve is quite expensive because the valve and valve body must be designed to withstand full reservoir hydrostatic pressure.

The coefficient of discharge C_d is about 0.6, as used in the equation

$$Q = C_d A\sqrt{2gh} \qquad [6]$$

in which A is the nominal size of the valve (equal to the area of the penstock), and h is the total head acting across the valve. The coefficient applies when the valve is in the wide open position.

18. Tube Valve

The components for a tube valve are shown in Figure 15. The tube valve is a variation of the needle valve. Essentially the tapered downstream portion and needle are cut off, so the sliding portion of the valve is just a short cylinder or tube. This reduces the cost of the valve and eliminates the cavitation problem, but in the absence of a smooth guiding boundary the jet is not as smooth. There is more spray and the jet tends to be unstable at openings less than 35% of maximum. The coefficient of discharge is somewhat lower than for the needle valve.

19. Hollow-Jet Valve

The hollow-jet valve is shown in Figure 16. This is really a further variation of the needle valve. In effect the entire downstream half of the needle valve is omitted. At the end cross section the jet is an annular ring, hence the designation hollow jet.

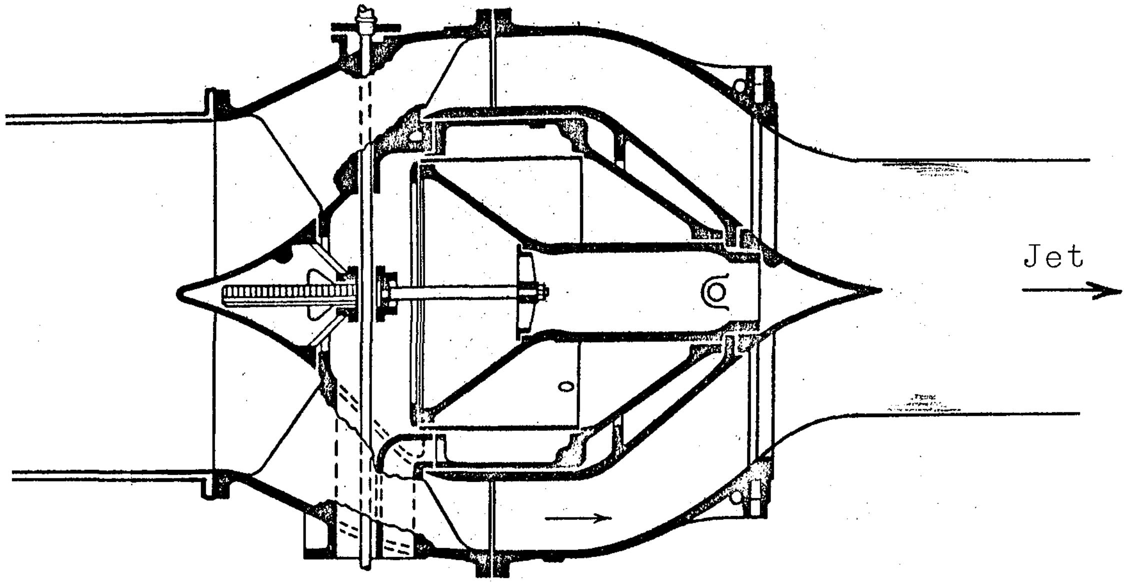

Figure 14. Needle Valve

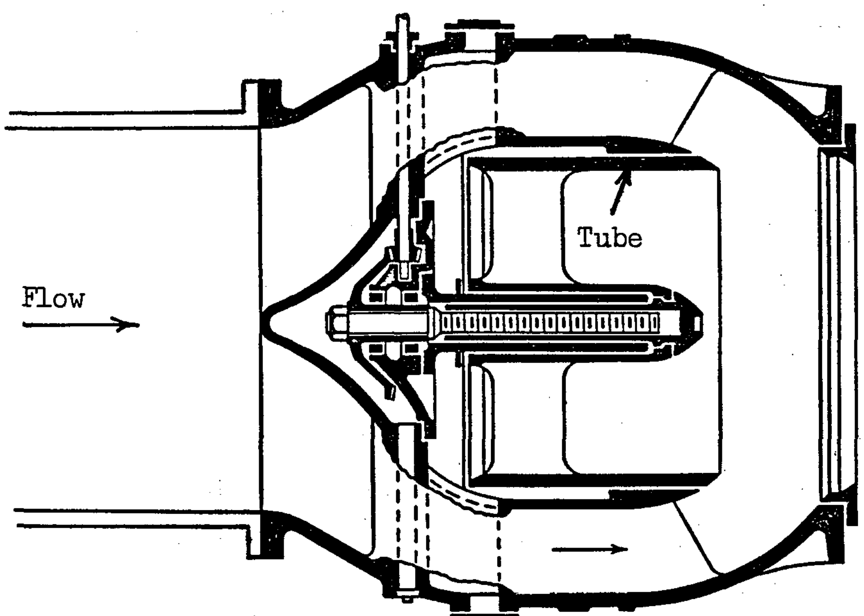

Figure 15. Tube Valve

An important feature of the hollow-jet valve is that the sliding part is upstream instead of downstream. When the valve is closed, the sliding part seats at the upstream end of the housing, and the valve body is at atmospheric pressure. The hollow jet produces a constant outside diameter jet at all openings. At design discharge the diameter of the hollow jet is twice as great as the jet from a needle valve for the same discharge. This feature is an advantage if the valve discharges into an energy dissipating structure, such as a hydraulic jump basin. The constant width jet is more amenable to treatment in a structure of fixed dimensions. It is not recommended that the hollow-jet valve be used for submerged operation. The coefficient of discharge is about 0.7 (wide open).

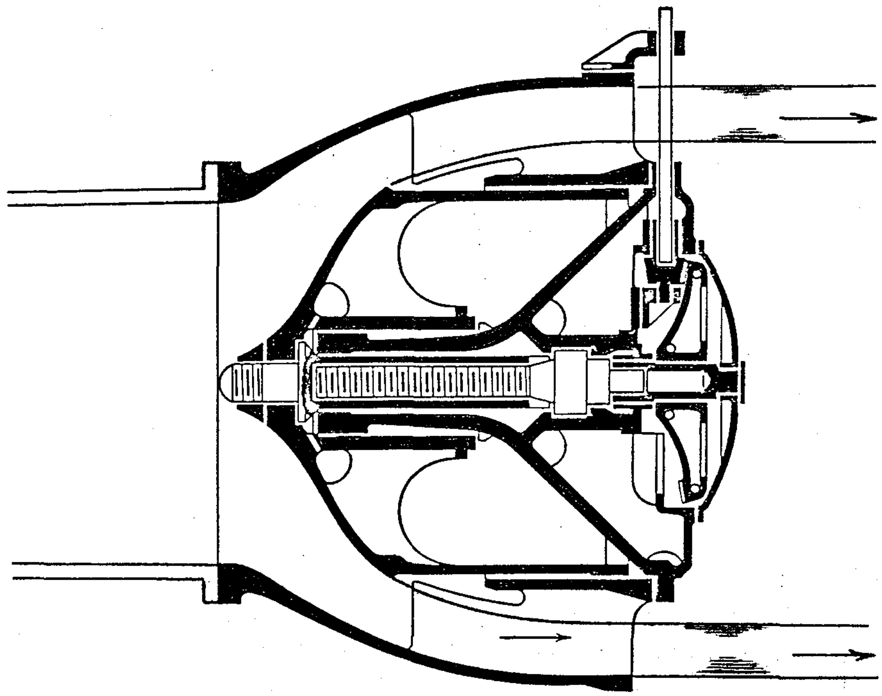

Figure 16. Hollow-Jet Valve

20. Howell-Bunger Valve

A cross section of this valve, also known as the cone valve, is shown in Figure 17. A fixed 90° cone is held in position at the end of penstock with vanes. The cone deflects the discharge away at 45° to the axis of the pipe forming a diverging hollow conical jet, resulting in dispersion of the flow over a wide area. The discharge is regulated with a sliding cylindrical sleeve which fits around the outside of the penstock.

The cone valve is simple, inexpensive, and free of cavitation. It has been used for heads up to 200 m. Energy is largely dissipated in atmosphere. A particular advantage is that the operating mechanism is entirely external. There are no moving parts inside the diameter of the penstock. The principal hydraulic forces on the sliding sleeve are radial.

Due to extensive lateral spray the cone valve requires a large receiving area downstream and is unsuitable for discharge directly into a canal or structure. The fine spray may be undesirable for cold weather operation. A variation of the cone valve, called the ring-jet valve, may be used to confine the discharge. A fixed hollow cylinder, with a larger diameter than the penstock, is placed around the cone at the end of the valve. The cylinder redirects the jet from a conical to an axial direction. The action is similar to that for the hollow-jet valve, however there is some backsplash. The coefficient of discharge for the cone valve is 0.85, and for the ring-jet, 0.78.

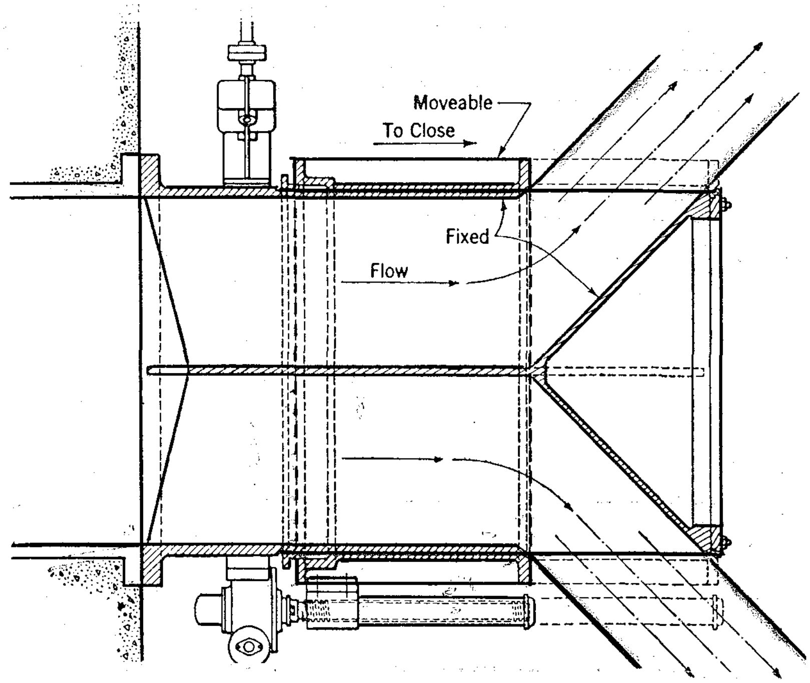

Figure 17. Howell-Bunger Valve

21. Butterfly Valve

This valve consists of a flat circular leaf, which pivots about an axis on its vertical diameter, and which is mounted inside a valve body with circular cross section. In large sizes, the valve diameter usually exceeds the penstock diameter in order to account for the loss of cross-sectional area due to the thickness of the valve leaf.

The valve is frequently used in a power penstock immediately upstream from the turbine scroll case. In this application the valve is left fully open or fully closed. When open, the leaf is in line with the flow to minimize head losses. At partial openings the discharge tends to be unsteady and the torque at the pivot is appreciable. Hence, the valve is not suitable for flow regulation, and is never used to discharge directly to atmosphere.

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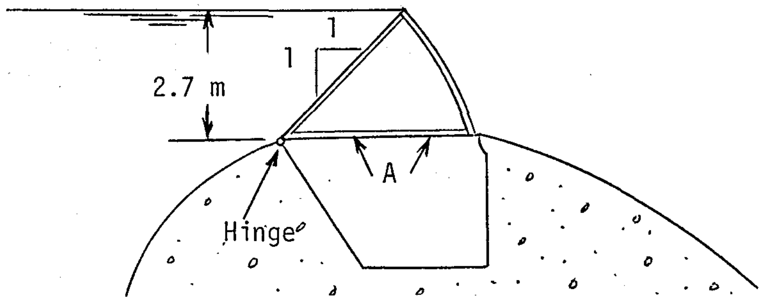
PROBLEMS

1. What types of spillway gate require:
   (a) No hoists
   (b) Screw-type hoists
   (c) Drum-type hoists

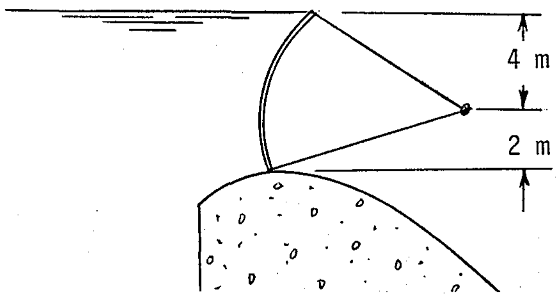
2. Calculate the diameter required for the steel stem for a 3 m by 6 m wide vertical slide spillway gate with bronze seats. The gate weighs 90 kN.

3. Sketch and label the principal components of a drum gate in the closed position. Why can longer spans be used for this gate than for the radial gate?

4. What pressure head must be maintained at A to keep the drum gate in the position indicated? Neglect the gate weight.



5. Determine the pin reaction for the radial gate due to the reservoir load. The gate is 12 m wide.



6. Discuss the conditions which make gate design a more difficult problem for an outlet than for a spillway.

7. Explain a procedure which makes it possible to check the operation of both the emergency and service gates at an outlet works without having to operate either one under a differential head.

8. Expalin hydraulic downpull, and measures which may be taken to reduce it.

9. Compute the size of vent pipe required to supply air to a 1.5 m square outlet under 45 m of head, using $\beta = 0.04\ (F\text{-}1)^{0.85}$ as the criterion.

10. Why are valves used instead of gates for high head flow regulation on an outlet?

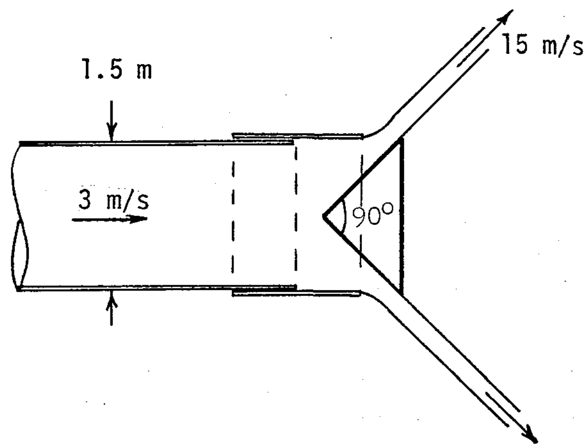
11. Explain the construction of a hollow jet valve. What advantage does it have over the needle valve?

12. Which type of valve produces a jet most suited to:

   (a) Discharge into a stilling basin
   (b) Discharge into a deep narrow rocky channel
   (c) Energy dissipation in the atmosphere.

13. Distinguish between the terms ring seal, ring gate, ring-follower, and ring-jet.

14. Calculate the longitudinal force on the cone of the Howell Bunger valve under the conditions indicated. Neglect all losses.



CHAPTER V

DIVERSION WORKS

A. GENERAL

1. Function and Components of a Diversion Works

A diversion works is a group of structures constructed for the purpose of diverting all or part of a stream from its natural course into an artificial channel. The artificial channel is usually a canal, but it may be a bench flume or a pipe. The channel is used to convey the water from the diversion works to some other point in the project. This may be a power plant, a storage reservoir for irrigation or water supply, or even another drainage course.

The principal components of a diversion works are the diversion dam and the intake structure to the artificial channel. Auxiliary structures, such as sluiceway, logway, or fishway, may also be required in many cases. When used, these must be designed as a part of the diversion works.

Diversion dams differ from storage dams in that they are low height dams with small or negligible storage. Of course high storage dams may also be used for diversion purposes, but they are not referred to specifically as diversion dams. Diversion dams are suitable where the natural flow is adequate to meet the demand without the need for storage. In addition, the topography must be such that the diversion can be made at the dam with a low head. This will be the case if the level of the surrounding terrain is close to the level of the river. The area to be served must be at a lower level if supply by gravity flow is required.

The intake structure at a diversion works is gated to permit regulation of the flow, and is often called a headgate. A sluiceway may have to be used in conjunction with the headgate, depending upon the type of diversion dam. A sluiceway is an auxiliary structure used to prevent a heavy silt load from passing through the headgate into the canal.

Logways may be needed in timber regions where logs must be floated downstream. This is a consideration apart from the headgate and sluiceway. A logway consists simply of a narrow chute over one end of the diversion dam. To conserve water when not in use the logway should have some means of control at the inlet. Fishladders will be required where upstream passage must be provided for migratory fish. The most common fishladder designs consist of multiple pools separated by weirs, orifices, or vertical slots.

B. DIVERSION DAMS

2. Requirements

There are three requirements to be met by a diversion dam:

(1) The dam must provide for the passage of the undiverted flow;

(2) The dam must create enough head to effect the diversion;

(3) There must be provision to discharge accumulated silt deposits.

Frequently there is no flow being diverted during periods of high flood. The undiverted flow is then the entire stream flow, and since the storage is negligible, the dam must be designed to pass the maximum flood anticipated, undiminished by storage. Usually a separate spillway is not feasible or economical at a diversion dam. Instead, the dam may consist of an overflow section or a gated structure spanning the entire width of the stream. If it is not necessary to use the whole length of the dam for a spillway, then the non-overflow portion may be constructed of earth, rock, timber, mass concrete, or reinforced concrete.

The head created by the dam must supply the depth and velocity head for the offtaking canal, as well as the head losses through the headgate structure. Since the diversion flow may be at the design discharge during the season of the year when the stream flow is low, the weir head on the overflow section may be small. The required weir crest elevation El_W for the overflow section of a diversion dam is given by

[1] $$El_W = El_C + D + h - H$$

in which El_C is the bed elevation of the offtaking canal, D is the flow depth in the canal at design flow, h is the head across the headgate with the gate wide open, and H is the head on the weir corresponding to the minimum expected weir discharge during the diversion period. The head h will usually be kept small, in the range of 0.15 to 0.30 m. Although this will necessitate the use of low velocities and will increase the size and cost of the headgate, it will usually decrease the cost of the dam to a greater extent because of the reduction in El_W.

The dead storage volume created by a diversion dam is small because of the low height of the dam. The result is that the dam may become silted up to the crest within a few years after construction. If this silt is allowed to pass into the canal system, it would form deposits which would lower the canal capacity or require frequent removal. Provision must be made to dispose of this excess silt before it enters the intake. A sluiceway, which is really part of the dam, may be used for this purpose.

3. Types of Diversion Dam

A diversion dam may be an open dam or an overflow dam. An open dam consists of a series of open bays between piers which support gates silled at the bed level of the river. A separate sluiceway is not required with an open diversion dam because the silt can be passed directly through the dam. During flood periods all gates may be wide open, in which case there is a minimum of obstruction to the flow. The open dam is advantageous in areas where the silt load is abnormally high. A second advantage is that upstream flood levels are lower than for a fixed crest overflow dam. This is important on a navigation river or if there are developed areas upstream which may be flooded. The disadvantage of an open dam is that the gates require operation and maintenance.

The overflow diversion dam is more common than the open dam. It is frequently constructed as a solid gravity overflow section with a horizontal hydraulic jump apron at the base of the weir. Although a separate sluiceway is required, overflow dams are generally more simple, durable, and offer less obstruction to the passage of ice or floating debris than an underflow gated structure.

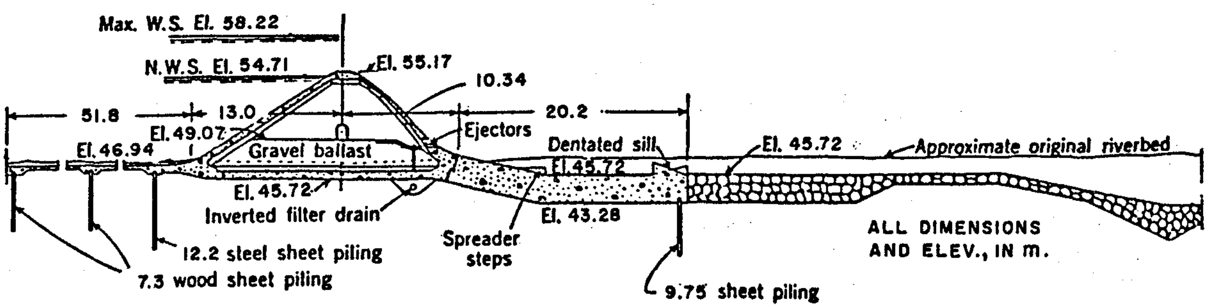


Figure 1. Cross section of Imperial Dam

An example of an overflow diversion dam, Imperial Dam on the Colorado River, is shown in Figure 1. Features of this dam are the hollow cross section with gravel ballast added for weight, upstream sheet piling to reduce seepage, and thick apron slab to resist uplift produced by the hydraulic jump. Pressure relief under the dam is achieved by a perforated transverse drain located in an inverted filter under the base at the downstream toe. This drain is connected to vertical pipes which terminate inside the hollow section of the dam above the gravel ballast. Any water which collects here is discharged by gravity flow through the ejectors to the downstream face. The ejectors are shaped so they will operate even when water is flowing over the dam. The maximum pore water pressure head under the base at the position of the filter drain cannot exceed the height of the vertical connecting pipe, or 3 m, even though the reservoir head may be up to 12.50 m.

Figure 2 shows a section view of a two-dimensional model of the Belly River Diversion Weir, located in Alberta. The water surface profile shown corresponds to the design flow (unit discharge 7.72 $m^3/s/m$), and the drop from headwater to tailwater is 5.0 m.

4. Design Features

The overflow section for a diversion dam is shaped to fit the natural trajectory of the nappe at design flow. A curved bucket should be used to deflect the flow onto a horizontal apron. The apron should be long enough to protect the erodible bed downstream. Diversion dams may have to pass considerable quantities of floating material every year, so basin floor blocks, if used, could be subjected to damage and require maintenance. A common solution to this problem is to combine the blocks with the end sill and leave the apron plain. A sill constructed in this manner is called a dentated sill, as used for Imperial Dam (Figure 1).

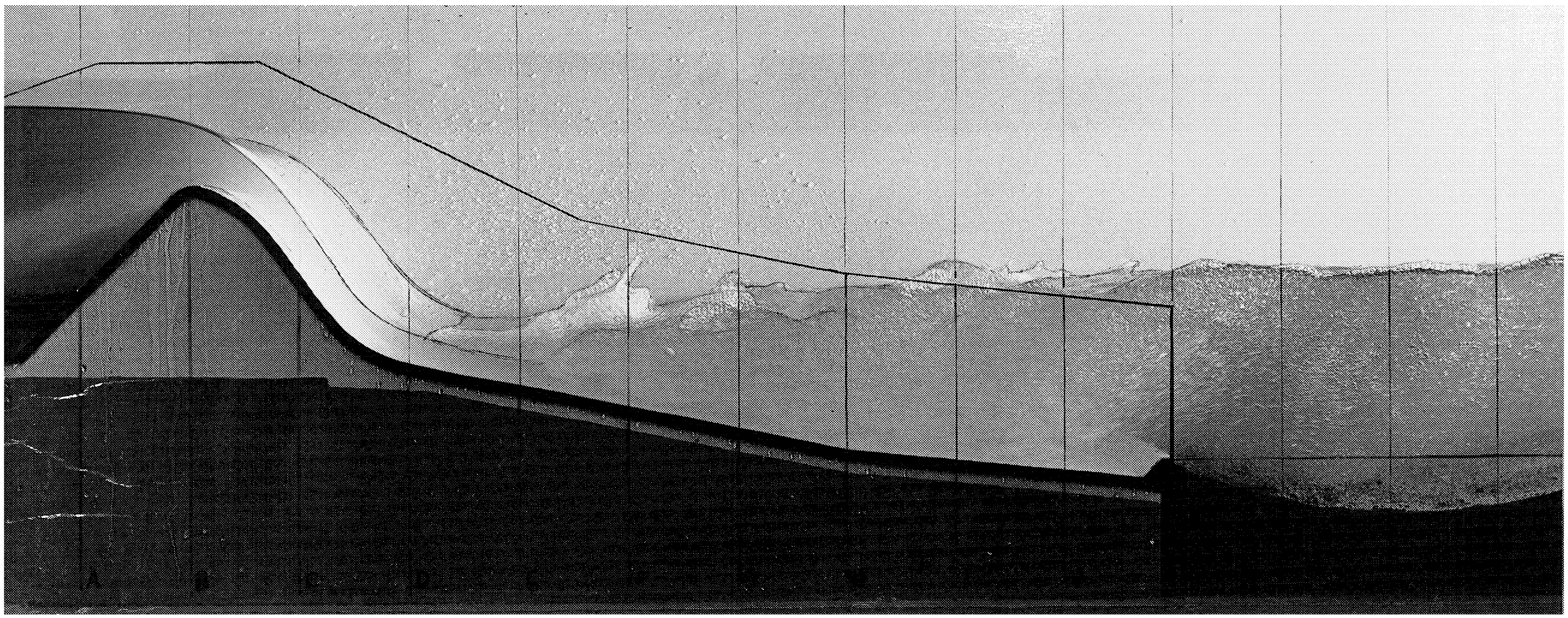


Figure 2 Model of Belly River Diversion Dam Overflow Section

The tailwater depth at a diversion dam is usually more than required to keep the hydraulic jump in the basin, even though the apron may not be depressed below the downstream bed level. This arises because the discharge intensity and drop height are much lower than for storage dam spillways. Moreover, it is characteristic that the tailwater level rises faster than the headwater level as the discharge increases. In fact, at maximum flood, the weir may become completely submerged or "drowned" by the tailwater, as indicated by Figure 3.

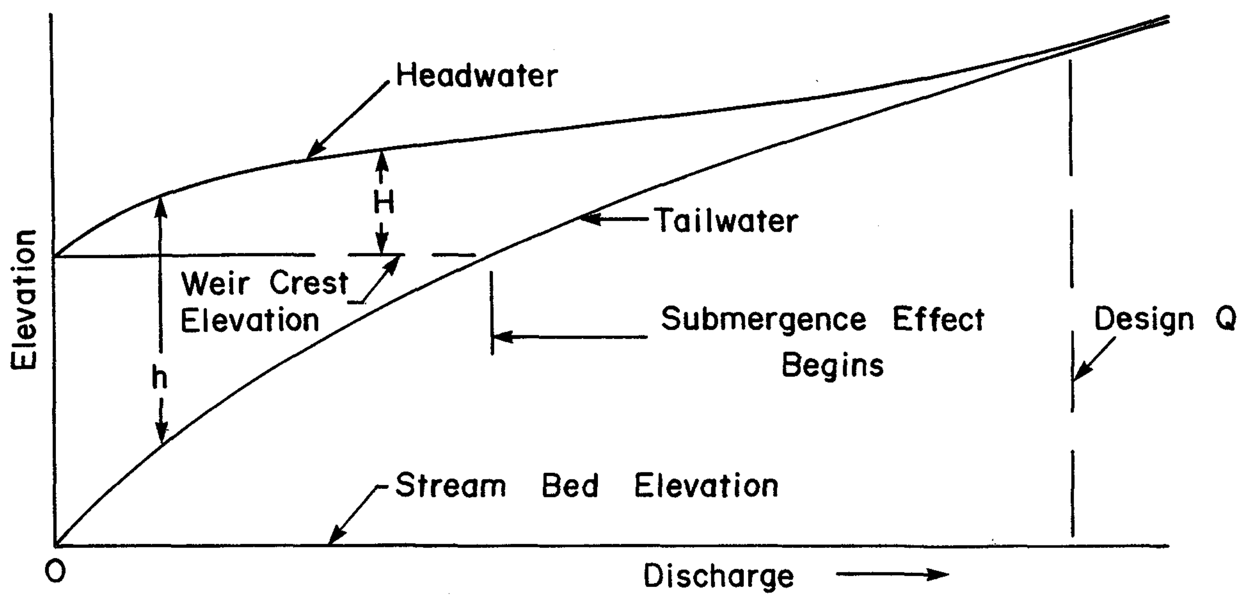


Figure 3. Typical Stage-Discharge Curves for a Low Height Weir

In designing the stilling basin it may be necessary to consider the variation in the tailwater rating curve with time. Initially the downstream channel should degrade as the stream sediment is trapped upstream in the reservoir. Eventually, however, this sediment load will be passed through the sluiceway. If the diverted flow is a large percentage of the incoming stream flow, the remaining channel discharge may not be large enough to transport the silt load downstream, and the downstream channel may actually aggrade. In this case the tailwater elevations for a given discharge would gradually increase with time, and ultimately may be greater than they were initially.

An interesting feature of the curves shown on Figure 3 is that the maximum differential head h across the structure occurs when the discharge is zero. This means that underseepage will be a maximum when overflow is a minimum, and this is the condition for which the upstream cutoffs are designed. Concrete cutoffs can be constructed by trenching and pouring if the foundation is cohesive, or by excavating, forming, and backfilling if the foundation is non-cohesive. Steel or timber sheet piling is used for driven cutoffs. Sheet piling is often less expensive, but is never perfectly watertight, and if there are boulders in the foundation it may not be possible to reach a suitable depth of penetration.

Diversion dams must be designed to resist the same kind of loads and satisfy the same stability criteria as other types of dams. The principal loads are water, silt, ice, uplift, earthquake, and self weight. Sliding rather than overturning stability invariably governs the design because diversion dams have a relatively broad base in comparison to their height.

Diversion dams are normally constructed directly on the natural river bed sediments because it is not economical to excavate to bedrock unless the depth of the sediment is unusually shallow. If the river deposit is deep it is possible that bedrock will not even be reached with the cutoff walls. Under these conditions, measures for controlling seepage and uplift pressure require special consideration, as discussed in Section 5.

5. Creep-Head Ratio

The creep path is the name given to the path followed by seepage flow as it percolates through the foundation from headwater to tailwater. If the percolation distance is too short, a steep hydraulic gradient will exist through the foundation, and transport of the particles may result. A progressive loss of foundation material may occur, starting at the point of egress of the seepage flow. This is called a piping failure because a continuous void or "pipe" develops in the foundation. Piping usually results in structure collapse due to loss of support.

Seepage flow will naturally follow the path of least resistance, and this may very well be along the contact line between the structure and the foundation because it is difficult to secure an intimate contact between the plane surfaces of the structure and the granular soil on which it rests. This was the basis for the original creep-head design method proposed by Bligh in 1912. The length of the creep path along the contact line is designated as L. This includes the vertical distances on both sides of the cutoff walls as well as the horizontal distances along the base of the dam and apron slabs.

The maximum differential head which may exist across the structure is designated as h. This is the head (headwater minus tailwater) which must be dissipated along the path taken by the underseepage. The ratio L/h is called the creep-head ratio, and this ratio must exceed a certain specified value, depending upon the foundation type, to insure safety against piping.

In 1935 Lane found that dams where more of the creep path length was vertical were safer against piping than where most of the creep path length was horizontal. He suggested that in calculating the creep path length, greater weight should be attributed to creep along steep or vertical surfaces than to flat or horizontal surfaces. The following arguments support this contention:

(1) Local settlements under the horizontal base of a structure may leave void spaces. This is referred to as "roofing" and is particularly dangerous in regard to piping possibilities. Settlement voids cannot occur along vertical surfaces, and hence the contact between a vertical cutoff and the foundation will be more intimate than under the horizontal base.

(2) Seepage through a stratified medium meets less resistance if it occurs parallel to the layers rather than perpendicular to them. Since natural river deposits are normally horizontally stratified, it follows that vertical creep

forced by cutoffs is more effective than a corresponding length of horizontal creep.

(3) Even in a homogeneous isotropic material, potential flow theory indicates that vertical cutoff is most important in preventing piping.

Lane studied some 278 dams, including many at which piping failures occurred, and proposed the following weighting factors: (1) Unity for contact surfaces steeper than 45°; (2) one-third for contact surfaces flatter than 45°; and (3) two for a creep path taken through virgin or undisturbed foundation material (called the short path). The value of L used in calculating the creep-head ratio is then the weighted creep distance and not the actual length of the path. The recommended safe values for the weighted creep-head ratio for different foundation materials are given in Table 1.

Frequently a reverse filter is placed under the downstream apron to collect seepage and prevent a pressure buildup. In this case the creep path along the contact line is calculated up to the position of the filter, which is considered as the point of egress of the seepage flow. The creep path length is reduced, but since a reverse filter also reduces the risk of a piping failure, it is permissible to use a smaller weighted creep-head ratio. Lane recommended a 20% reduction in the ratio, as shown by the values in the second column of Table 1. As well as decreasing uplift pressures, therefore, use of a filter drain may also decrease the required amount of vertical cutoff.

It is interesting to note that a silty foundation, which generally has the lowest shearing strength, lowest bearing capacity, and greatest potential for frost heaving, is also the type of foundation which is most susceptible to a piping failure. Boulders permit a high rate of seepage, but the particle size is so great that transport can be prevented even with a fairly low L/h ratio. Clay sizes are fine, but the combination of low permeability and high cohesion reduce suceptibility to piping. Silt sizes combine the properties of being coarse enough to permit significant seepage, and fine enough to be readily transported, so the highest L/h ratio is required to prevent piping.

TABLE 1
RECOMMENDED MINIMUM WEIGHTED-CREEP-HEAD RATIOS

Foundation Material	Without drain	With filter drain
Boulders and gravel	2.5	2.0
Coarse gravel	3.0	2.4
Medium gravel	3.5	2.8
Fine gravel	4.0	3.2
Coarse sand	5.0	4.0
Medium sand	6.0	4.8
Fine sand	7.0	5.6
Silt	8.5	6.8
Soft clay	3.0	2.4
Medium clay	2.0	1.6
Hard clay	1.8	1.5

Given the foundation type and the dimensions of the structure, the required depth of the cutoff to satisfy the weighted-creep-head ratio can be calculated. If this depth is greater than can be constructed, two or more cutoffs of reduced depth may be used instead. The space between the cutoffs should be at least equal to 6/5 times the cutoff depth, otherwise the full benefit of the multiple cutoff will not be obtained. The factor 6/5 originates from the requirement that the weighted creep length along the short path between the tips of the cutoffs should at least equal the weighted creep length along the contact line. In practice, a cutoff spacing of two or more times the cutoff depth is generally used.

6. Uplift

Uplift pressure acting on an overflow diversion dam presents two distinct possibilities for failure of the structure. Failure can occur by sliding of the dam, or by actual uplifting of the relatively thin apron slab on the downstream side of the overflow section. It is important that these possibilities be considered and properly designed for.

Bligh originally suggested that a reasonable approximation of the uplift pressure distribution could be found by assuming the head h is dissipated linearly along the creep path L. In line with his observations that vertical cutoff is more effective than horizontal cutoff. Lane suggested that the weighted creep path length should be used in the calculation. The pressure at various points may be found by applying the equation

[2] $$h_x/h = (L - L_x)/L$$

in which h_x is the piezometric head ($P/\gamma + Z$) at any point x along the creep path. This head is expressed relative to the tailwater level, which is taken as datum, and therefore the piezometric head is zero for any point in the tailwater. Since h is the difference between headwater and tailwater, then h is also the magnitude of the piezometric head for any point in the reservoir. By Lane's method L_x is the weighted creep path up to point x, and of course L is the total weighted creep path length. The length ($L - L_x$) represents the remaining path length along which the remaining head h_x is dissipated. Piezometric head must be used for h_x and h in order to account for the variation in pressure head with elevation, and the tailwater level must be taken as datum for this piezometric head. Equation 2 shows that at the point of ingress, where $L_x = 0$, then $h_x = h$; and at the point of egress where $L_x = L$, then $h_x = 0$. This must be true if a head of h is to be dissipated in a length of L.

The actual uplift pressure distribution depends upon the configuration of the base, the depth to impervious strata, the ratio of horizontal to vertical permeability, the intimacy of contact between the dam and foundation, and the location of filters, weep holes, or drains. For a homogeneous isotropic foundation, pressure distribution can be predicted by potential flow theory. The mathematics is rather complex, but the flow net, which is a graphical representation of potential flow, can be used to solve a variety of such problems.

Uplift pressure as determined by a flow net analysis for a simple dam is illustrated in Figure 4. The solid points show the calculated level of base pressure at each equipotential line. The open circles show the pressure level as determined from base piezometers on a two-dimensional model of the dam. The foundation for the model consisted of a deep bed of uniform Ottawa sand, carefully placed to secure close contact with the boundaries. In this case the actual uplift pressure (experimental) shows excellent agreement with the flow net (theoretical).

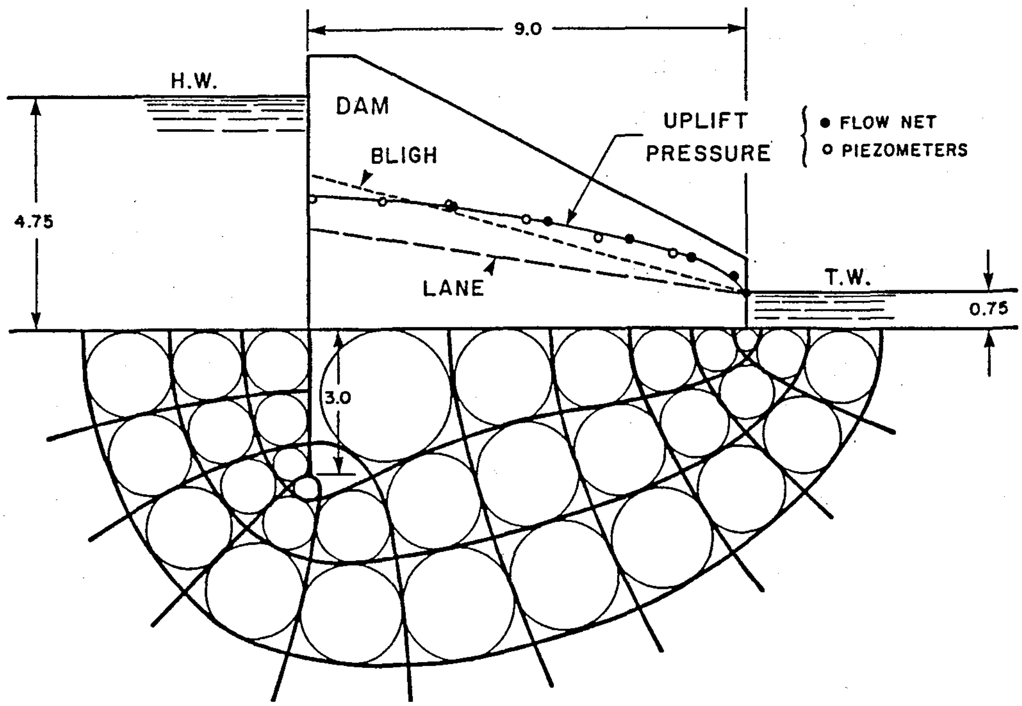


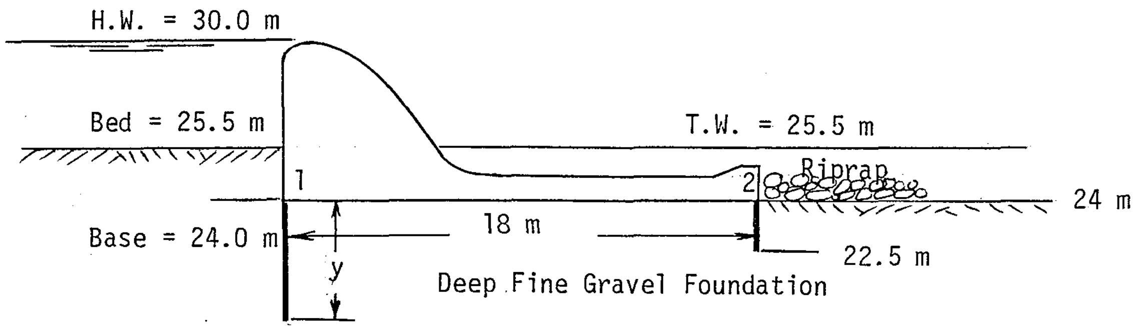
Figure 4. Uplift Pressure Distribution

Also shown on Figure 4 are the two linear approximations of the uplift pressure, calculated according to Bligh (upper dashed line) and Lane (lower dashed line). It is evident that for this case the uplift force calculated using the weighting factors suggested by Lane is on the unsafe side, whereas the Bligh method agrees reasonably well with the actual pressure. While it is true that Lane's method will always give less uplift force than Bligh's, it does not follow that it will necessarily give a lower force than the actual uplift. In fact, it can be shown by flow net analysis that the actual uplift for the dam in Figure 4 would be even less than Lane's value if either an impervious strata was located at a depth of 5 units below the base of the dam, or if the horizontal permeability of the foundation was ten times the vertical. In the latter case it is necessary to use a transformation involving the square root of the ratio of horizontal to vertical permeability before the flow net method can be applied. In either case there would be an increased percentage of the total head dissipated around the vertical cutoff, and a corresponding reduction in the uplift pressure.

Flow net analysis is time consuming and cannot be adapted to account for differences in contact between horizontal and vertical surfaces along the creep path. The utility of the linear approximation methods lies in their simplicity. Cutoff depth should always be determined by Lane's weighted creep-head theory. In the interest of safety, however, uplift should be calculated according to Bligh if there is any doubt about the composition of the foundation. Lane's method may be used where horizontal stratification is known to exist. On important structures, all methods, including the flow net, should probably be investigated.

Example 1:

For the dam shown in the sketch, find the required depth for the upstream cutoff, and compute the uplift force on the base per unit width.



The required L/h value for the foundation by Lane's criterion is 4.0, and since h = 4.5 m, the required weighted creep path length is 4 x 4.5 = 18 m. Hence,

$$1.5 + 2y + (1/3)18 + 2 \times 1.5 = 18$$

or $y = 3.75$ m

The creep path length according to Bligh would then be 30 m, and from [2] the piezometric head at point 1 is

$$h_1 = 4.5\,(30 - 9)/30 = 3.15 \text{ m}$$

Since $h_1 = P_1/\gamma + Z_1$ and $Z_1 = -1.5$ m, then $P_1/\gamma = 4.65$ m.

Similarly at point 2

$$h_2 = 4.5\,(30 - 27)/30 = 0.45 \text{ m}$$

and $P_2/\gamma = 1.95$ m

The uplift force becomes

$$U = 18 \times 9.81\,(4.65 + 1.95)/2 = 582.7 \text{ kN/m}$$

C. HEADGATE AND SLUICEWAY

7. Headgate

The headgate is a gated control structure used to regulate the flow into the diversion channel. It is so named because it is the first gated structure on the canal system. The headgate is located in one of the abutments at the end of the diversion dam. A typical layout is shown in Figure 5, and a section through the headgate is shown on Figure 6.

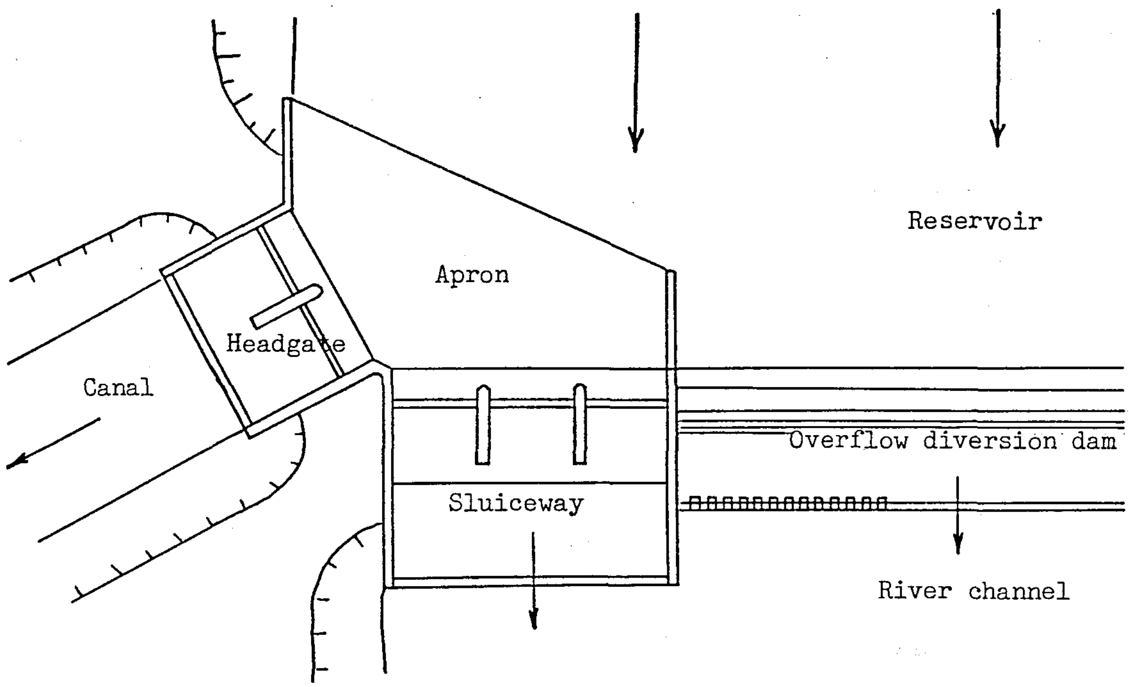


Figure 5. Layout for Diversion Dam, Sluiceway, and Headgate

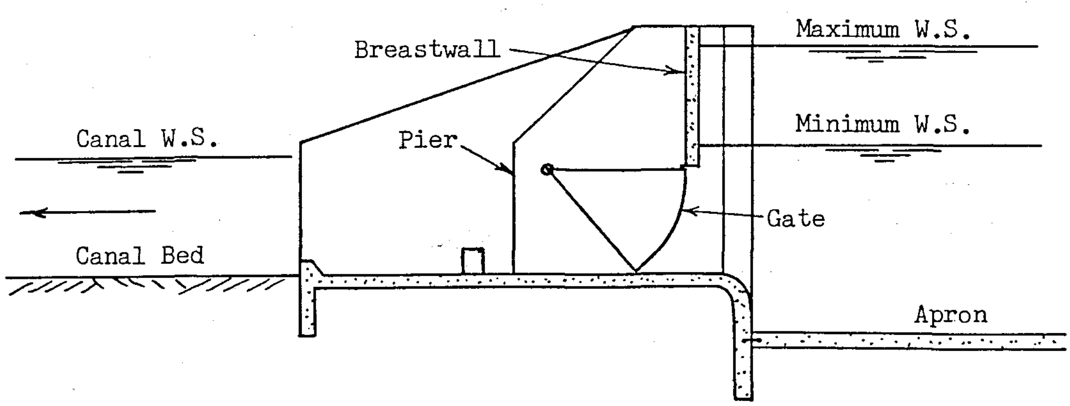


Figure 6. Section Through Headgate

Radial or slide gates may be used. A large gate area is used in order to keep the velocity and head loss to a minimum. Velocities through the gate opening of 1.5 to 2 m/s are common at design discharge and minimum pool level. The head loss is then kept to less than 0.3 m. Higher velocities would require a higher weir crest for the entire diversion dam. The required size of the gate opening may be calculated from the submerged orifice equation

[3] $$Q = CA\sqrt{2gh}$$

in which Q is the design discharge, A is the area of the gate opening, h is the differential head across the gate opening, and C is the coefficient for the gate. It is conservative to take h as the difference in level between the reservoir and the canal water surface because the actual effective differential head across the gate opening will be marginally greater. The water level immediately downstream from the gate will be slightly below the canal water surface due to a small recovery in the headgate basin. The coefficient C will vary from a low of 0.60 for a square lip breastwall, as shown in Figure 6, to a high of 0.97 for a streamlined gate opening, as shown in Figure 7. The higher coefficient allows the use of a smaller gate, but the breastwall must be constructed with a curved bottom lip.

Energy dissipation in the headgate at the design discharge is not a problem when the reservoir is at the minimum level because of the low velocities through the gate. However, at less than design discharge the canal water surface will be lower, and a significant differential head may exist across the headgate structure. The gates must be partially closed to control the discharge, and the extra head must be dissipated. It is to be expected that the worst condition for design will occur when the reservoir is at maximum pool elevation and the headgate is operated at design discharge, since both the head and discharge are a maximum. The headgate basin must be designed to accommodate a hydraulic jump, or submerged jump, for this condition. Floor baffles and an end sill may be used. The design procedure is similar to that used for the breastwall check structure, as discussed in Section 20, Chapter 9. It is interesting to note that the gates are sized for the minimum reservoir water surface elevation, whereas the basin must be sized for the maximum reservoir water surface elevation.

8. Sluiceway

In an overflow type diversion dam a separate sluiceway is required. The functions of a sluiceway are to maintain a channel to the headgate during flood periods, and to discharge accumulated silt deposits from in front of the headgate during normal operating periods. During flood periods bed load could deposit at the headgate entrance if there was no current and a dead area occurred at that location. By operating the sluiceway at full capacity during floods, a current is maintained and the channel is kept clear. During normal operating periods when most of the flow is being diverted, a dune may build up progressively toward the headgate and eventually sediment could pass into the canal. This can be prevented if the sluiceway is opened periodically to discharge the accumulated deposit from in front of the headgate. Alternatively the sluiceway may be set to operate continuously while a diversion is being made.

In order to accomplish the foregoing objectives the design must meet the following requirements:

(1) The sluiceway must be located adjacent to the headgate;

(2) The axes of the headgate and sluiceway should be at an angle, usually between 45° and 90°. This is desirable in order that the sluiceway flow will be drawn across the headgate entrance.

(3) The sluiceway approach apron and gates should be silled about 1 m below the headgate sill. This will allow some dead storage area for silt below the headgate sill.

(4) The sluiceway discharge capacity should be about twice the headgate capacity. This is necessary to create high sluicing velocities on the approach apron.

A very substantial stilling basin must be designed for a sluiceway because it may have to operate when the discharge over the diversion dam is low. The tailwater depth in the river channel will be correspondingly low, much lower than at flood stage, yet the sluiceway may have to be operated at or near full capacity in order to effectively flush out silt deposits. The slab elevation for the hydraulic jump stilling basin will have to be much lower than the basin slab for the diversion dam proper. A section view of the sluiceway for Imperial Dam is shown in Figure 7. It is noteworthy that the basin slab is 3.05 m lower than for the diversion dam itself (Figure 1).

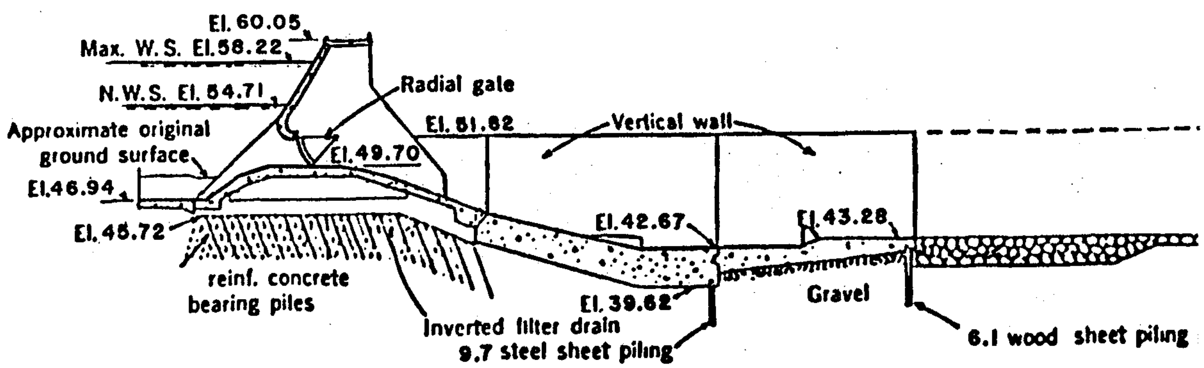


Figure 7. Section of Imperial Dam Sluiceway

Figures 8 and 9 show the headgate and sluiceway for the Belly River Diversion Dam. Figure 8 shows the sluiceway (on the left) and headgate from the upstream side. Figure 9 shows headgate, from the downstream side, discharging into the canal.

D. FISHWAY

9. Importance

Fishways are structures designed to facilitate the passage of fish upstream or downstream over natural obstacles (waterfalls) or man-made barriers (dams). Certain species of fish, called migratory fish, travel downstream as fingerlings, and when grown return to the place of their birth to spawn. The fish migrate downstream to reach a larger body of water where the food supply is more plentiful. The most notable example of this type in Canada is migration of the west coast salmon. Some other important migratory fish are pickerel, grayling, char and trout.

Figure 8. Belly River Sluiceway and Headgate, Alberta

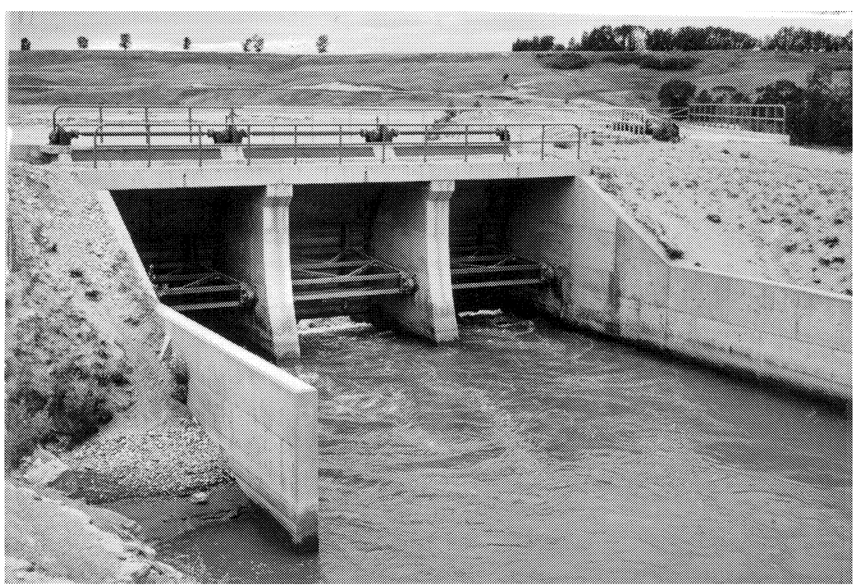


Figure 9. Belly River Headgate

In areas where fish have a commercial value or are important for sport fishing, provision must be made for fish migration whenever a dam is built. The downstream migration is usually not a difficult problem. Fingerlings have been passed over spillways, through sluices, and even through hydraulic turbines, with a surprisingly low mortality rate. On high dams fish may be taken up by automatic elevators or tank trucks. A fish ladder is not usually feasible for heights greater than 15 or 20 m since the fish get discouraged from their apparent lack of progress. On diversion dams the fish ladder is in common use. In prairie areas where the fish run is small, it is usually possible to incorporate a fish ladder into the diversion dam at a modest cost. In the Pacific Northwest where salmon runs are heavy, ladders may be relatively costly structures.

10. Principles of Design

Fish ladders are usually planned according to the following principles:

(1) The layout should be designed so that there is a significant velocity in the area approaching the ladder. In their upstream migration the fish use the current as a direction guide, so if the entrance to the ladder is located in a dead area the fish may not find it. The velocity should be about 1 m/s. Creation of such a velocity condition is called "attraction water', as it is intended to attract the fish.

(2) The exit for the fish from the ladder should be in a quiet area well away from the overflow section or sluiceway, otherwise the fish may be carried back downstream.

(3) Maximum velocities in the fish ladder should not exceed the burst speed (or darting speed) for the fish. This is the speed that the fish can swim for a few seconds, and is in the order of 8 to 12 body lengths per second. A typical velocity is 2.5 m/s. At this velocity the drop in elevation between pools is limited to about 0.3 m, since this drop is converted to velocity head between pools. On short ladders higher drops have sometimes been used, up to 0.6 m. It has been observed that 2.5 m/s is a velocity that the fish can comfortably swim against for a short duration. Higher velocities tend to discourage some fish from using the fishway, whereas lower velocities would increase the length and cost of the ladder. This velocity is the basis for determining the size of openings between pools.

(4) Average velocities in the fish ladder should be about 0.30 to 0.45 m/s. This velocity is the basis for selecting the fish ladder discharge from $Q = VA$, in which A is the cross sectional area of the ladder. This relatively low average velocity has been found necessary to allow rest stops for the fish while going up the ladder. Fish do not feed during upstream migration, but may rest about 4 hours each day.

(5) The volume of the fishway should provide from 0.06 to 0.12 m^3 of water per fish, depending on the size of the fish. It has been observed that, given sufficient space to manoeuver, the fish will not injure themselves even on the sharpest corners or baffles.

Given data on the peak migration rate and the rate of climb, a suitable ladder can be designed with no other information except the foregoing five principles. Fish are overly cautious when proceeding in unfamiliar channels and the average rate of climb is surprisingly low, often only 2.5 to 3.5 m/h. Migration rates are estimated from biological tagging data. As an extreme example, the peak run of sockeye salmon on the Fraser River has been estimated at 20,000 fish per hour.

11. Types of Fish Ladder

The fish ladder consists of a rectangular chute with a gentle slope of 1 vertical to 8 or 12 horizontal. The chute is divided into a number of pools by cross walls. The opening through the walls may be an orifice, weir, or slot. The alignment may include curves or bends. Because of the flat slopes which must be used, it may be necessary to use a 180° bend (i.e. switchback) in order to accommodate the total length required.

Most early fish ladders were the orifice or weir type as shown in Figure 10 (a) and (b). The positions of the openings could be alternated to permit better energy dissipation and prevent a continuous shooting jet. The size of the openings was calculated from conventional weir or submerged orifice formulae. Use of the orifice type ladder with fixed orifice area has been largely discontinued due to the need to closely regulate the discharge. Weir type ladders are still used. The weir on many of the Columbia River fish ladders in Washington spans the full width of the pool, as shown in Figure 11.

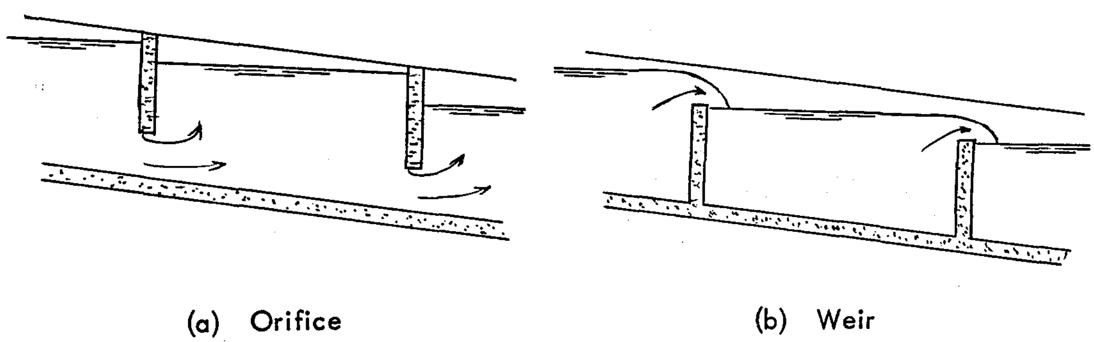


Figure 10. Orifice and Weir Fish Ladders

A recent and popular design for fish ladders is the vertically slotted baffle, as shown in Figures 12 and 13. The sloping chute is divided into a number of rectangular pools with a length somewhat greater than the width, with each pool separated by a vertical wall containing a vertical slot near one end. The slot is designed to cause jet flow diagonally across the pool. The advantages of the vertical slot ladder are:

(1) Ascent is possible at any depth the fish chooses. This depth selected may vary with the time of day, turbidity of the water, or lighting conditions.

(2) There is no jumping involved, as in the weir type ladder.

(3) The flow pattern is essentially constant at all depths, and it is the same for every pool. The plan view flow pattern is shown in Figure 12.

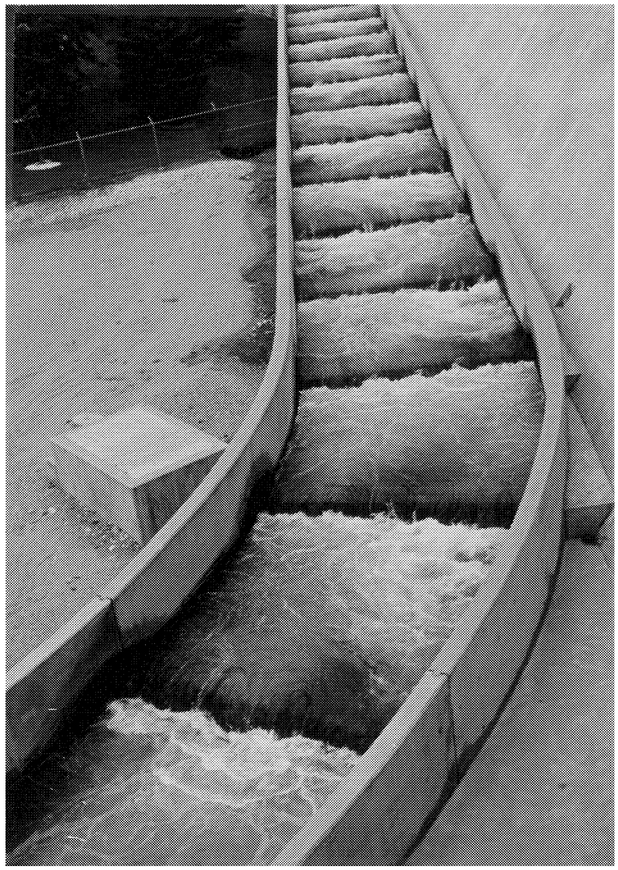


Figure 11. Weir Type Fish Ladder, Wells Dam, Columbia River

(4) The vertical slot design is adapted to use where the headwater and tailwater levels may vary over wide limits, yet regulation of the discharge is not necessary.

(5) Weir type ladders may trap sediment if there is no sediment control. Sediment discharge is easily accommodated through the vertical slot.

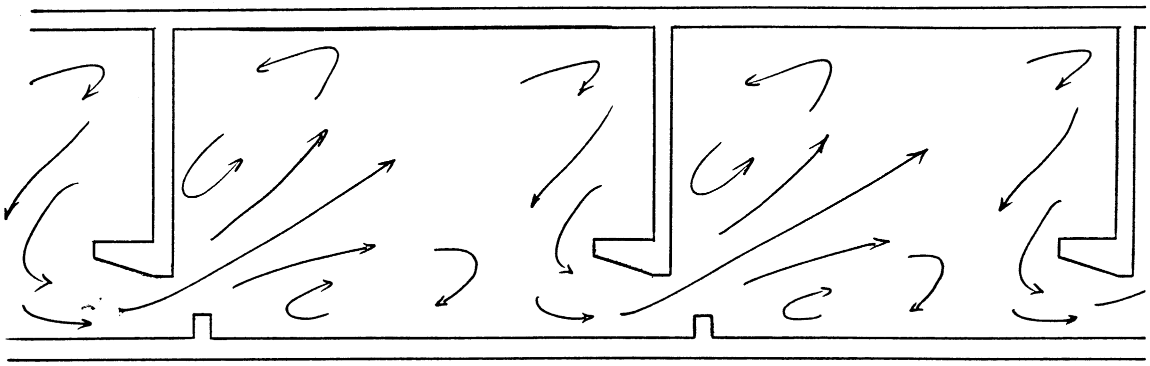


Figure 12. Plan View Flow Pattern for Vertical Slot Fish Ladder

Figure 13. Hydraulic Model of Vertical Slot Fish Ladder

Minimum slot widths are usually 0.2 m for 1 kg fish or 0.3 m for 2 kg fish. The slot widths and discharge are related approximately by

[4] $$Q = 0.7\, D_2\, W_s \sqrt{2g\,(D_1 - D_2)}$$

in which W_s is the slot width, and D_1 and D_2 are the depths immediately upstream and downstream from the baffle wall.

It should be noted that height of climb over a diversion dam is less than the height of the dam itself. The fish need climb only from tailwater to headwater, and as shown in Figure 3, this may be substantially less than the height of the dam. The stream hydrographs of past record must be examined for the period corresponding to the fish run in order to establish the most probable upper and lower limits of the water levels.

Another variation in fish ladder design is the Denil type. The Denil fish ladder consists of a straight inclined chute with closely spaced baffle walls on both sides and across the bottom. The baffles are normally set at an angle to the vertical, with the top upstream from the bottom. A relatively large portion of channel between the sidewall baffles is available for the main flow down the centre, and through which the fish pass on the upstream migration. The baffles create considerable turbulence and the associated energy dissipation limits the velocity of the main flow to about one-fifth of what it would be if the baffles were not present. The fish may rest in the recesses behind the baffles when necessary. The Denil system is adaptable to any chute cross-section, and to a wide range of headwater and tailwater elevations.

Example 2:

Calculate the hydraulic dimensions for a vertically slotted fish ladder for a 4 m differential head across a diversion dam, given the peak migration rate as 160 fish/h, rate of climb 3 m/h, ladder volume of 0.1 m^3/fish and average pool depth of 0.9 m.

Average time in ladder for each fish = 4/3 = 1.33 h
Number of fish in ladder at one time = 160 x 1.33 = 213
Volume of ladder = 213 x 0.1 = 21.3 m^3
Number of pools (at approx. 0.3 m drop) = 4/0.3 = 13
Volume per pool = 21.3/13 = 1.64 m^3
Horizontal area of pool = 1.64/0.9 = 1.82 m^2
Pool dimensions in plan = 1.2 m wide by 1.5 long
Total length of ladder = 13 x 1.5 = 19.5 m
Slope of ladder = 4/19.5 = 0.205
Vertical area of pool = 0.9 x 1.2 = 1.08 m^2
Ladder discharge (at 0.3 m/s average velocity) = 1.08 x 0.3
= 0.324 m^3/s
Pool depth upstream from wall $D_1 = 0.9 + 0.3/2 = 1.05$ m
Pool depth downstream from wall $D_2 = 0.9 - 0.3/2 = 0.75$ m

$$\begin{aligned} \text{Slot width } W_s &= Q/(0.7\, D_2 \sqrt{2g\,(D_1 - D_2)}) \\ &= 0.324/0.7 \times 0.75 \sqrt{2g\,(1.05 - 0.75)} \\ &= 0.254 \text{ m} \end{aligned}$$

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PROBLEMS

1. A diversion dam built on a deep fine gravel foundation has a base width of 9 m. There is no drain under the base. The headwater will be 7.6 m above the base when the tailwater is 3 m. What depth should be used for a single cutoff upstream? What will be the total uplift force on the base by Bligh's criterion?

2. Prove that for most effective use of a series of cutoffs, the spacing between the cutoffs should equal or exceed 6/5 of the cutoff depth.

3. Calculate the weighted-creep-head ratio for Imperial Dam, shown in Figure 1, when the reservoir is at maximum water surface elevation. Assume that the top of the vertical drain pipe connected to the inverted filter discharges to atmospheric pressure at elevation 48.8 m.

4. A double cutoff consisting of 2 rows of sheet piling 4.5 m deep and 3 m apart is used below a dam with a 18 m base, of which the last 6 m is underlain by a free draining select gravel layer. The first row of piling is at the upstream toe of the dam. The headwater is 9 m and there is no tailwater. Calculate the pressure head at the bottom tip of the second row of piling by Lane's criterion.

5. (a) Determine the required height of the gate opening for the headgate shown in Figure 6, given the canal bed elevation 0.0, canal water surface 2.9 m, minimum reservoir water surface 3.0 m, and design discharge 2.4 $m^3/s/m$. Use a $C = 0.6$ for the gate opening.

   (b) If the maximum reservoir water surface during flood flow over the diversion weir is 6 m, prove that the tailwater depth (2.9 m) is sufficient to keep the hydraulic jump in the basin.

6. Why is it advisable at a headworks to have the discharge capacity of the sluiceway greater than that of the headgate?

7. (a) What approximate area of orifice would be required for an orifice type fish ladder with individual pools 3 m long and 1.5 m wide, with an average depth of 1 m? (Use $C = 0.6$ for the orifice).

   (b) If the fish climb an average of 3 m/h, and a space of 0.09 m is allowed for each fish, what migration rate could be handled by the ladder?

CHAPTER VI

DROP STRUCTURES

A. INTRODUCTION

1. Purpose of Drop Structures

A common method of overland water conveyance is by a canal or ditch. In order that the system be relatively maintenance free, the canals should be designed as stable sections under the imposed conditions. This requires that the maximum velocity, and hence maximum slope, be limited. If local ground slopes along the proposed route of the canal are steeper than permissible canal slopes, the excess grade must be accounted for by a steeper reach or an abrupt drop in the bed level. Various methods of achieving this drop in elevation include reinforced concrete free flow drop structures, pipe drop structures, riprap lined channels and grassed channels.

If the discharge is small and the flow is intermittent, as in the case of storm runoff in a drainage ditch, a grassed channel may be used. Obviously, however, a grassed channel could not be used for a sustained flow because, among other things, the grass would have no opportunity to grow. A stone paved channel can be used for a continuous discharge, but again the discharge must be relatively small and the drop height reasonable because of the prohibitive size and amount of stone that would otherwise be required. Free flow drop structures, such as the reinforced concrete vertical and chute drop, have practically no upper limit as far as drop height and discharge are concerned. Even for smaller flows and drop heights a concrete or timber structure may be more suitable than a pipe, or cheaper than stone paving. Overflow drop structures have been used for flows from 1 to 150 m^3/s, and for drops from 0.5 to 15 m. Pipe drop structures are often used where a crossing is required, because the pipe can be buried and the fill used as a roadway, thereby eliminating the need for a bridge.

Regardless of the method used, the basic objective is to dissipate the excess energy of flow associated with the loss of elevation head, and the structures may all be classified as energy dissipating devices. For the grassed and riprapped channels this energy is dissipated in excess channel friction. For the other structures the energy is dissipated in excess turbulence associated with the use of baffles or stilling basins, or both.

In this chapter, the free flow drop structures and pipe drop structures are discussed. The grassed channel and riprapped channel are covered in Chapter 8 dealing with flexible channel linings for erosion control.

2. Permissible Canal Slopes

A channel carrying a constant discharge in erodible material may be found to scour either the bed or the banks, changing depth, width, and slope, until ultimately a state of balance is attained, at which time the channel is said to be in regime. A regime channel is obviously more stable than a non-regime channel. The latter, in attempting to attain regime, may be very troublesome and incur costly maintenance. The natural stable channel is relatively wide and shallow when compared to the most hydraulically efficient cross section. Further, the required width to depth ratio for a stable channel is not a constant, but depends on the soil type and the discharge. It is a well established fact that the width to depth ratio is greater for large channels than small ones, and greater for channels in sandy soil than clayey soil.

Extensive observations, notably in India and the United States, have shown that dimensions for a stable channel can be related to both the design discharge and soil type. The effect of these variables is indicated in the following relationships, which may be used to estimate the approximate channel dimensions:

(a) For a channel excavated through a clayey soil:

[1] $$W = 3.0\,Q^{1/2}$$

[2] $$D = 0.75\,Q^{1/3}$$

(b) For a channel excavated through a sandy soil:

[3] $$W = 4.0\,Q^{1/2}$$

[4] $$D = 0.60\,Q^{1/3}$$

In [1] to [4], W is the average water width of the channel, D is the flow depth and Q is the discharge. The relationship between the bottom width b and average width is

[5] $$b = W - ZD$$

in which Z is the side slope of the canal, expressed as horizontal to vertical. The wetted perimeter is

[6] $$P = b + 2D\sqrt{1 + Z^2}$$

and the hydraulic radius is

[7] $$R = DW/P$$

Once the channel elements have been determined the required slope can be found by applying Manning's equation, written in terms of the slope as

[8] $$S = V^2n^2/R^{4/3}$$

Typical values of Manning's n for earth channels are given in Table 1. Channel slopes are usually based on the normal value, but consideration may have to be given to the possible flow depths which would result at n values slightly greater or slightly less than normal. At higher n values the canal will flow deeper, encroaching on freeboard. On the other hand, canal structures may have to be set to accommodate the lower flow depths which would occur at lower n values.

As shown in Example 1, permissible canal slopes are surprisingly flat, often much flatter than prevailing ground slopes. Some large mainline canals have a drop of less than

1 in 10,000 so it is not surprising that some method of accounting for excess grade is required.

TABLE 1
VALUES OF MANNING'S n FOR EXCAVATED EARTH CHANNELS

	Minimum	Normal	Maximum
Earth, straight and uniform			
New, clean	0.016	0.018	0.020
Weathered, clean	.018	.022	.025
With short grass, few weeds	.022	.027	.033
Earth, winding and sluggish			
Clean	.023	.025	.030
Grass, some weeds	.025	.030	.033
Dense weeds or aquatic plants	.030	.035	.040
Gravel, straight and uniform			
Clean	.022	.025	.030
With weedy banks	.024	.028	.033
Gravel, winding and sluggish			
Clean	.025	.030	.035
With weedy banks	.030	.035	.040

Example 1:

A ditch to convey 1.4 m^3/s through a 7 km reach of clay type soil must be designed. The natural ground slope in the direction of flow is 0.001. Approximately how many drop structures will be required in this reach?

From Equations 1 and 2

$$W = 3.0 \times 1.4^{1/2} = 3.55 \text{ m}$$

$$D = 0.75 \times 1.4^{1/3} = 0.84 \text{ m}$$

The velocity $V = Q/DW = 1.4/(0.84 \times 3.55) = 0.47$ m/s

Using 1 1/2:1 side slopes, the bed width $b = 3.55 - 1.5 \times 0.84 = 2.29$ m, and the hydraulic radius $R = 0.56$ m. With Manning's $n = 0.025$, the slope is

$$S = 0.47^2 \times 0.025^2/0.56^{4/3} = 0.00030$$

In 7 km the ground level will drop 7 m. The permissible drop in canal bed level, at a slope of 0.0003, will be 0.0003 x 7000 = 2.1 m. Therefore, 7 - 2.1 = 4.9 m of drop must be taken up by drop structures. In order to minimize the amount of excavation required to construct the canal, drop heights would have to be limited to a value not greatly exceeding the canal flow depth, probably in the range of 1 m. Therefore, 5 drops at 0.98 m each would be required.

It is interesting to note that the shape of the cross section for a most efficient hydraulic section (i.e., least wetted perimeter for the given area and side slope) would have b = 0.70 m and D = 1.20 m. Such a narrow deep section would be subjected to severe bank erosion. Use of most efficient hydraulic sections is usually restricted to channels lined with concrete, wood, or metal. In any case the most efficient hydraulic section is not adapted to situations where excess grade must be accommodated. Being more efficient, the canal would have to have an even flatter slope (0.00027 for the above example), and more of the drop would have to be taken up at the drop structures.

B. CHUTE DROP STRUCTURE

3. Development

Most concrete drop structures built before 1940 were of the vertical type, in which the discharge would pass over a vertical weir and fall into a "stilling pool" at the base of the weir. Energy dissipation was achieved by turbulent mixing in the pool in much the same manner as it occurs at the base of a waterfall. According to early design criteria, the floor slab for the stilling pool should be depressed below the downstream canal bed by as much as one quarter of the flow depth. Continued study of drop structures has led to a better understanding of the principles involved, and improvements in the design and performance have resulted. However, the maximum drop height obtainable with the vertical drop has been limited by the difficult stability problem and expensive construction associated with the high vertical retaining walls which would be required for the weir and abutments. The chute drop structure was evolved for the higher drops and is now in common use for all heights. Essentially, the vertical drop structure concentrates the drop in elevation at one section, whereas the chute drop structure spreads the drop over a short reach of channel. Although more floor slab is required, this is more than compensated for by the reduction in wall height. Typical chute drop structures are shown in Figures 1 and 2. A definition sketch, showing various components, is given in Figure 3.

The components of the chute drop structure are basically the same as the chute spillway, as covered in Chapter 2. In each, there must be some means of controlling the flow at the top and dissipating the energy at the bottom. A few of the points of difference are discussed in the following sections.

4. Width of Structure

The exit width of the drop structure should be great enough to permit discharge into the canal near regime conditions. In this regard, the width may be estimated from

[9] $$W = 2.3\sqrt{Q}$$

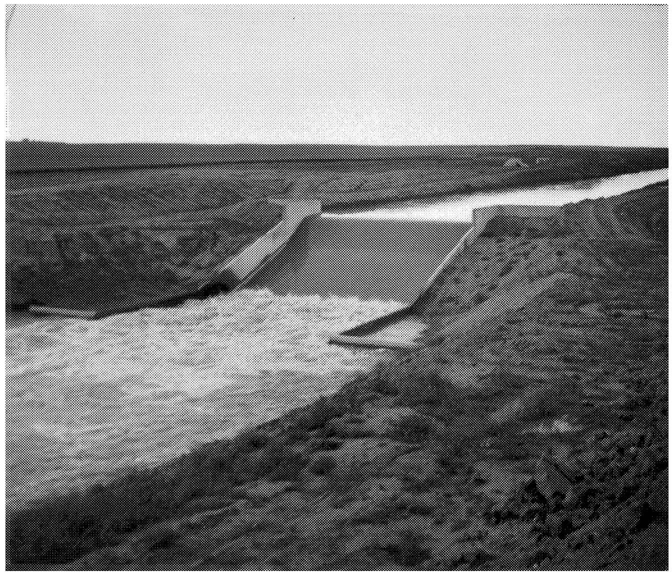

Figure 1. Concrete Chute Drop Structure in Operation

Figure 2. Small Timber and Sheet Metal Chute Drop Structure

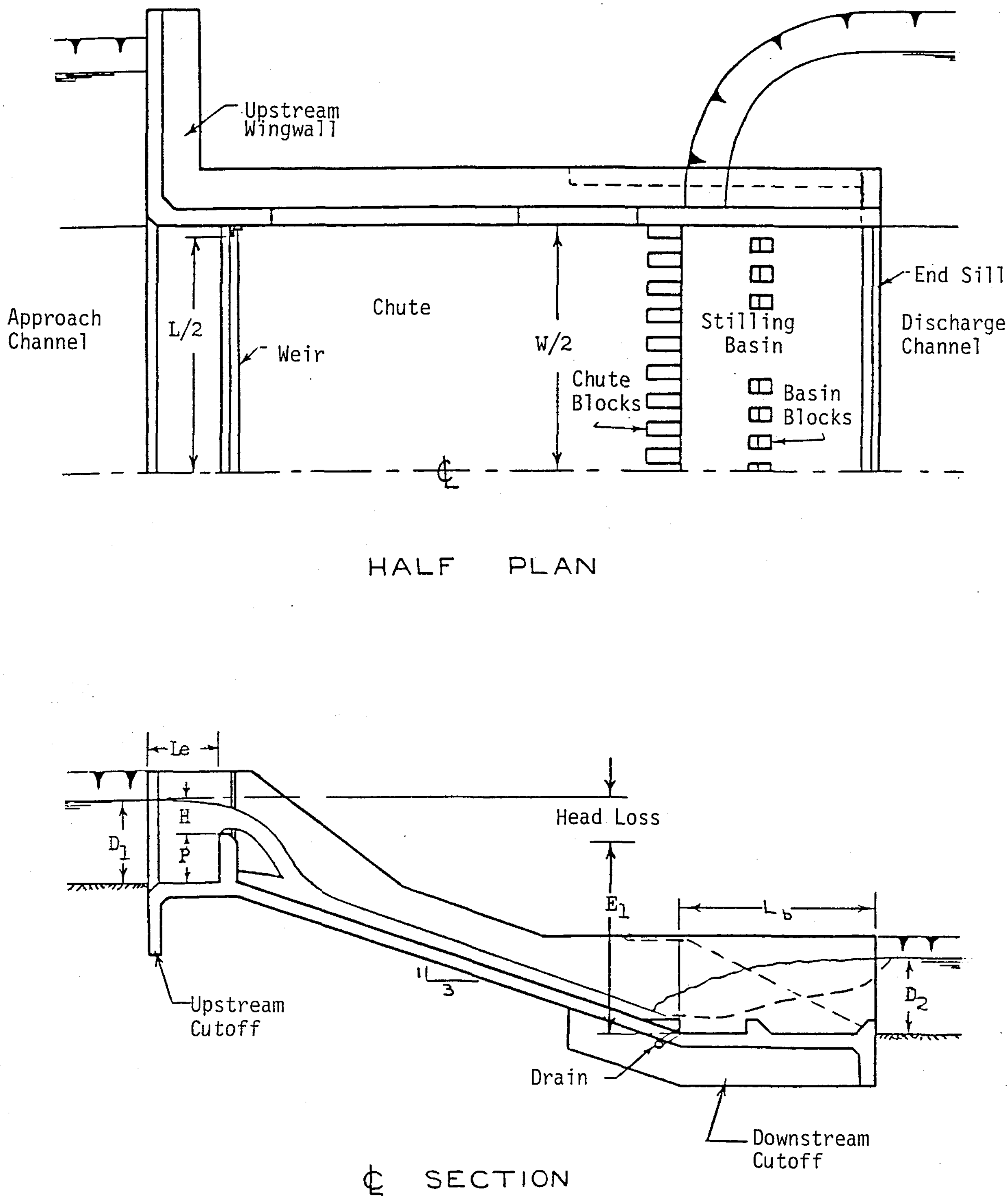

Figure 3. Definition Sketch for a Chute Drop Structure

As a guide for selecting spillway stilling basin widths, a value of 1.8 $\sqrt{Q}$ was suggested. Since canal structures will operate at or near full capacity a relatively large percentage of the time, the extra width is beneficial. The advantages of a wide exit are reduced basin wall height (due to smaller jump), reduced stilling basin length, reduced exit velocity, reduced dynamic forces per unit width of basin, and reduced potential uplift due to the jump in the basin.

Although a wide basin is preferable, a narrower one may be less expensive, particularly if the structure is a combined drop and check with piers and gates. However, 2.0 $\sqrt{Q}$ should be considered as the lower limit for the drop width. This width will be much less than the average canal width given by [1] and [3].

In many spillway structures which have a long chute, it is expedient to make the chute narrower than the crest section or the stilling basin. This type of design requires a converging and diverging transition in the chute. By comparison, canal drop structures are relatively short, and the complications of converging and diverging the flow more than offset any advantage gained by restricting the chute width. It is recommended that the structure be made constant width throughout (i.e., parallel walls). This has the advantage of simplified hydraulic and structural design, and insures good hydraulic performance for the full range of the flow.

5. <u>Upstream Cutoff and Inlet</u>

The depth of the upstream cutoff can be calculated by applying Lane's weighted creep-head method. Actually, the method does not apply exactly because it was developed for a wide structure where the seepage flow is essentially two-dimensional. For a narrow structure the seepage flow pattern is three-dimensional, and the point of egress to which the creep should be computed is less definite. However, a satisfactory design will result if the weighted creep path is computed up to the first lateral drain under the slab near the top of the chute. It is assumed that this drain will be at atmospheric pressure.

An adequate cutoff is important to reduce the danger of piping, reduce the seepage load on the drainage system, and to reduce the danger of uplift pressures under the sloping chute. It is noteworthy that a good cutoff can partially compensate for a poorly installed drainage system. The upstream wingwall is really a lateral extension of the cutoff. It must provide the same degree of cutoff protection around the end of the structure as is required underneath.

It is necessary to have a short approach length between the upstream wingwalls and the weir. This is required to keep the high velocities at the approach to the weir within the confines of the structure and to establish essentially two-dimensional flow at the weir. An inlet length of twice the head on the weir is ample, provided this is enough to satisfy the sliding stability requirements for the control section.

The approach velocities are low enough so that warped or curved inlet walls are not required for this type of structure. However, to avoid right-angled corners projecting into the flow, a 45 degree chamfer should be used at the intersection of the wingwall with the sidewall, and at the top of the vertical cutoff. This chamber may have a side dimension of 0.1 D_1 in length.

6. Weir

The weir is that part of the drop structure which controls the depth D_1 in the upstream canal. This depth is predetermined so as to give acceptable velocities in the canal at full capacity flow. The necessary weir height P is given by

[10] $$P = D_1 + V_1^2/2g - H$$

in which V_1 is the flow velocity in the canal and H is the total head on the weir.

The weir cross section could be broad crested, trapezoidal, ogee, or vertical. For drop structures of the type considered the vertical weir is the most economical, requiring the least quantity of materials and labour. Unlike the case for spillways, a high coefficient weir is of no value for a drop structure. The higher the coefficient, the greater is the required weir height, increasing cost.

The weir nappe for a vertical weir must be ventilated, otherwise a clinging nappe will result. The clinging nappe is sustained by subatmospheric pressure which develops under the nappe in the absence of an air supply. Although the pressure is ordinarily well above cavitation pressure, it is still low enough to be undesirable. Model tests show that the non-ventilated vertical weir can sometimes result in unsteady flow and that the discharge coefficient has a wide range of variation. In any case, it is unsatisfactory to design for a clinging nappe since it may accidentally become ventilated by floating debris which may catch on the weir. The resulting drop in the weir coefficient would result in an increase in the upstream canal depth.

The actual air demand of the overfalling nappe is not large, but if air is not supplied subatmospheric pressures would eventually develop no matter how small the demand. The nappe can be ventilated with a vent pipe embedded in the abutment wall, with the top open to atmosphere and the bottom below the weir crest on the downstream side. However, the simplest and least costly method for ventilation is with an end contraction on the abutment wall at each end of the weir. This causes the nappe to spring clear of the wall and maintain an air space from atmosphere to the area under the nappe. Air supply will be adequate if the end contraction projects into the flow by an amount y, such that

[11] $$y = \sqrt{0.0008\ WH}$$

with Y, W and H all in m.

The weir coefficient for a ventilated vertical weir remains constant at 1.837 despite changes in weir head because the nappe is free to change trajectory as the head changes. This is to be contrasted with other weir types where the nappe adheres to the face of the weir, the nappe shape remains fixed, and the coefficient increases as the head increases. The vertical weir with nappe vents will have a net crest length $L = W - 2_y$, and hence the head and discharge are related by

[12] $$Q = 1.837\ (W - 2y)\ H^{3/2}$$

The vertical weir will require some thickness for its structural strength, yet in order to have a reliable and constant weir coefficient, it is desirable to approximate the condition of a sharp crested weir. This can be done if an upper limit is placed on the width x of the flat top part of the weir so as to prevent it from interfering with the nappe. This limit must be related to the design head. It can be shown from the data on nappe coordinates in Table 1, Chapter 2, that if x = 0.13H, then the flat top of the weir will not affect the nappe unless the discharge is less than 10% of design discharge. The stem of the vertical weir will usually be thicker than this, in which case a 45 degree chamber should be used to make up the extra thickness. The details of the weir and end contraction are shown on Figure 4.

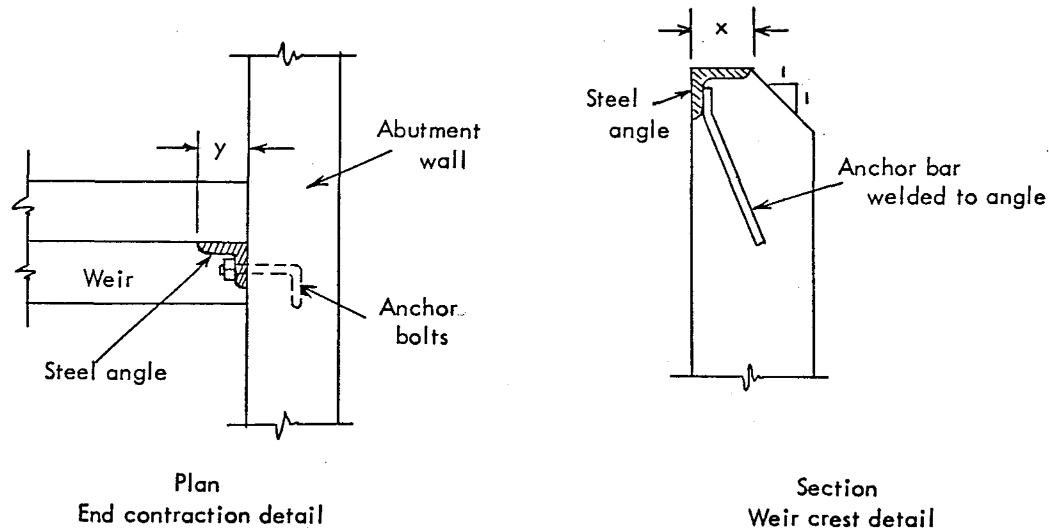

Figure 4 Details of Weir Crest and End Contraction

7. <u>Chute</u>

The chute is that portion of the structure which carries the flow from the weir to the stilling basin. The design considered here is based on the use of a chute slope of 0.333 (1V:3H). This is usually flat enough to be stable, unless the drop height is unusually high, and can be easily excavated and concreted.

In order to design the stilling basin, the energy of flow at the base of the sloping chute must be evaluated. At the top of the chute there is some energy loss due to the turbulence in the pool which forms under the nappe. This pool is agitated by the overfalling jet. Model tests indicate the loss is between 0.22 H and 0.24 H. It is therefore conservative to assume the energy loss as 0.2 H. Manning's n for the concrete chute should be about 0.012. A family of energy loss curves has been calculated for q = 0.25 $m^3/s/m$ to q = 4.50 $m^3/s/m$, assuming the nappe loss at 0.2 H, Manning's n = 0.012, and with the hydraulic radius equal to the depth. These curves are shown on Figure 5. The designer can readily pick off the available energy at the toe of chute (for a given drop and discharge) and proceed directly to the calculation of v_1 and d_1 by simultaneous solution of the continuity and energy equations

[13] $$q = v_1 d_1$$

[14] $$E_1 = d_1 + v_1^2/2g$$

Because of the nature of the assumptions, the actual energy available may be slightly less than the computed value E_1 given on Figure 5. This will not significantly affect the design as the error, if any, will be small and on the safe side. If slopes flatter than 0.333 are used for the chute the losses will be greater and the available specific energy will be reduced.

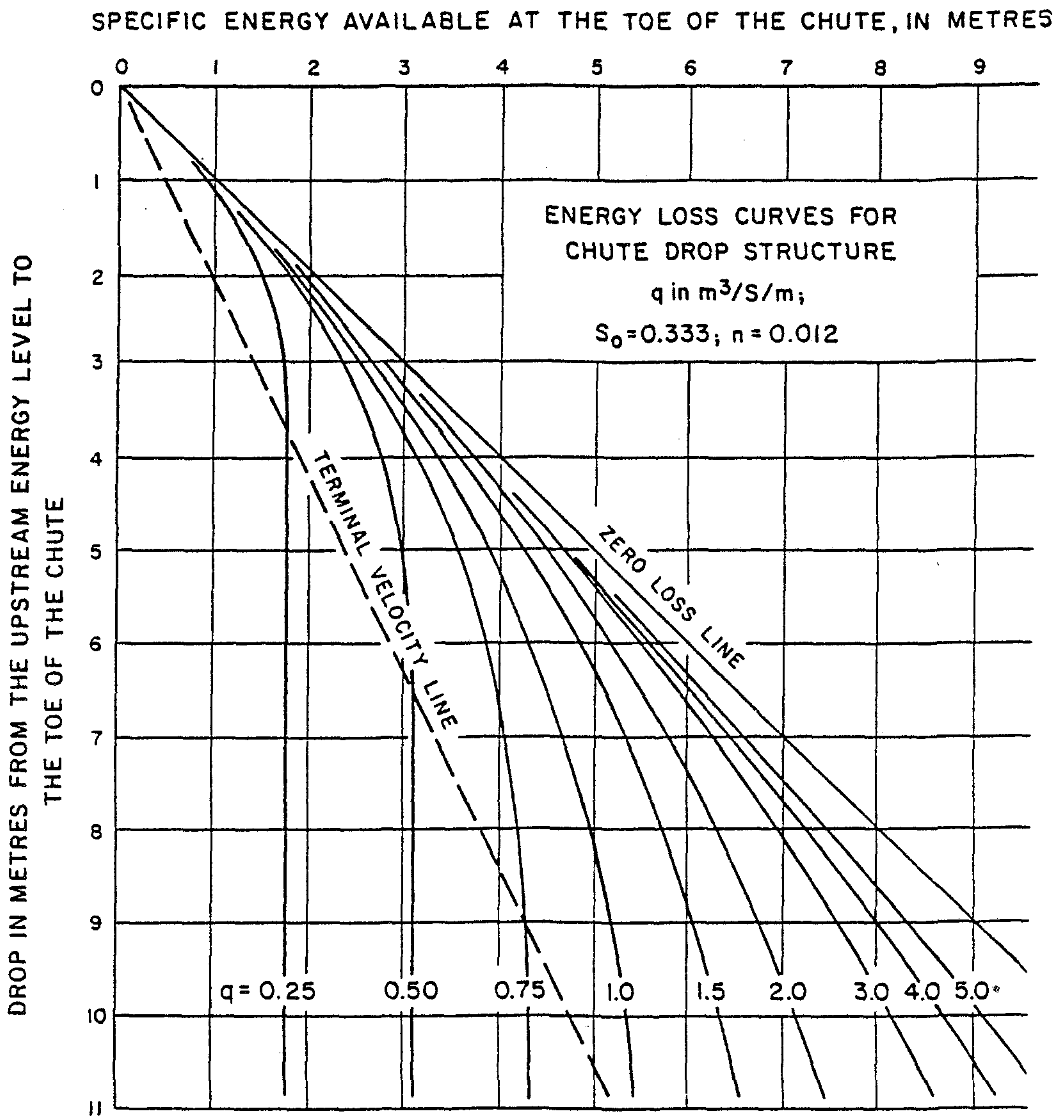

Figure 5. Energy Loss Curves for Chute Drop Structure

8. Stilling Basin

In the stilling basin the excess energy of flow resulting from the drop is dissipated by the hydraulic jump.

The necessary length of the stilling basin, when equipped with blocks and sill, is related to the height of the jump by

[15] $$L_b = 3d_2$$

It is not considered necessary to totally eliminate the erosion potential of the discharging flow. If the design is such that the erosion itself is stable and does not endanger the structure, the design may be considered as satisfactory. If it is desirable to completely eliminate erosion below the structure, this can be done more cheaply with riprap than with greater length of structure.

The proper elevation setting of the stilling basin floor is of greater importance than the length of the basin. The floor must be set d_2 or D_2 below the downstream water level, whichever is greater, to insure the formation of a hydraulic jump and avoid a floor setting above the canal bed. The value d_2 is the computed jump depth and D_2 is the natural downstream canal depth.

On a line of closely spaced drops the weir on one drop may control the tailwater on the preceding one. The water depth in the canal can be accurately calculated, and this figure should be used to set the stilling basin floor. If the canal controls the tailwater, then the normal canal depth must be evaluated. The canal systems are often laid out on the basis of $n = 0.025$. This is done primarily to insure that the canal will have adequate capacity in the event of weed growth or silting. However, in new or cleaned out condition, the n value can be as low as 0.020. From the point of view of basin design, the value of $n = 0.020$ is the safe figure to use for tailwater evaluation.

It is evident that the suggested method of evaluating energy available and tailwater depth will almost invariably result in a basin setting which will give more than enough depth required for jump formation. This excess tailwater in no way impairs the efficiency of the stilling basin, since the jump starts sooner (on the sloping chute) and the exit area of flow is larger (due to the increased depth).

Details of the floor baffles, wall height, downstream cutoff, and pressure relief system are the same for the chute drop structure as for the chute spillway (Chapter 2, Section 27).

C. VERTICAL DROP STRUCTURES

9. Design

If the drop height of the chute drop structure is small enough, the structure becomes essentially a vertical drop, in that the nappe does not land on the slope, but in the basin. In this case the sloping floor could be omitted and the stilling basin floor extended back to the base of the vertical weir. This same design could be used for any height, but the cost increases rapidly for higher vertical drops because of excessive abutment wall height. However, lower drops (up to 2.5 m) may in many instances be less expensive if designed as vertical drops instead of chute drops. A definition sketch for the vertical drop is shown in Figure 6.

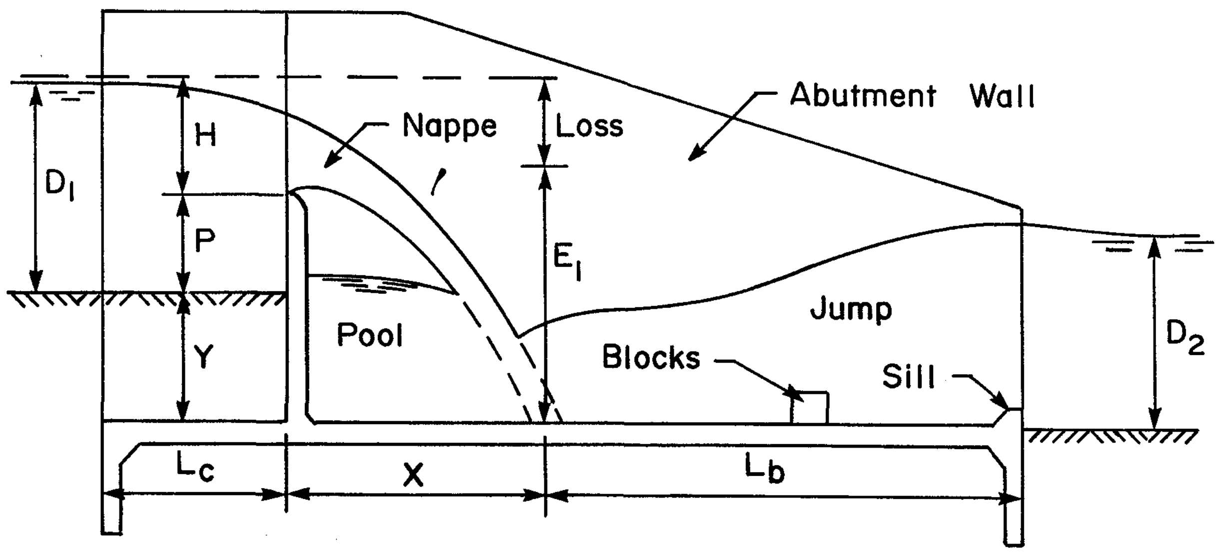

Figure 6 Definition Sketch for a Vertical Drop Structure

If the structure is to be designed as a vertical drop, then to get results equivalent to those obtainable with the chute drop, certain principles must be adhered to, to wit:

(1) The design for the width of structure, approach and discharge channel, inlet to weir, height of weir, and ventilation of nappe are the same as for the chute drop, as given by [10], [11] and [12].

(2) The length of the stilling basin must be 3 d_2 beyond where the nappe strikes the floor, which is considered as the start of the jump. The position of the nappe and energy E_1 available at the base of the nappe must be calculated.

(3) The use of chute blocks naturally does not apply. However, basin blocks and an end sill are designed as for the chute drop. The blocks must be positioned 1.5 d2 in front of the structure exit.

(4) Since the vertical drop is shorter than the chute drop, it may be necessary to use downstream wingwalls to retain the backfill. Vertical drops designed according to these principles are suitable for any range of discharge from 0.10 m^3/s, as in a highway ditch or farmer's ditch, up to several hundred m^3/s, as in a main irrigation canal. A large reinforced concrete vertical drop is shown in Figure 7.

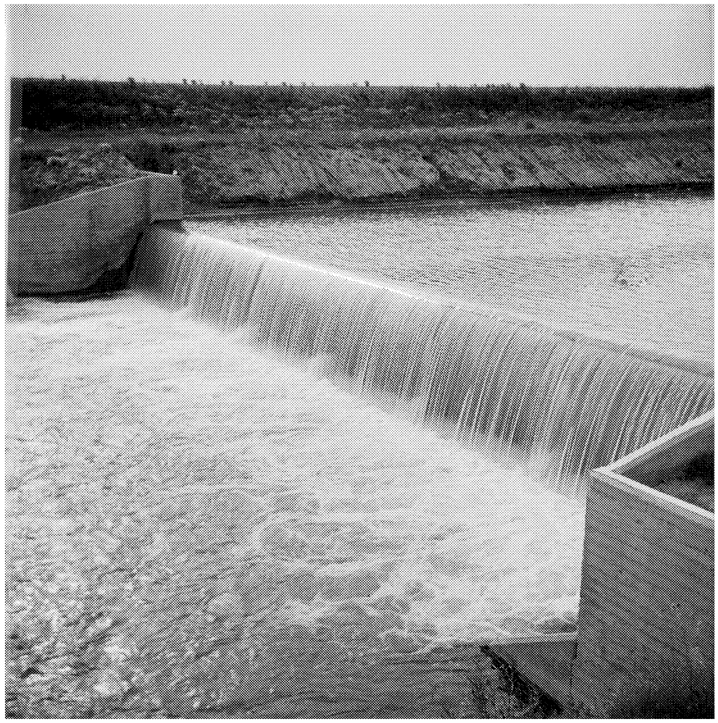

Figure 7 Vertical Drop Structure in Operation

10. Length of Drop Structure

The total length of the vertical drop structure is $L_e + X + L_b$, or $2H + X + 3d_2$, which is the equivalent. The term X is taken as the effective position where the nappe strikes the floor. It is evident that both X and L_b will increase simultaneously as either the head H increases or as the height of the drop Y increases.

If the potential specific energy of the flow at the base of the nappe is defined as E_1', then

[16] $$E_1' = H + P + Y$$

This is the energy that the flow would have relative to the floor of the stilling basin if there was no energy loss in the nappe. However, a pool will form under the nappe. It can be shown by application of the momentum equation that such a pool is necessary in order to provide the horizontal force required to deflect the nappe horizontally. This pool is agitated and rotated by the overfalling nappe, and so absorbs energy. The actual specific energy at the base of the nappe is then E_1, which is less than E_1' by the amount of the energy loss. This loss should be taken into account, otherwise the basin length will be longer than necessary.

It has been shown by experiment that X and E_1, by coincidence, are numerically equal for all values of the head and drop height. These values can be determined from the data given in Table 3. The nature of variation in these values is entirely logical. When H is small the nappe falls close to the weir. Since the nappe is thin and lands almost vertically a larger percentage of the head is lost. When H is large the nappe falls well away from the weir. In this case the nappe is thick and strikes the floor at a much flatter angle, so a smaller percentage of the head is lost.

TABLE 3
NAPPE POSITION AND SPECIFIC ENERGY FOR A VERTICAL DROP

H/(P + Y)	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8
X/E_1'	0.45	0.62	0.74	0.82	0.87	0.90	0.92	0.93
E_1/E_1'	0.45	0.62	0.74	0.82	0.87	0.90	0.92	0.93

After E_1 is determined, v_1, d_1, d_1 and L_b are determined in sequence, as in the case of the chute drop. Usually the natural downstream canal depth D_2 will be greater than the required jump height d_2, so the floor will not have to be placed below the canal bed.

D. BAFFLED CHUTE DROP STRUCTURE

11. Design

The hydraulic performance of the drop structures described in previous sections is related to the downstream tailwater depth, and hence a foreknowledge of the tailwater rating curve is required for the design. A satisfactory curve can be determined by calculation for most artificial channels. Should the structure discharge into a natural drainage course, as in the case of a canal wasteway, for example, the stage-discharge relationship may be completely unknown. Either data for the channel may not be available, or the channel properties may be so variable and complex that a tailwater evaluation cannot be made. Under these circumstances a hydraulic jump stilling basin may not be suitable. A structure is required in which the energy will be dissipated without a stilling basin. The baffled chute is such a device.

The baffled chute has certain elements in common with both the chute drop structure and the multiple orifice pipe drop structure described in Section 17. It is a chute type structure, but the energyis dissipated within the structure while the flow is dropping in elevation, rather than at the end. This is achieved by a series of baffle blocks closely spaced along the full length of the sloping chute, as shown in Figure 8. In this way the velocity at the end of the chute is no greater than at the start, and a conventional stilling basin is not used.

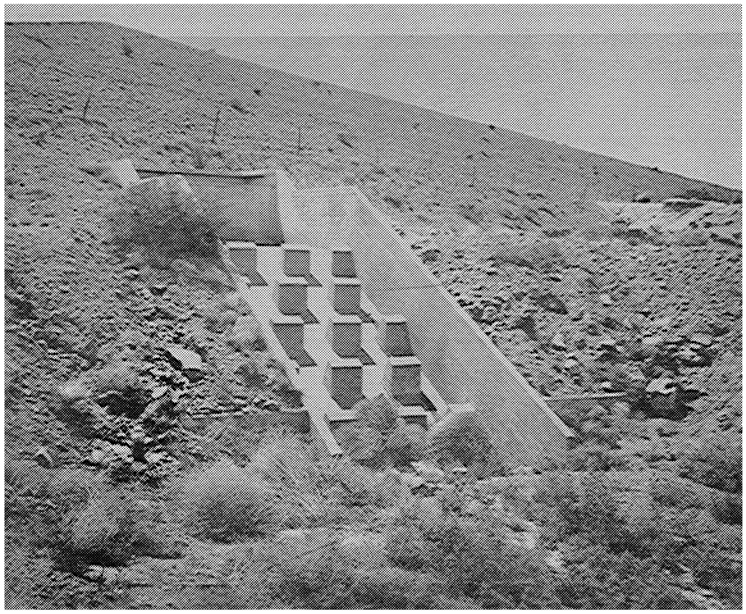

Figure 8. Prototype Baffled Chute

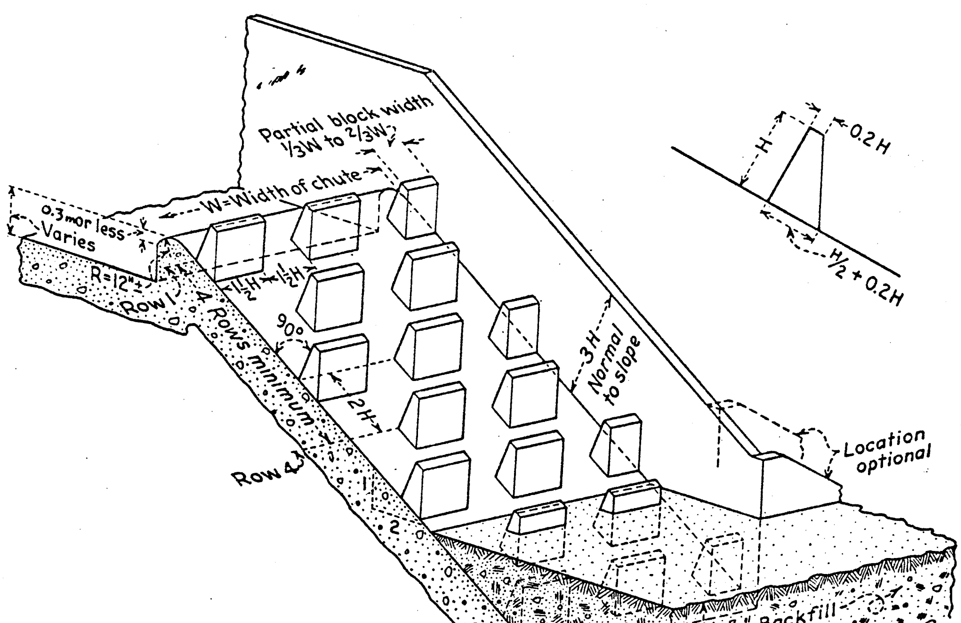

Figure 9. Baffle Details

All dimensions for the baffled chute are related to the baffle height H, as indicated on Figure 9. In turn, H must be set equal to 0.8 times the critical depth of flow at the design discharge, or

[17] $$H = 0.8\,(q^2/g)^{1/3}$$

The disadvantages of the design are that the spray is not entirely confined to the chute, the baffles may catch debris during lower flows, and extra forming is required during construction.

E. RESERVOIR INLET STRUCTURES

12. Description

The reservoir inlet structure is used to drop water from a supply canal into a storage reservoir. It may be a gated or ungated structure, and in many respects it resembles a chute spillway or chute drop structure. The one important difference is that the tailwater level for a reservoir inlet structure may vary over a wide range. Whereas the standard level floor hydraulic jump basin has the floor elevation set for a particular tailwater at the design discharge, energy dissipation for the reservoir inlet structure must take place at whatever elevation the reservoir is located. This is best achieved with the sloping floor hydraulic jump.

It is important to note that the design for a hydraulic jump on a continuous slope can be used for any chute type structure where for some reason there will be a wide variation in tailwater. For example, a chute spillway may discharge into the river at a point immediately above the confluence with an adjoining stream. The tailwater at the stilling basin may be high or low, depending upon the discharge in the other channel. Another example is a wasteway at the dead end of a canal system which discharges back into the river. The tailwater on the wasteway, being dependent upon the river flow and not the flow in the wasteway, may vary over wide limits.

13. Momentum Equation for the Sloping Jump

The momentum equation must be satisfied for the sloping jump just as for the level jump. However, in the case of the sloping jump, the weight of the jump must be considered in the analysis. The weight of the jump is dependent upon the length, and as this is unknown and must be found by experiment, it is not possible to solve directly for the jump height by the momentum equation as in the case of the jump on a level floor. Given the jump dimensions, however, it is possible to prove that the momentum equation is satisfied.

Referring to the definition sketch on Figure 10, the length of jump L_j is

[18] $$L_j = (d_2' - h_j - d_1)/S_o$$

The vertical depth at the end of the roller is taken as d_2' in order to avoid confusion with d_2, as used for the level floor jump. The small error between the vertical depth and the normal depth d_1 at the toe has been neglected in this equation. The momentum equation, written for forces and momentum parallel to the slope, will take the form

[19] $$F_2 \cos\theta - F_1 - W \sin\theta = q\rho\,(v_1 - v_2)$$

from which

[20] $$(\gamma d_2'^2/2)\cos\theta - (\gamma d_1^2/2)\cos\theta - K\gamma[(d_1 + d_2')/2]\,L_j \sin\theta = q^2\rho\,(1/d_1 - 1/d_2')$$

The third term represents the component of the weight of the jump down the slope. The factor K accounts for the fact that the true cross-sectional area of the jump is greater than $(d_1 + d_2')L_j/2$ because the surface of the jump is curved above a straight line joining the toe to the end of the roller. The value of K must be slightly greater than unity.

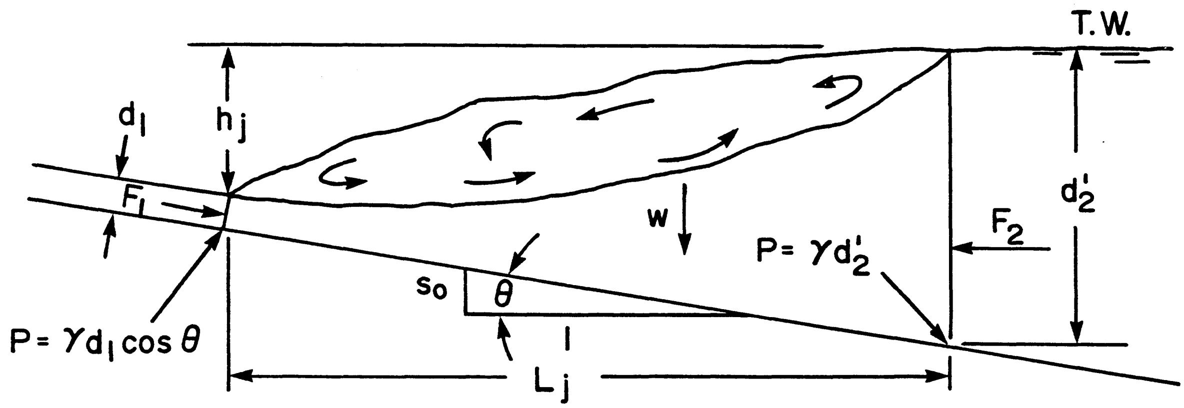

Figure 10 Definition Sketch for a Jump on a Slope

Equation [20] shows that d_2' depends upon Lj as well as K, d_1 and θ. Since there are 2 unknowns the equation cannot be solved. The jump dimensions must be determined from experiment, and therefore [20] is primarily of academic interest.

14. Jump Dimensions

The classical jump equation

[21] $$d_2 = 1/2\,(\sqrt{8F_1^2 + 1} - 1)\,d_1$$

shows that for a level floor jump

[22] $$d_2 = \mathrm{fcn}(F_1, d_1)$$

It is evident that for a sloping floor jump

[23] $$d_2' = \mathrm{fcn}(F_1, d_1, S_o)$$

which accounts for the additional variable, namely S_o. Equations [22] and [23] may be combined to yield

[24] $$d_2' = \text{fcn}(d_2, S_o)$$

in which the effect of Froude number is accounted for implicitly in the sequent depth term d_2.

The experimental solution for [24] is

[25] $$d_2' = (1 + 4.5\ S_o)\ d_2$$

Although d_2' is always greater than d_2 for all positive values of S_o, the jump height h_j actually decreases as the chute slope increases. The value h_j has been determined as

[26] $$h_j = (1 - 1.8S_o)(d_2 - d_1)$$

It should be pointed out that [26] alone is sufficient to locate the position of the toe of the hydraulic jump. As shown on Figure 10 the toe of the jump occurs at a point on the surface of the jet located at a vertical height of exactly h_j below the tailwater level. Other jump dimensions can then be solved from [18] and [25]. The range of possible jump locations is required in order to design wall heights, floor slab thicknesses, and the drainage system. In the absence of a pressure relief system, the floor slab must be designed to resist the full uplift force due to the jump.

15. Design

The sloping chute could terminate with a standard level floor stilling basin set d_2 below the minimum tailwater level at design discharge. The basin would be completely submerged during high tailwater. As an alternative to the level floor basin, a continuous slope could be used right to the end of the structure. In this case the floor slab at the end of the slope should be set $(1 + 3S_o - 4S_o^2)d_2$ below the minimum tailwater, as shown in Figure 11. The advantage of the continuous slope is a somewhat simpler design and a shorter overall length of structure. On the other hand the elevation at the end of the floor slab will be lower and the excavation may be deeper than if the slope is terminated with a level floor basin.

The other item requiring attention for the sloping jump design is the height of the adjacent sidewalls. These walls must be high enough to exclude tailwater from the jump area over the entire length of slope on which the jump may form. Fortunately, it is not necessary to contain the full depth d_2' at the end of the jump. Model tests show that the performance will be comparable to that obtained with the standard basin if the wall height is made in accordance with

[27] $$h_w = (1.05 + S_o)d_2$$

Although this height is less than d_2', it is still greater than the wall height necessary for the level basin. Further, the necessary height increases as the slope increases. For this and stability reasons it is unlikely that a slope steeper than $S_o = 0.25$ would be used in practice.

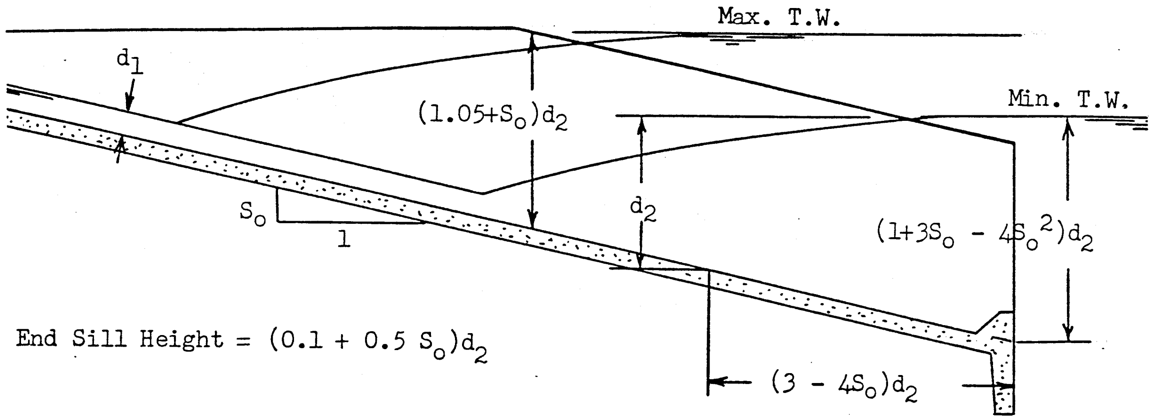

Figure 11. Design for Jump on a Continuous Slope

Figure 12 shows the appearance of a jump on a prototype chute with a continuous slope of 0.25, designed according to the recommendations in Figure 11.

Figure 12. Jump on a Continuous Slope, Cadillac Dam, Saskatchewan

F. PIPE DROP STRUCTURES

16. General

Where crossings are required on a canal consideration may be given to combining the crossing with the drop structure. For an open type drop structure a bridge would be required in any case, so there may be little to be gained by combining the structures. However, if a closed conduit is used, the conduit can be buried and the fill used for a roadway.

Corrugated steel pipe is often used for smaller pipe drop structures. It is easily handled, being light in weight, and may be installed rapidly. Although the wall friction is considerably greater than for concrete pipe, this is an asset for a drop structure because it helps to dissipate the excess energy.

One disadvantage of a pipe drop structure is that it will not accommodate an overload. The head varies as the discharge squared for any closed conduit, hence even a modest increase in discharge may result in an excessive head rise and overtopping of the upstream canal banks. Open type structures, on the other hand, may accommodate a 50% increase in discharge before overtopping.

A common design error in pipe flow calculations is to assume that the pipe will flow full just because the inlet is submerged. For example, a pipe may be sized for a velocity of 2 m/s flowing full, this velocity having been selected as one which could be easily resisted at the outlet with riprap alone. However, if the pipe is steep, free surface flow may occur in the pipe. Should the depth be less than half the pipe diameter, for example, the velocity at the outlet may be 5 or 6 m/s. Use of a smaller pipe may result in full pipe flow, but the velocity at the outlet would still be high. This problem has been solved in practice by dissipating the excess energy by a series of orifices in the pipe, by an impact basin at the end of the pipe, or with a drop-shaft energy dissipator.

17. Multiple Orifice Design

By this method the pipe is sized so that the outlet velocity V_p, based on full pipe flow, is so low that an outlet structure is not needed. A velocity must be selected which can be resisted by riprap alone, usually 2 m/s or less. The required pipe area is determined from the continuity equation

[28] $$A_p = Q/V_p$$

The difference between the drop height and the friction loss in the pipe represents excess head which must be dissipated by other means, otherwise the pipe will not flow full as assumed. A series of thin metal plate orifices are installed in the inclined portion of the pipe. The orifice area A_o is sized so that the head loss across the orifices accounts for most of the surplus head. The loss per orifice may be calculated from the classical equation for head loss at an abrupt expansion

[29] $$h_L = (V_j - V_p)^2 /2g$$

in which V_j is the velocity of the contracted jet produced by each orifice. Since

[30] $A_j = Q/V_j$

and

[31] $A_o = A_j/C_c$

the required size of the orifice can be computed. In [30] and [31] A_j represents the area of the contracted jet, Ao the area of the orifice, and C_c the coefficient of contraction. The value of C_c depends upon the ratio of the orifice area to pipe area A_o/A_p, and is given in Table 4.

TABLE 4
EFFECT OF AREA RATIO ON ORIFICE COEFFICIENT

A_o/A_p	0.3	0.4	0.5	0.6	0.7
C_c	0.65	0.67	0.69	0.71	0.74

It is desirable that the losses be distributed over the length of the sloping pipe rather than concentrated at one point. Although this will require a greater number of orifices, the openings will be larger, thus reducing the risk of plugging by debris and avoiding excessive jet velocities inside the pipe. In order to effectively dissipate the energy at each orifice they must be spaced at least 2 pipe diameters apart with the last orifice located at the end of the sloping pipe. Generally, spacings will be increased for pipes with flatter slopes.

Some design details are shown on Figure 13 for a pipe with a slope of 0.5. At the bottom of the inclined pipe a horizontal pipe length of at least 5D should be laid to allow the flow to adjust to a normal velocity distribution after the orifice and bend. This pipe should be placed so that at design flow the crown of the pipe is at, or slightly below, tailwater level. Since the risk of plugging is greater than for a plain pipe if debris is allowed to enter the pipe, it is recommended that a trashrack be used. This is particularly important for pipes which are too small to permit access for cleaning. A bar spacing of 150 mm is recommended for trashracks for canal structures.

<u>Example 3:</u>

Calculate the number and size of orifices necessary to discharge 0.7 m^3/s at less than 2 m/s from a pipe drop, if the drop in water surface elevation is 2.4 m. A corrugated steel pipe with n = 0.024 will be used, and installed as shown in Figure 13.

A 700 mm pipe will have an area of 0.385 m^2 and produce a mean velocity at the outlet of 1.82 m/s, which is acceptable. The length of the inclined pipe will be

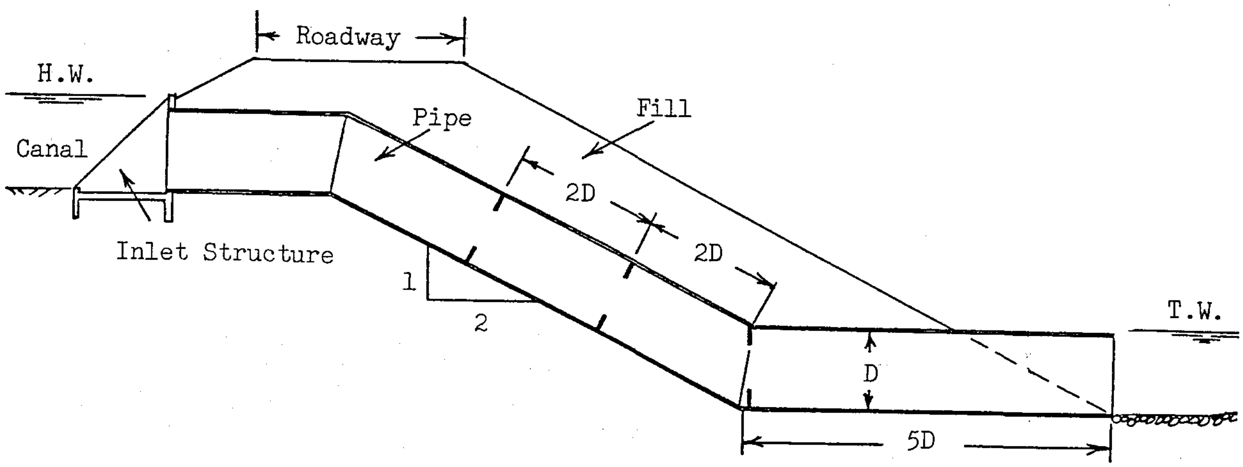

Figure 13 Multiple Orifice Pipe Drop

$\sqrt{2.4^2 + 4.8^2} = 5.37$ m, and since the orifices must be spaced at 1.4 m, three orifices can be placed in this length. The total length of pipe will be about 10 m, and therefore the natural inlet, friction, and outlet loss may be calculated as

$$h_L = [0.5 + (2g \times 0.024^2 \times 10)/0.175^{4/3} + 1.0]\ 1.82^2/2g = 0.448 \text{ m}$$

The remaining excess head is 2.40 - 0.448 = 1.952 m, or 0.651 m per orifice. From [29],

$$(V_j - 1.82)^2/2g = 0.651 \text{ m}$$

from which $V_j = 5.394$ m/s

and therefore $A_j = 0.7/5.394 = 0.123$ m^2

Selecting a trial value for Cc of 0.685, then

$$A_o = 0.123/0.685 = 0.180 \text{ m}^2$$

Since $A_o/A_p = 0.468$, C_c from Table 4 is 0.684. The trial value is satisfactory and the orifice diameter will be 0.479 m.

18. Impact Basin

For flow velocities up to 8 m/s and discharges up to 10 m^3/s, an impact basin may be used at the end of the pipe to dissipate the excess energy of flow. The impact basin consists of a rectangular box-like structure with the long dimension parallel to the axis of the pipe. The discharge from the pipe is intercepted by a solid hanging baffle which extends over the full width of the structure. The bottom of the baffle is placed at the same elevation as the invert of the pipe, so the jet is forced to spread out to pass under the baffle. In a sense the baffle effect is similar to that produced by a sluice gate, and the flow becomes more or less uniformly distributed as it passes under the baffle.

Figure 14 shows the impact basin proposed by the United States Bureau of Reclamation. As originally developed, this design was not intended for use as a canal structure, but rather for discharge into channels where the natural tailwater would be insufficient to force a hydraulic jump. Turbulent mixing in the area upstream from the baffle will reduce the energy content to a certain upper limit, regardless of the energy level of the incoming flow and regardless of the absence of any tailwater. However, to insure that the supercritical flow under the baffle does not pass through the structure unimpeded, a high end sill is required. With the top of the sill level with the bottom of the baffle, sufficient tailwater is produced in the basin to submerge the jet.

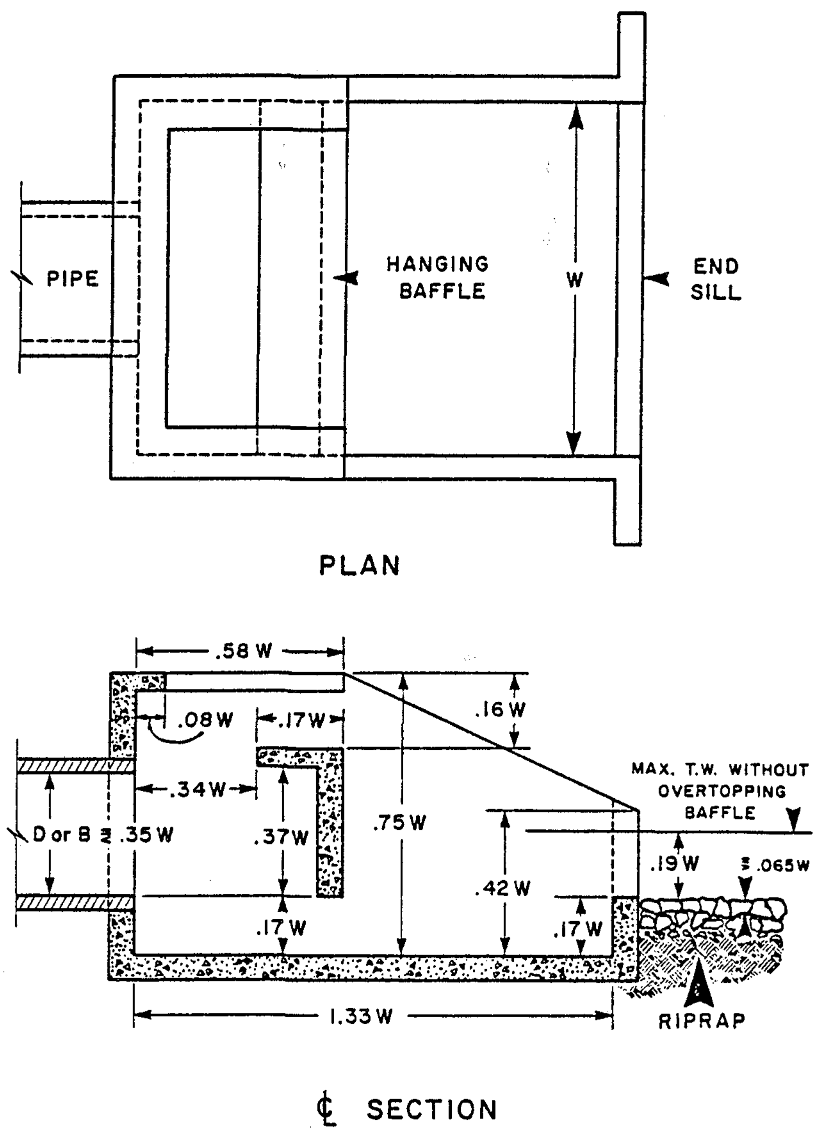

Figure 14 USBR Impact Basin for Pipe Outlet

Despite a significant energy reduction in the impact basin, flow conditions over the end sill are far from ideal. In the absence of any natural tailwater outflow will occur at critical depth, with a large boil over the sill and a plunging jet downstream. Accordingly, very substantial riprapping is required for the discharge channel.

The basin width recommended for the impact basin is a function of the discharge, and may be calculated from

[32] $$W = 1.9Q^{0.4}$$

with W in m and Q in m^3/s. All other dimensions may be expressed in terms of W, as indicated on Figure 14. Up to 10 m^3/s, [32] gives a width not greatly different than $W = 1.8\sqrt{Q}$ as recommended for spillway stilling basins, and in fact at 1.63 m^3/s the width is the same by either equation. However, [32] must be used for the impact basin to insure dynamic similarity between the prototype and the model upon which the equation was based. This is simply the Froude model law, from which the scale ratio for discharge is $Q_R = L_R^{5/2}$, or conversely $L_R = Q_R^{0.4}$

Discharge at critical depth would result in a depth of 0.16W over the riprap. Should some tailwater be present, conditions will be improved. At a tailwater depth of 0.19W, shown on Figure 14, overtopping of the baffle by part of the discharge is imminent. Higher tailwater levels will produce overtopping, but this is not serious since the clearance to the top of the structure is sufficient to allow the entire discharge to pass over the top in the event the opening under the baffle becomes plugged.

For use as a canal structure where definite tailwater levels exist, the outflow conditions can be greatly improved by a few minor modifications to the impact basin, as indicated in Figure 15. The high end sill is replaced by a low one, and the floor is lengthened to accommodate a row of floor blocks. The floor is placed 0.36W below tailwater at design discharge. There is no point in retaining the high end sill when adequate submergence can be obtained by natural tailwater. Elimination of the sill results in improved velocity distribution and greatly reduced average velocity at the exit. Flow conditions are comparable to those obtained with the other types of drop structure, and the need for riprap is minimal.

The floor blocks for the modified impact basin are 0.10W in height and 0.07W in width. Spaces between the blocks equal the block width except at the sidewalls where the spaces are reduced to 0.045W. Hence there are seven blocks and eight spaces (i.e. 7 x 0.07W + 6 x 0.07W + 2 x 0.045W = W). The rationale behind the requirement for block height is that the block height should be equal to the jet depth produced by the hanging baffle, or approximately 0.6 x 0.17W.

An impact basin designed according to this criteria is shown in Figure 16.

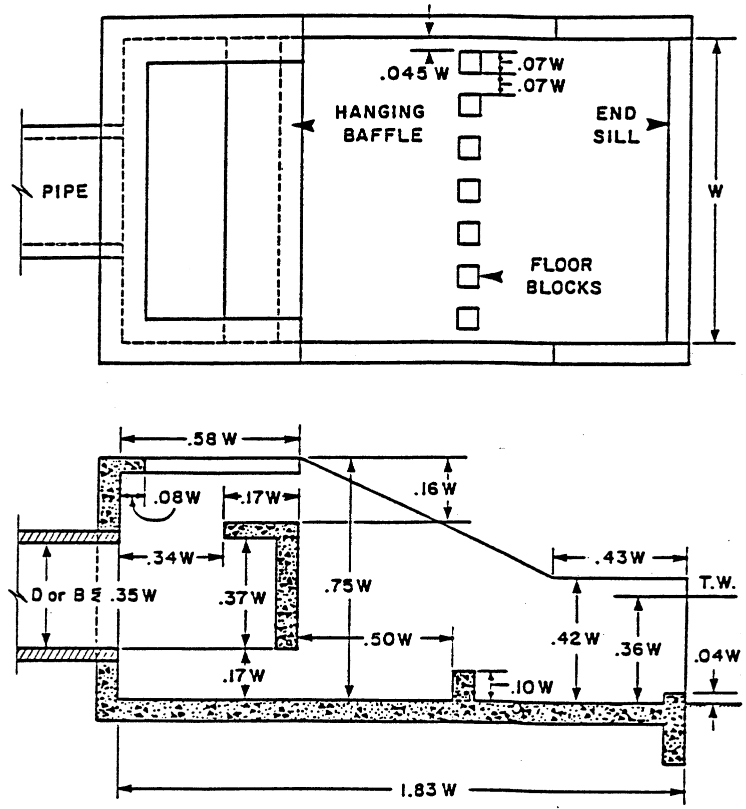

Figure 15. Modified Impact Basin

Figure 16. Impact Basin for Storm Sewer Outfall, Saskatoon, Saskatchewan

19. **Drop-Shaft Energy Dissipator**

The drop-shaft energy dissipator can be used under circumstances similar to those for which an impact basin may be used, that is, high velocity inflow in a small pipe under head, or in a larger pipe on a steep slope, flowing partly full. The excess energy of flow is dissipated by turbulent mixing in the pseudo hydraulic jump at the bottom of the drop-shaft. The primary difference from the impact basin is that the inflow enters the drop-shaft well above the outflow level, and the outflow discharges into a pipe rather than an open channel. The drop-shaft energy dissipator may be particularly adapted to the case where there is a high drop at the outfall from a storm sewer, as is often the case when the sewer discharges into a river with steep high banks. Design details for the structure are shown in Figure 17. In this figure, the drop height (h) is defined as the drop between the energy level in the inflow pipe to the invert of the outflow pipe, and the head loss in the drop-shaft (h_L) is the drop in energy level from the inflow to the outflow pipe.

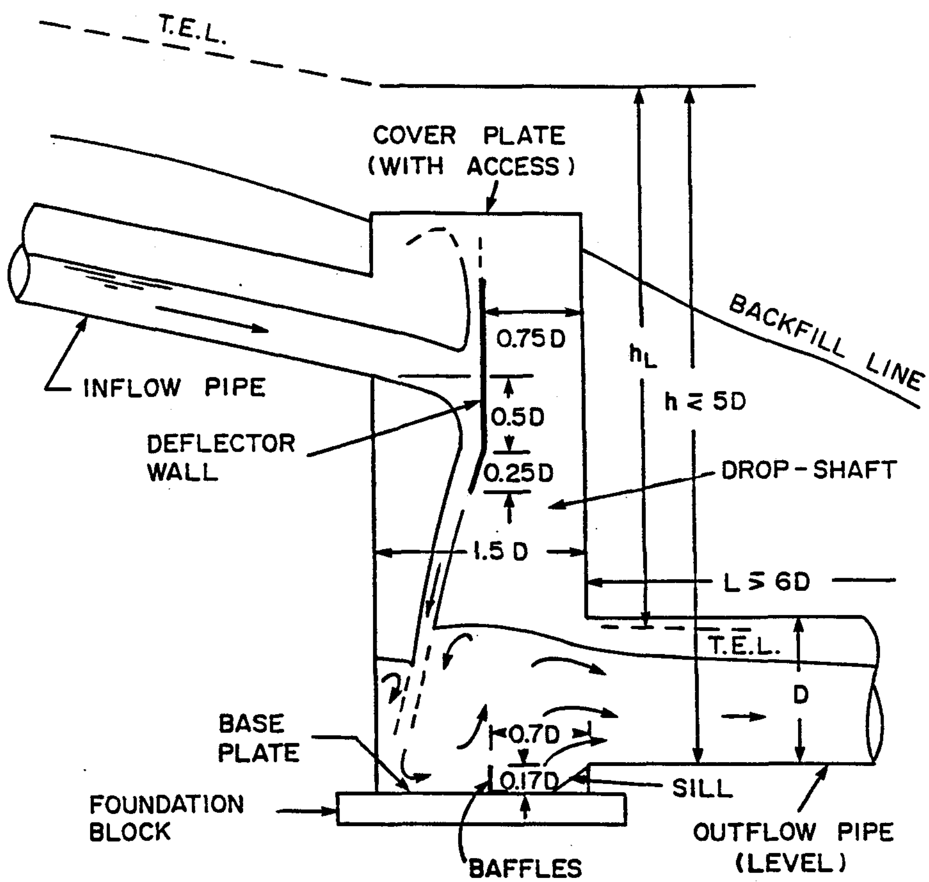

Figure 17. Drop-Shaft Energy Dissipator

The drop-shaft consists of a vertical concrete or corrugated steel pipe with an enlarged diameter (minimum size 1.5 times the diameter of the outfall pipe). The inflow pipe may vary widely in size, slope and elevation. The outflow pipe diameter should be sized to minimize outlet velocities, such that $Q/D^{5/2}$ is between 1 and 2, or

[33] $$0.76Q^{0.4} < D < 1.0Q^{0.4}$$

with D in m and Q in m^3/s. All other dimensions for the drop shaft are related to the outflow pipe diameter in the same way that all impact basin dimensions are related to the basin width.

If the outflow pipe discharges into a canal for warm weather flows only, it may be submerged and flow full. For full pipe flow the outlet velocity depends on the pipe area only, and the lower limiting diameter of $0.76\ Q^{0.4}$ may be satisfactory. Local riprapping of the bed at the outlet would be required as covered in Section 20 of Chapter 10 on Culvert Hydraulics. If the installation is for storm sewer discharge, the outflow pipe may be located above winter ice levels of the receiving stream and the pipe outlet may not be submerged during operation. The flow will pass through brink depth at the end of the pipe before dropping to the tailwater level. In this case, velocities will be considerably larger than corresponding to full flow, and a larger pipe size, approaching $1.0Q^{0.4}$, is more appropriate. The outflow velocity naturally will be smaller for a larger pipe. A depressed riprap splash pad should be placed to receive the discharging jet, as covered in Part D of Chapter 7 on Stone Structures. When an unsubmerged outlet pipe is used, it should have a length of about 6D downstream from the drop-shaft in order to allow development of some uniformity in the flow preceding the outlet.

The important components of the structure are the deflector wall (or plate), the recess at the base, floor baffles and sloping sill. These components are also important features of the impact basin. The deflector wall is by far the most important feature. The impacting jet from the inflow pipe is deflected toward the upstream side of the base of the drop shaft, allowing the full diameter of the drop shaft for energy dissipation prior to the flow entering the outflow pipe. The jet deflection is achieved by the lip, of the height 0.25D, located at the bottom of the deflector wall. The direction of the lip should be toward the intersection between the upstream wall of the drop-shaft and the base, and hence, the lip angle will vary with the height of the drop-shaft. Hydraulic models have shown that there is intense turbulence and air entrainment at the bottom of the drop-shaft, and the hydraulic action closely imitates that for a normal hydraulic jump with a counter-clockwise surface roller. If the deflector wall is omitted, a deep clockwise rotating pool forms in the drop-shaft, and the jet flow down the downstream wall of the drop-shaft is deflected into the outflow pipe with considerable residual energy, producing supercritical flow and velocities 2 to 3 times greater than when the wall is in place. A schematic view of the flow pattern with and without the deflector wall is shown in Figure 18 and 19.

The floor baffles consist of 4 short angle irons welded to the base plate. The baffles are the same height as the recess below the invert of the outflow height, 0.17D, and have a width equal to the space between, such that the 5 spaces and 4 baffles account for the full width of the base. The sloping sill, which may be a welded plate or trowelled asphaltic concrete, is intended to produce smooth entry to the outflow pipe at the invert. The baffles and sill are most useful for the situation where the outflow pipe is short and unsubmerged. If the outflow pipe is long(>>6D) or submerged, the baffles and sill can be omitted since the flow at the end of the pipe will be the same with or without these features (e.g., full pipe flow for a submerged pipe).

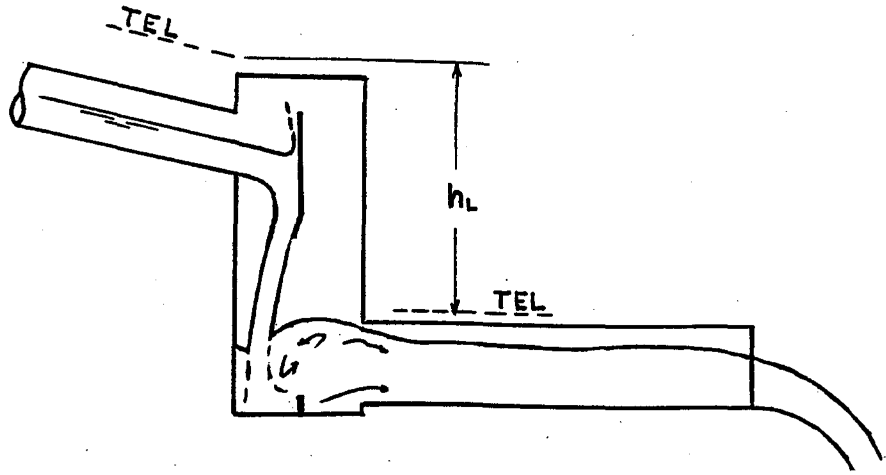

Figure 18. Flow Pattern with Deflector Wall in Place

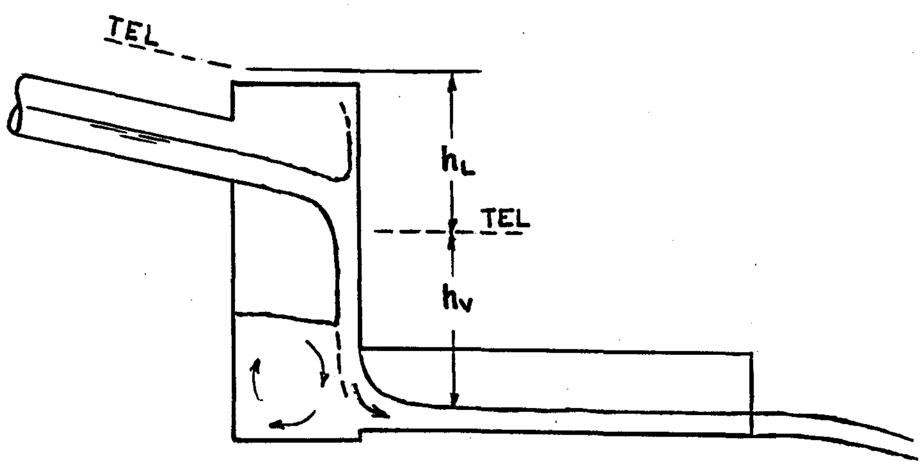

Figure 19. Flow Pattern without Deflector Wall

Other features of the structure include the foundation block, which may be a precast concrete pad, the base plate and cover plate. The steel base plate may be fastened to the foundation block with anchor bolts. The cover plate should have a padlocked access hatch and must contain the vertical spray produced by the portion of the discharge directed upward upon impact with the deflector wall. Models indicate that the spray height could reach at least 80% of the velocity head of the inflow if not contained. Ladder rungs would normally be placed insider the drop-shaft under the access hatch. The deflector wall may be a steel plate bolted to the sides of the drop-shaft with angle iron lugs for connectors. The plate and connections must be designed to carry the impact load of the inflow jet, equal to QρV. If necessary, a strut or stiffener can be placed on the downstream side of the deflector wall.

In the absence of any energy dissipation, the specific energy in the outflow pipe would equal the vertical height from the invert of the outflow pipe to the total energy level (TEL) of the inflow at the drop shaft. This height is defined as h. Since the bottom lip of the deflector wall should be above the crown of the outflow pipe, it is unlikely that the drop-shaft could be adapted to the situation unless the h value is at least 3D. For a value of h up to 5D, the drop-shaft diameter D_s should be 1.5D. It is reasonable that a larger water volume would be required at the bottom of the drop-shaft to properly dissipate the energy of flow for either an increase in Q or an increase in drop height h. The effect of a change in Q is accounted for through [33] with D_s = 1.5D, and in fact the relationship that D_s varies as $Q^{0.4}$ simply follows the similitude law for discharge. However, an increase in h to a value greater than 5D must be accounted for separately, and can be done by noting that, for a given discharge, the volume of a normal hydraulic jump varies approximately as the square root of the input head, or $\forall$ varies as $\sqrt{h}$.

The pool volume at the bottom of the drop-shaft will vary as D_s^2, because for a given outflow pipe diameter and discharge the flow depth in the pipe will be the same, and the depth in the pool will likewise be the same. Thus $D_s^2 \propto \sqrt{h}$, or $D_s \propto h^{0.25}$ from which

[34] $$D_s = 1.5D\,[h/(5D)]^{0.25}$$

given $h \geq 5D$. If h = 10D, [34] would give D_s = 1.78D, for example.

It was initially expected that if the smaller size pipe and drop-shaft was selected ($D = 0.76\ Q^{0.4}$), it would be necessary to increase the depth of the recess at the base in order to produce adequate energy dissipation. This was not the case. The recess of 0.17D was satisfactory for all designs. Recess depths of 0.7D, 0.34D, and 0.17D were investigated on a physical model, and it was found that flow conditions in the outfall pipe were essentially the same for each. Any benefit due to a deeper pool was offset by the flow contraction at the inlet of the outfall pipe. The jet moving vertically upward along the downstream wall of the drop-shaft (toward the invert), caused a pronounced contraction inside the pipe inlet. With the shallow pool and sloping sill, the flow entered the pipe essentially horizontally, without contraction at the invert, and flow depths in the outfall pipe were just as great as they were for the larger recess depths. Conditions in the outflow pipe are independent of the drop height because energy dissipation in the intensely turbulent counter-clockwise roller automatically adjusts to whatever is needed (similar to the impact basin). For a free outlet, for all values of $Q/D^{5/2}$ between 1 and 2, it was found that the flow depth at the end of the outfall was in basic agreement with the theory for brink depth, which, for a circular section is approximately 0.725 of the critical depth, viz.

[35] $$y_b \approx 0.725 y_c$$

Figure 20 shows the calculated energy of flow versus depth for part full flow in a circular pipe. The energy for any value of $Q/D^{5/2}$ is always a minimum at critical depth. With the free outlet (unsubmerged) and short outlet pipe (6D), the flow in the outfall pipe upstream from the brink was always deeper than y_b, but generally somewhat smaller than critical depth. However, as shown on Figure 20, the specific energy versus depth curve is so flat in the vicinity of critical depth, that the energy content is virtually the same whether the flow depth is slightly more or less than critical depth. Thus, whether the upstream flow depth is slightly more or less than critical depth has no significance in terms of the velocity attained by the free jet at the end of the pipe, because the energy level for the freely discharging jet at the end of the pipe is basically at the minimum level anyway, provided always that the design incorporates the deflector wall. If the deflector wall is omitted, the flow in the outfall pipe will occur at less than half of critical depth, and the energy of flow will be greatly increased.

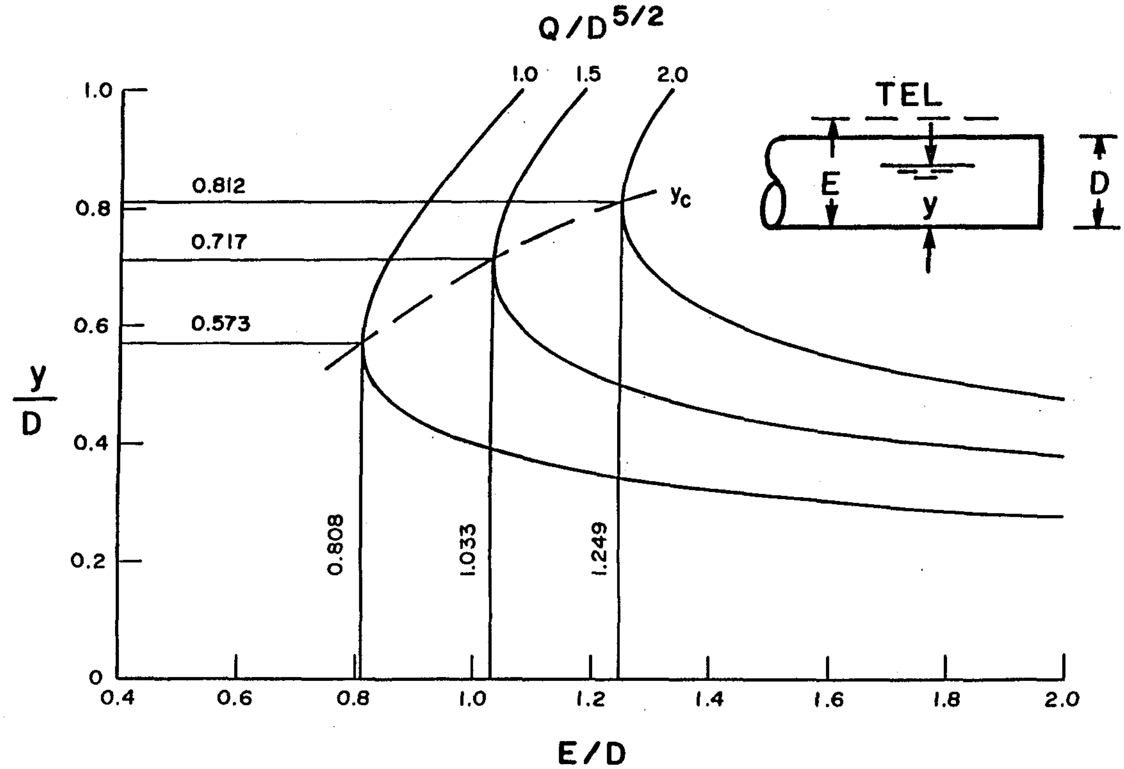

Figure 20. Flow Depth vs. Energy of Flow for Part Full Flow in a Circular Pipe

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PROBLEMS

1. A canal for 28 m^3/s must be excavated through a sandy soil. Prevailing ground slopes along the route are 0.001. How far apart should 1.5 m drop structures be located? Use n = 0.02 and 2:1 side slopes for the canal.

2. Discuss the pros and cons of a grassed channel versus stone paving to protect a steep grade.

3. An approximate rule for selecting the basin width of canal structures is $W = 2.3 \sqrt{Q}$. This is greater than is usually selected for spillway stilling basins. What is the reason fo this difference?

4. Sketch the centre-line section of a chute drop structure and label all the components.

5. A chute drop structure has a vertical weir 0.7 m high at the crest, and a drop in elevation of 3 m from upstream canal bed to downstream canal bed. The chute slope is 0.333. Determine the flow velocity at the toe of the hydraulic jump when the head on the weir is 0.5 m.

6. (a) Calculate the required weir height P for a reinforced concrete chute drop structure for 56 m^3/s if the upstream canal depth and velocity are 3 m and 1 m/s respectively. Use C = 1.837 for the weir coefficient.

   (b) If the drop in canal bed grade is 6 m, what specific energy will be available at the bottom of the chute?

7. A vertical drop structure for 30 m^3/s is to be designed for a drop in canal grade of 2 m. The upstream canal depth and velocity is 2.4 m and 1 m/s respectively. Calculate the total length of the structure parallel to the direction of flow.

8. What structural consideration places an upper limit on the economical height of a vertical drop?

9. A 1.5 m diameter concrete pipe is used as a canal wasteway structure to discharge 3.54 m^3/s down a 20% slope to a natural creek bed 13.7 m lower than the canal.

The n value is 0.012, and if allowed to flow partly full, the terminal velocity in the pipe will be 12.5 m/s, and an impact basin or outlet structure will be required. Instead, it is decided to dissipate the energy with a multiple orifice design. Calculate the number and size of orifices required to make the pipe flow full.

10. A hydraulic jump forms on a continuous slope of 6 horizontal to 1 vertical. If $d_1 = 0.3$ m and $v_1 = 15.24$ m/s, what sidewall height should be used adjacent to the jump? In the absence of pressure relief, what will be the maximum uplift pressure head? Calculate the total uplift force per unit width of structure for the whole potential uplift triangle.

11. A hydraulic jump occurs on a continuous slope of $S_o = 0.25$. At the toe of the jump $v_1 = 15.24$ m/s and $d_1 = 0.3$ m. Show that jump dimensions given by Equations 18, 25 and 26 satisfy the momentum theory for the sloping jump, if the factor K = 1.02.

12. The 900 mm inflow pipe to a drop-shaft energy dissipator carries 1.57 m^3/s, flowing half full. The invert elevation of the pipe is 97.0 m. A 1200 mm diameter pipe with invert elevation of 93.0 m is used for the outfall pipe, which discharges the flow at critical depth. Show that the head loss in the drop-shaft (h_L) will be 83% of the drop height (h), and that the flow velocity in the outfall pipe will be 2.35 m/s.

CHAPTER VII

STONE STRUCTURES

A. INTRODUCTION

1. History

Construction of civil engineering works with stone or rock dates from biblical times. Stone is generally abundantly available and easily accessible. It is often available in a range of sizes from surface deposits, river banks and gravel pits, or it can be quarried from rock outcrops. Stone is strong, heavy, chemically inert and durable. These features make stone a desirable and often economic building material.

In a sense the history of civil engineering is written in stone. Many stone structures built for fortifications, transportation, worship, housing, water supply and entertainment are still in existence today, testifying to the durability of stone as a construction material. Examples include the Pyramids of Egypt, dating from 4700 B.C., the Acropolis in Greece from 400 B.C., the Great Wall of China from 200 B.C., Stonehenge in Great Britain, date unknown, the Coliseum in Italy from the 1st century, the Leaning Tower of Pisa from the 10th century, the Aztec Temples in Mexico from the 14th century, and many more. These structures were built without the modern day advantages of steel reinforcement, epoxies or high strength cement. In fact the need to construct a roof, window opening or door opening without steel to carry tension led to the early development of the arch and dome. Arch technology was well advanced many centuries ago, and the principle is still used today in concrete arch bridges and arch dams.

An indication of the strength of natural stone in comparison to concrete is given in Table 1.

TABLE 1

CRUSHING STRENGTH OF BUILDING MATERIALS
(Normal Upper Limits)

Material	Strength in kPa
Concrete (man made)	50,000
Sandstone (sedimentary)	140,000
Limestone (sedimentary)	210,000
Marble (metamorphic)	210,000
Granite (igneous)	280,000

2. Application in Hydraulic Engineering

In hydraulic engineering stone has been used for rockfill dams, breakwaters, jetties, spurs, groynes, pier protection, bank and shore protection, and scour control at hydraulic structures. In the decade of the 1970's, however, with a booming world economy and significant emphasis on mega projects, there was a tendency to construct increasing numbers of more sophisticated reinforced concrete structures. This is not always appropriate technology. In developing countries in particular, use of locally available materials and labour intensive methods makes good socio-economic sense.

If very large discharges or velocities are to be handled, this will usually dictate the need for reinforced concrete, because stone sizes required for stability become prohibitively large under these conditions. However, there are many instances of the need for smaller size structures for drainage, irrigation and water resource projects. When stone of a suitable quality and quantity is available near the site of such a structure, it should be considered as a possible construction material. Advantages include simple design and construction, the absence of the need for skilled labour or quality control, and speed of construction. Of course selection of a building material should always be based on least cost, including consideration of life of structure and operation and maintenance, but stone should never be dismissed out of hand.

In this chapter the use of loose dumped stone is considered for bank and shore protection, for basins below drops or pipes, for fuse plug dams, for river spurs, and for gabion structures. Use of rock for stabilization of steeply sloping channels is covered in Chapter 8 on Flexible Channel Linings for Erosion Control.

B. BANK PROTECTION

3. Resistance to Flowing Current

The velocity to be resisted for bank protection is the local velocity in the vicinity of the bank. These velocities are usually some fraction or multiple of the mean flow velocity in the channel, depending on the velocity distribution. In a straight channel with parallel banks the bank velocity may typically be 2/3 to 3/4 of the mean velocity. In a converging channel the velocity tends to be more uniform, and the bank velocity may equal the mean velocity. In a curved channel the bank velocity can exceed the mean velocity by 1/3 or more. The areas of attack are on the inside bank of the bend at the start and on the outside bank at the end. At the nose of a bridge abutment, spur, or other projection the local velocity may be twice as great as the mean velocity due to flow concentrations produced by the geometry. In the absence of more detailed information from models or stream flow observations, it is recommended that the multiplying factors in terms of mean velocity be 2/3 for a straight channel, 1 for a converging channel, 4/3 for a curved channel and 2 at the nose of projections.

The size of stone riprap required to stabilize an otherwise erodible bank depends on the adjacent velocity and the steepness of the bank. A relationship which has been in use since 1945 by the Division of Highways, State of California is

$$W = 0.00002\, V^6\, S_g/[(S_g - 1)^3 \sin^3(\rho - \alpha)] \tag{1}$$

In this equation W is the stone weight W in lb, V is the stream velocity near the bank in fps, S_g is the specific gravity of the stone, α is the bank slope in degrees, and ρ is a constant recommended at 70 degrees. The equation has many elements which agree with theoretical considerations. The sixth power exponent of the velocity can be derived and has been known since 1829 (Leslie's sixth power law). The $(S_g - 1)^3$ term is to account for the fact that the stone is submerged, and therefore subjected to buoyancy. Intuitively ρ should be the angle of repose of the stone, but an angle of 70 degrees is about double a typical angle of repose. However, if ρ is taken as 35 degrees the equation gives stone sizes that are unrealistically large. The coefficient is empirically derived from observations by California State Highways.

Since individual stones are not weighed in practice, it is convenient to specify the stone by size rather than weight. In the relationship

[2] $$d = 0.225 \sqrt[3]{W}$$

d is the diameter in ft of a spherical stone with a specific gravity of 2.65 (the most typical value) and a weight of W in lb.

Substitution of [1] into [2], along with $\rho = 70$ degrees and $S_g = 2.65$, gives

[3] $$d = 0.00512V^2/\sin(70 - \alpha)$$

Actual riprap will consist of stones of different sizes (i.e., the sample will be graded). This leads to the definition of the median size d_m, which is the equivalent spherical diameter, by weight, of the nominal stone. The weight of all the larger stones in the sample comprise 50% of the total weight. Similarly 50% of the weight of the sample consists of stones smaller than the median.

The California equation is based on the assumption that two-thirds of the stone will be heavier than the weight specified in [1]. To convert this to d_m, a multiplier of 1.2 is recommended. If, in addition, d_m is specified in mm and V in m/s, [3] will become

[4] $$d_m = 20.17V^2/\sin(70 - \alpha)$$

If the actual specific gravity of the stone is different than 2.65, the required stone size may be found by multiplying the value from [4] by $1.65/(S_g - 1)$.

Stone sizes calculated from[4] for various flow velocities and bank slopes are shown in Table 2. It is seen that once the slope becomes quite flat, such as 1:4, the benefit in terms of size reduction due to further flattening is minimal, whereas the volume of stone required to cover the slope would increase appreciably. Therefore it is recommended that where riprap protection is needed a bank slope flatter than 1:4 normally should not be used.

4. Resistance to Wave Action

Dumped stone riprap is one of the oldest methods of resisting wave attack. It is commonly used on the upstream face of earth dams for this purpose. However, there are many other situations where wave attack must be resisted. For example, around the perimeter of a sewage lagoon, a fly ash lagoon, a sodium sulphate precipitating basin, a water storage pond, along a low height earth dike or along the banks of a river. In each of these cases the depth of the water may be relatively shallow. Wind generated waves will be affected by the water depth, and will not be classified as deep water waves as discussed in Section 9 of Chapter 1.

TABLE 2

STONE SIZES FOR BANK PROTECTION
Median Stone Size d_m in mm
(Equivalent Spherical Diameter)

Velocity V in m/s	Side Slope V:H 1:2	1:3	1:4	1:5	1:6	1:8
1.0	29	26	24	23	23	23
1.5	66	58	55	53	52	51
2.0	117	103	97	94	92	91
2.5	183	161	152	147	144	142
3.0	264	232	219	212	208	204
3.5	359	316	298	289	283	278
4.0	469	412	389	377	370	363

From $d_m = 20.17\ V^2/\sin(70 - \alpha)$

Specific gravity = 2.65

Wind generated waves in water of limited depth have been studied in some detail by Sverdrup, Bretschneider, Thijsse and others. Thijsse's chart for determination of the significant wave height h_w as a function of the fetch F, water depth D and wind velocity V is shown on Figure 1. The parameters on the axes and on the curves are all non-dimensional, with h_w, D and F expressed in m, V in m/s, t in seconds, and g is acceleration due to gravity (9.81 m/s^2).

The upper solid curve on Figure 1 is the limiting maximum wave height corresponding to deep water ($gD/V^2 \rightarrow \infty$). Smaller values of gh_w/V^2 are given for smaller values of gD/V^2. The limiting maximum wave heights given by the Thijsse chart are slightly greater than given by [12] in Chapter 1 (recommended from studies by the Corps of Engineers). The dashed line on Figure 1 can be used to determine the minimum wind duration necessary to develop the indicated wave height.

Bank protection against wave action is based on the fetch length perpendicular to the bank. In the case of a river this refers to the width across the river and not the length along the flow path, which would be much greater. In case of a lagoon, heavier protection may be required on the bank facing the prevailing wind where a higher design wind velocity may have to be considered. Effective fetch should be used in cases where the dimension perpendicular to the wind direction is small (Table 5, Chapter 1).

The California Bank and Shore Protection Manual gives

[5] $$W = 4.62 h_w^3 S_g / [(S_g - 1)^3 \sin^3(\rho - \alpha)]$$

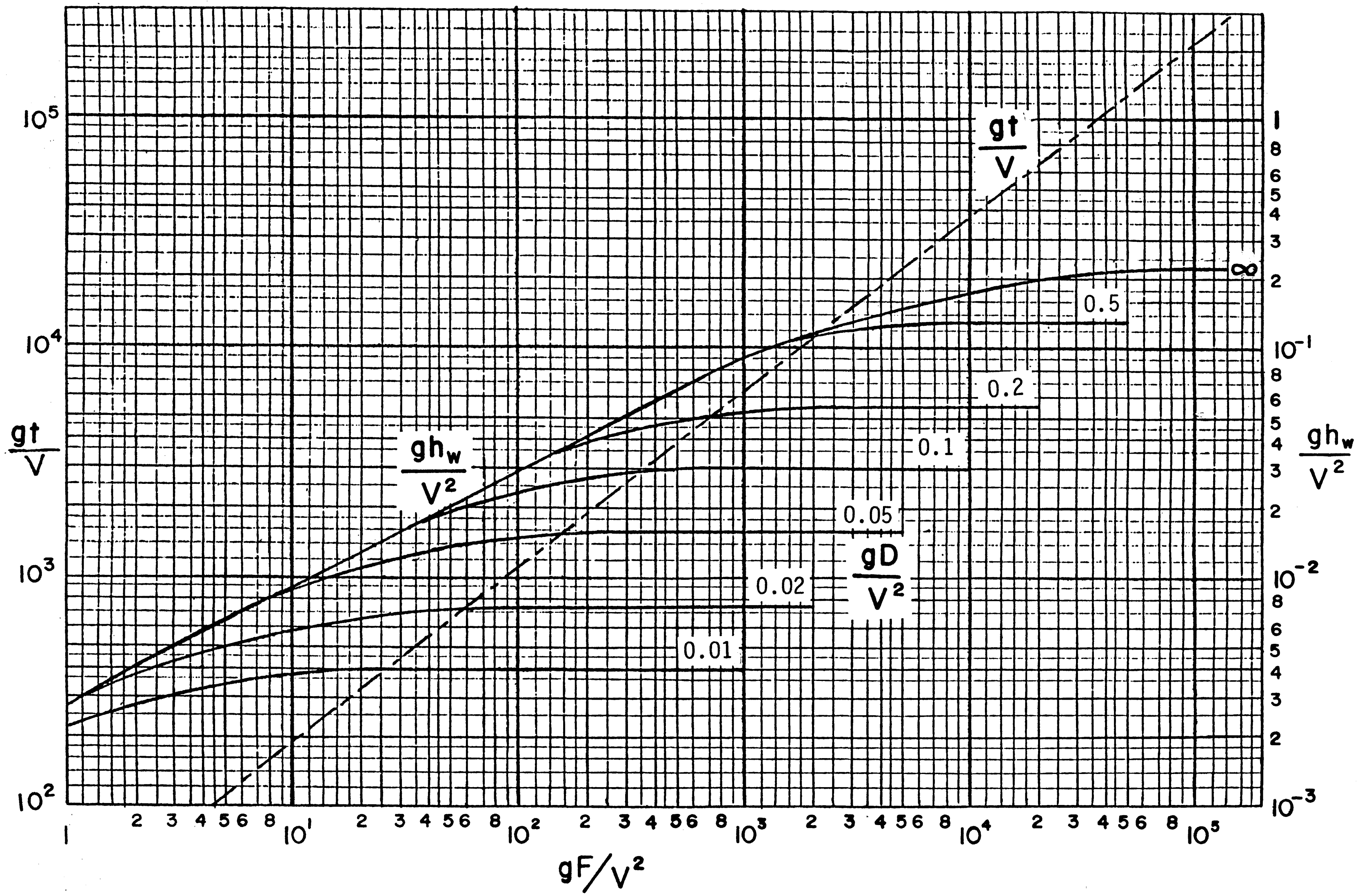

Figure 1. Wind Generated Waves in Water of Limited Depth (after Thijsse)

for the stone weight required to resist wave action, given W in lb and h_w in ft. By applying the 1.2 multiplier, as before, and converting to S.I. units with d_m in mm, h_w in m, $S_g = 2.65$ and $\rho = 70$ degrees, the equation becomes

[6] $$d_m = 377\ h_w/\sin(70 - \alpha)$$

Given $h_w = 0.353$ m and $\alpha = 14$ degrees (for a 1:4 bank slope), [6] gives $d_m = 160$ mm, as an example.

In all cases, whether for protection against current or wave attack, the riprap layer must have a thickness of at least 1.5 d_m, and must be underlain by a suitable bedding layer. A graded gravel is often used, but recently geotextiles have come into common use. Filter fabrics are pervious to water but prevent migration of foundation material. They are strong and apparently unaffected by soil chemicals or temperature.

<u>Example 1:</u>

A river 400 m wide has an exposed earth bank along one side. The side slopes of the bank are 1 vertical to 3 horizontal. The flow velocity adjacent to the bank during flood stage is 3 m/s and a cross wind of 90 km/h may occur blowing toward the bank at the same time as the flood. The river depth is 2.5 m. What size of riprap should be used for bank protection?

The case of simultaneous current and wave action has not been researched to the point where design criteria have been established. However, as a first step, the separate effects of current and wave action may be determined.

From Table 2, the required median stone size to resist current is 232 mm. For waves, $gF/V^2 = 9.81 \times 400/(90000/3600)^2 = 6.28$, and $gD/V^2 = 9.81 \times 2.5/25^2 = 0.0392$. From Figure 1, $gh_w/V^2 = 0.007$, for which $h_w = 0.446$ m. From [6], $d_m = 377\ h_w/(\sin$ $(70 - 18.4) = 377 \times 0.446/\sin 51.6 = 214$ mm.

The stone size for wave action is only slightly smaller than for current. There is no doubt that the 232 mm size would be inadequate if the waves and current occurred simultaneously. At this point an engineering judgment must be made. Generally the dislodging force due to waves acts down the dip line of the slope, whereas for current it is parallel to the bank. The resultant force for both of these forces acting together will be the square root of the sum of the squares. This effect could be accounted for approximately by selecting the stone size as $(214^2 + 232^2)^{1/2} = 315$ mm. This is conservative because the effect of the component of stone weight parallel to the bank slope is common to both waves and current, and is already included in the respective stability equations for stone size. In other words the tangential weight component has been double counted. Accordingly a stone size of 270 mm is recommended.

C. STONE BASIN VERTICAL DROP STRUCTURE

5. Application

The stone basin is an alternative method of handling the problem of nappe impingement below a vertical weir. It is an adaptation of the reinforced concrete vertical drop structure to smaller heads and drop heights. The advantage of the method is speed of construction, absence of the need for skilled labour, and possible utilization of inexpensive local deposits of stone.

In the case of a nappe spilling over a vertical drop into an erodible bed, a scour hole will develop in the bed. As the scour depth increases the depth of the water cushion or stilling pool will likewise increase, until the transporting force of the submerged jet is balanced by the resisting force of the bed particles. With sand size bed particles the ultimate scour depth can easily exceed the height of the drop itself. The vertical part of the structure, which may consist of stacked gabions, a creosoted timber wall on posts, a reinforced concrete wall or even driven steel sheet piling, may fail due to undermining or collapse due to loss of support. This can be prevented by using coarse granular material to limit the depth of scour.

6. Hydraulic Characteristics

The flow over the weir forms a nappe which plunges into the tailwater, as illustrated in Figure 2. The tailwater depth depends upon the discharge and the hydraulic characteristics of the downstream channel. Initially the nappe is deflected horizontally at the initial bed level. At the bottom of the plunging jet the drag forces on the individual stones exceed the resisting forces, so a scour hole begins to form. The scoured material piles up in a ridge or mound in the low velocity area immediately downstream from the scour hole. As the hole deepens the velocity at the bottom of the plunging jet is reduced, and the drag force decreases accordingly. Eventually equilibrium is reached and the scour profile becomes stabilized, similar to the profile shown on Figure 2.

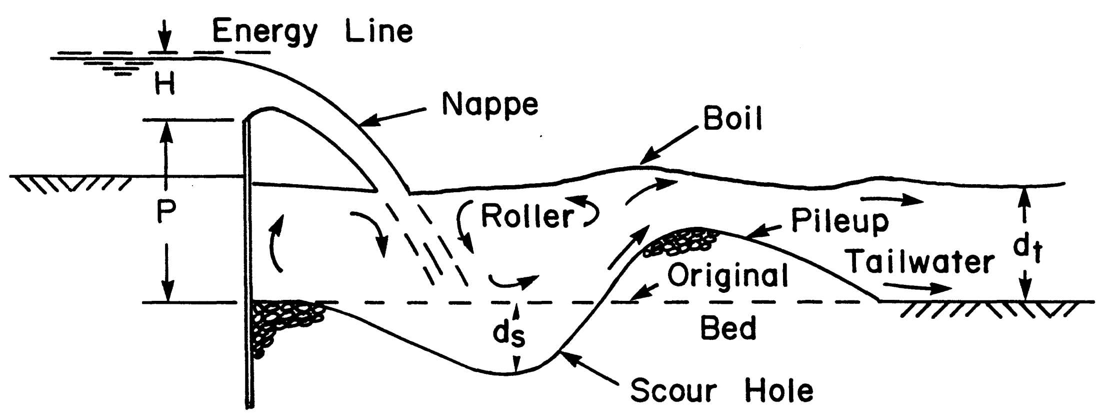

Figure 2. Scour Formation in a Stone Bed

Simple deductive reasoning suggests that the stable scour depth d_s will depend upon the head H, drop height P, tailwater depth d_t, stone size dm and specific gravity S_g, as in

[7] $$d_s = f(H, P, d_t, d_m, S_g)$$

In practice the head would be determined from the unit discharge in the equation

[8] $$H = (q/C)^{2/3}$$

For a vertical weir the value of C should be taken as 1.84. For a free overfall, at critical depth, C should be taken as 1.70.

An increase in head means greater discharge and increased momentum of the plunging jet, causing an increase scour. An increase in drop height means a higher nappe velocity, also increasing scour. A heavier stone (either a larger d_m or larger S_g) will increase the resisting force, decreasing scour, and a deeper tailwater pool will dissipate more of the energy of the jet, also reducing scour. Unfortunately the relationship between these variables is complex and cannot be deduced analytically.

Given a constant $S_g = 2.65$, the remaining variables in [7] may be put in the non-dimensional form

[9] $$d_s/P = f(H/P, d_m/P, d_t/P)$$

The quantitative relationship between these ratios has been established from model tests and is given in Figure 3. The trend of the curves is entirely logical. The $d_s/P = 0$ intercept on the abscissa corresponds to the point where movement will just begin under the imposed H, P, d_t and d_m values. If d_m or d_t is decreased, or H or P increased, the stone will move and a scour hole will develop. The importance of tailwater is also clearly indicated. If the tailwater is raised, then both the height of fall is reduced and the depth of the pool increased, resulting in a marked decrease in scour depth.

The volume of stone removed from below the initial bed level must be deposited above the initial bed level. The actual depth of the deposit may be more or less than the depth of scour, depending upon the tailwater condition. The top of the deposit will always be below the tailwater level at design flow. When the tailwater at the design point is relatively deep, the depth of the deposit may exceed the scour depth; if the tailwater is shallow, the flow over the top of the pile may be supercritical at the design point, and this will limit the height to which the deposit can rise.

Under real operating conditions the bed will be subjected to a range of H/P and d_t/P values as the discharge increases up to the design value, and decreases back to zero. Furthermore, this cycle may be repeated many times. Repeated hydrograph tests on a model showed that a deep pileup above the original bed is undesirable because the mound is eroded after the tailwater drops below the top of the pile during reduced discharge. Since the mound is an integral part of the ultimate stable shape of the scour hole, it must be maintained. This can be achieved by initially placing the top of the stone bed below the downstream channel bed by an amount 2/3 d_s. In order to secure an adequate cutoff the depth of the vertical wall must extend below the bottom of the excavation for the stone. Therefore, it is unlikely that it would be economical to design for a scour depth d_s

any greater than 0.3P. To meet this limitation in the field it may be necessary to increase d_m, decrease P (by using more drops), or decrease H (by using a wider structure).

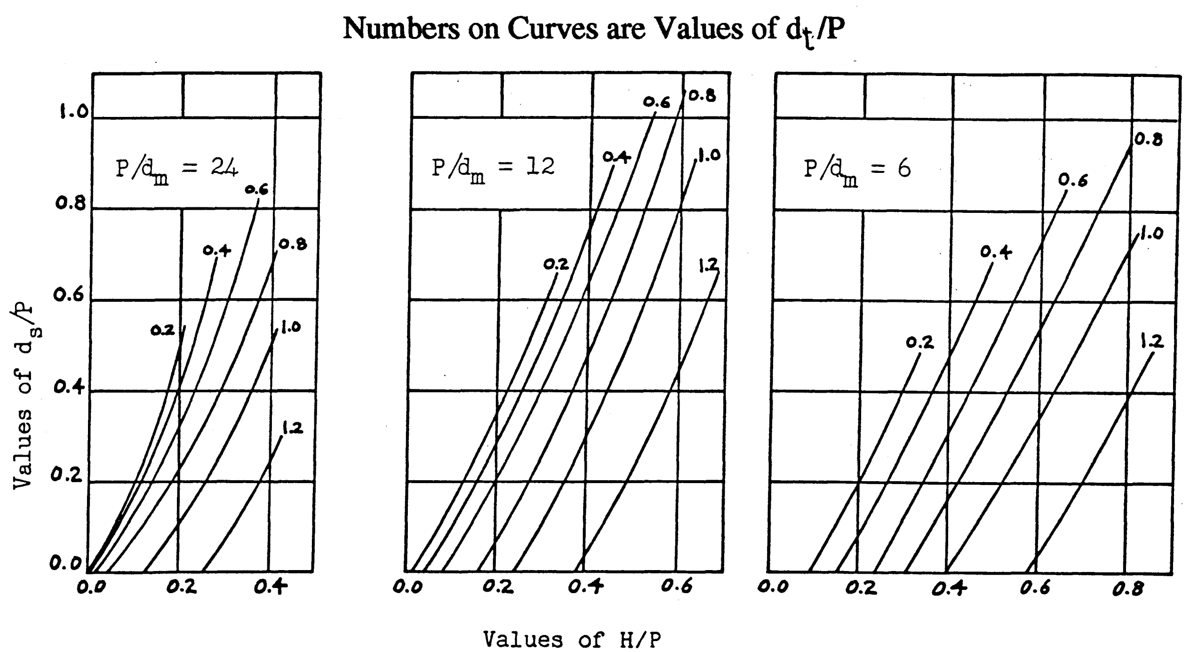

Figure 3. Non-Dimensional Curves for Scour Depth

7. Design

The recommended layout for the stone basin is shown in the definition sketch, Figure 5. A stone layer 1.5 d_s deep, or in no case less than $d_s + 2d_m$, is placed in an excavation at the base of the weir, with the top of the layer 2/3 d_s below the normal downstream channel bed. In determining d_s from Figure 3, it is important to note that P and d_t are measured to the surface of the stone layer as originally placed. The depression of stone below the natural bed serves two purposes, as follows:

1) The increase in d_t results in a decrease in d_s, and a smaller volume of stone may be used.

2) Stone removed from the layer during scour is piled up in a mound downstream. Although it is affected by the changes in the variables, roughly the height of the mound above the original stone bed equals the depth of scour below it. In practice, this mound should not extend greatly above the original channel bed, otherwise the performance will be adversely affected at lower discharges when the tailwater level drops below the top of the mound. The elevation of the mound is reduced to a satisfactory position by using the lowered position for placement of the stone bed.

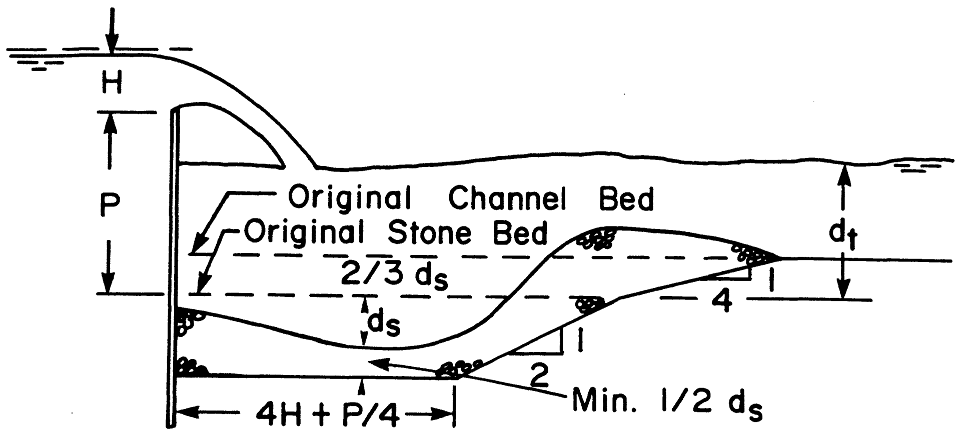

Figure 4. Recommended Design for Stone Basin

The vertical weir portion of the drop structure could be constructed of boards, sheet metal on posts, timber or steel sheet piling, a concrete wall or stacked up gabions. For very low height drops, as in highway ditch checks, creosoted boards supported by posts may be adequate. Driven sheet piling would serve well for intermediate height drops, but the piling probably would have to be tied back with wales if the cantilever length is too great.

Since the data reported is valid only for two-dimensional flow, third dimension effects at the ends of the weir would have to be eliminated by constructing vertical abutment walls. These could be constructed with the same materials and by the same methods as for the weir. The sidewalls would have to extend downstream about twice the base length of the stone bed. Gabions have been used with success to form vertical sidewalls for hydraulic structures.

In order to prevent migration of foundation material under the stone bed, a filter layer should be placed at the boundary of the excavation line. This is particularly important if the underlying material contains fine sand or silt. A graded gravel may be used, but recently various filter fabrics have become commercially available and are popular as a substitute.

The design curves on Figure 3 were determined from experiments on uniform sized material. In practice the dumped stone will normally consist of a range of sizes (i.e. graded). In this respect a factor of safety is automatically incorporated in the design because graded stone is superior to uniform sized stone of the same medium diameter due to the process of armouring. In a verification test using graded material it was observed that as scour progressed the smaller stones were removed and deposited in the mound, leaving the larger stones to form an armour layer at the surface of the scour hole. In effect, the median size of the stones in the surface layer increases as scour progresses, and scour depths are less than indicated by Figure 3.

Again it should be noted that the values read from Figure 3 apply to stone with a specific gravity of 2.65. However, a lower specific gravity can be compensated for by using a larger stone size, producing the same scour depth. The charts can still be used but the indicated value of P/d_m on the chart must be multiplied by $(S_g-1)/1.65$.

D. STONE BASIN FOR OVERHANGING PIPE

8. Application

The overhanging pipe outlet refers to a situation where the tailwater into which the pipe discharges is below the invert of the pipe. Under these conditions the pipe discharge occurs as a free jet with a vertical drop of varying magnitude, depending on the height of the invert above the bed level of the downstream channel. The overhanging pipe may be used to permit free drainage, to accommodatean existing grade line for the pipe, to allow easy access for inspection, to prevent ponding in the pipe which might encourage silting, or to prevent damage to the pipe outlet by floating debris or ice, such as might occur for the case of a pipe discharging into a river.

9. Hydraulic Characteristics

Given a pipe diameter D_o and specific gravity $S_g = 2.65$ for the stone, the non-dimensional scour in a rock bed d_s/D_o will be dependent on the discharge intensity parameter $Q/D_o^{5/2}$, the drop height ratio Z/D_o, the tailwater depth ratio d_t/D_o and the stone size ratio d_m/D_o. In these ratios d_s represents the scour depth below the original bed level, Q is the pipe discharge, Z is the drop height from the pipe invert to the original bed level, d_t is the tailwater depth above the original bed level and d_m is the equivalent spherical diameter of the median size of the riprap, that is the size for which 50% is finer, by weight. Symbolically,

[10] $$d_s/D_o = f(Q/D_o^{5/2}, Z/D_o, d_t/D_o, d_m/D_o)$$

with Q in m^3/s and all other terms in m.

It is evident by simple deductive reasoning that the scour will be increased by any increase in discharge or drop height, and reduced by any increase in tailwater depth or stone size. Because of the number of variables and the complex three dimensional nature of the flow phenomena, the relationship between the variables cannot be determined analytically and must be determined by experiment, as was the case for the stone basin structure.

Experiments by Smith and Johnson were made using a 100 mm clear plastic pipe discharging into a stone bed. Three discharges, four drop heights, three tailwater depths and two stone sizes were selected. These were selected to give $Q/D_o^{5/2}$ values of 1.66, 2.50 and 3.33; Z/D_o ratios of 1, 2, 3 and 4; d_t/D_o ratios of 0.4, 0.8 and 1.2; and d_m/D_o ratios of 0.23 and 0.31. The test data are shown plotted in Figures 5, 6 and 7. For other values of d_m/D_o interpolation or extrapolation will be required in using the figures. If S_g is not 2.65, the actual d_m/D_o must be modified using the multiplier $(S_g - 1)/1.65$ to determine the equivalent value for use in Figures 5, 6 and 7.

Combinations of the foregoing ratios led to 57 separate tests. During each test the rock dune which formed above grade on the downstream side of the scour hole was removed and the discharge continued until scour was stabilized without the dune present. The philosophy of this approach was to find a depth for the riprap at which no movement would occur, from which recommended dimensions for a stable rock basin or plunge pool could be established.

It will be noted that the curves as drawn do not necessarily pass through all the experimental points in every case. It was necessary to draw the curves in such a manner that each curve was consistent with the other curves with the same stone size and with the other curves with the same drop height. In addition the systematic trend for varying d_s/D_o and $Q/D_o^{5/2}$ had to be maintained. The result is that any single curve is governed to a large extent by all the results, and not just the three individual test points for that curve alone. It is always possible to draw a precisely smooth curve through any three points, but as a number of factors could contribute to scatter in individual test points, such a curve would not necessarily be truly representative of the process in every case. The curves, therefore, are the best consistent systematic fit to the collective data.

The results clearly demonstrate that scour increases significantly with an increase in discharge, increase in drop height or decrease in stone size. The scour is decreased with an increase in tailwater depth, but this variable has the least effect, except when Z/D_o is small. For example, if $d_t/D_o = 1$ and $Z/D_o = 1$, then the tailwater level is at the pipe invert. In this case the free drop is largely eliminated, the jet may not plunge to the bed and bed scour may not occur at all.

Normally the stone available in prototype applications will have a wider gradation than the relatively uniform size used on the model. As a result, scour taken from Figures 5, 6 and 7 will be conservative for design.

In order to avoid bank erosion, the pipe must extend beyond the embankment slope. This extension is needed to avoid jet impingement on the slope above the tailwater level, a condition which cannot be resisted with riprap unless the stone is extremely large.

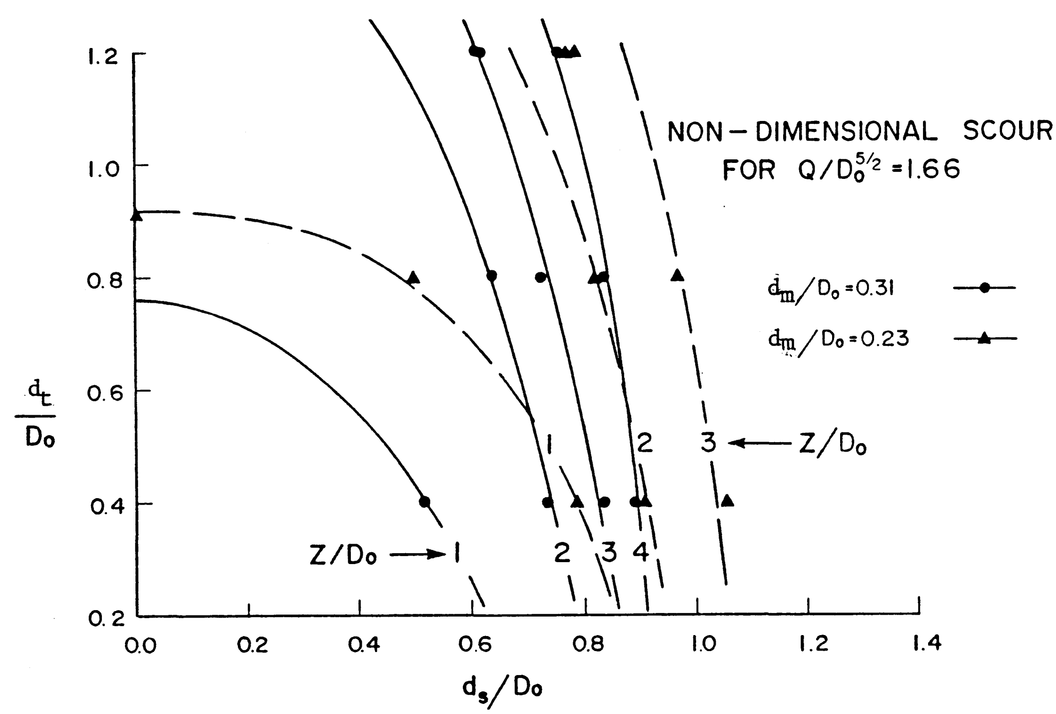

Figure 5. Scour in a Stone Basin Below an Overhanging Pipe - $Q/D_o^{5/2} = 1.66$

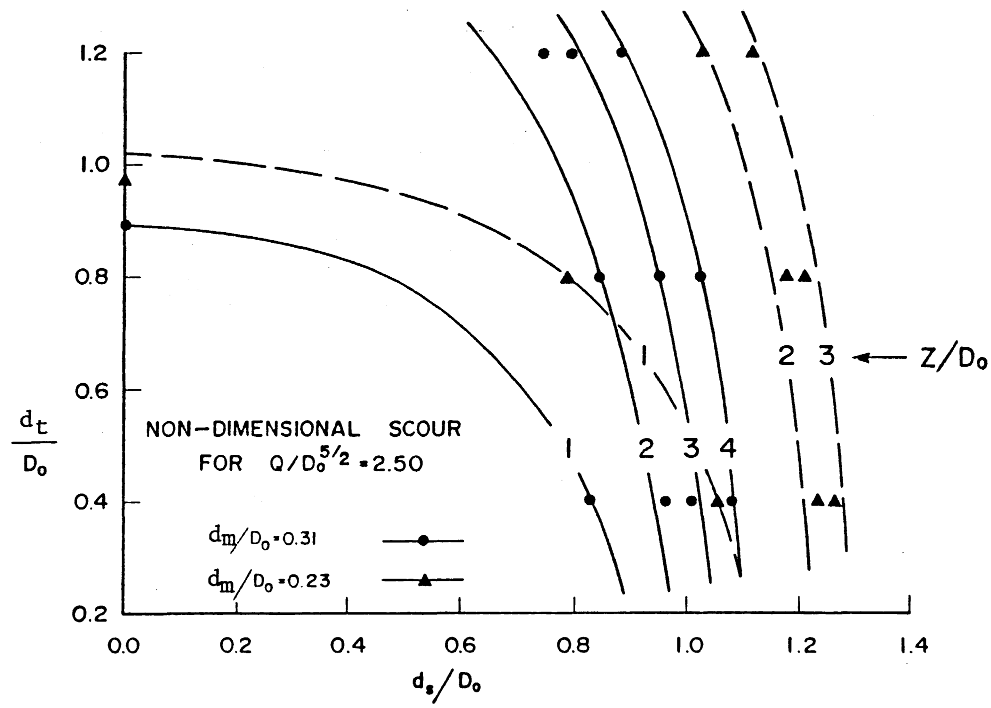

Figure 6. Scour in a Stone Basin Below an Overhanging Pipe - $Q/D_o^{5/2}$ = 2.50

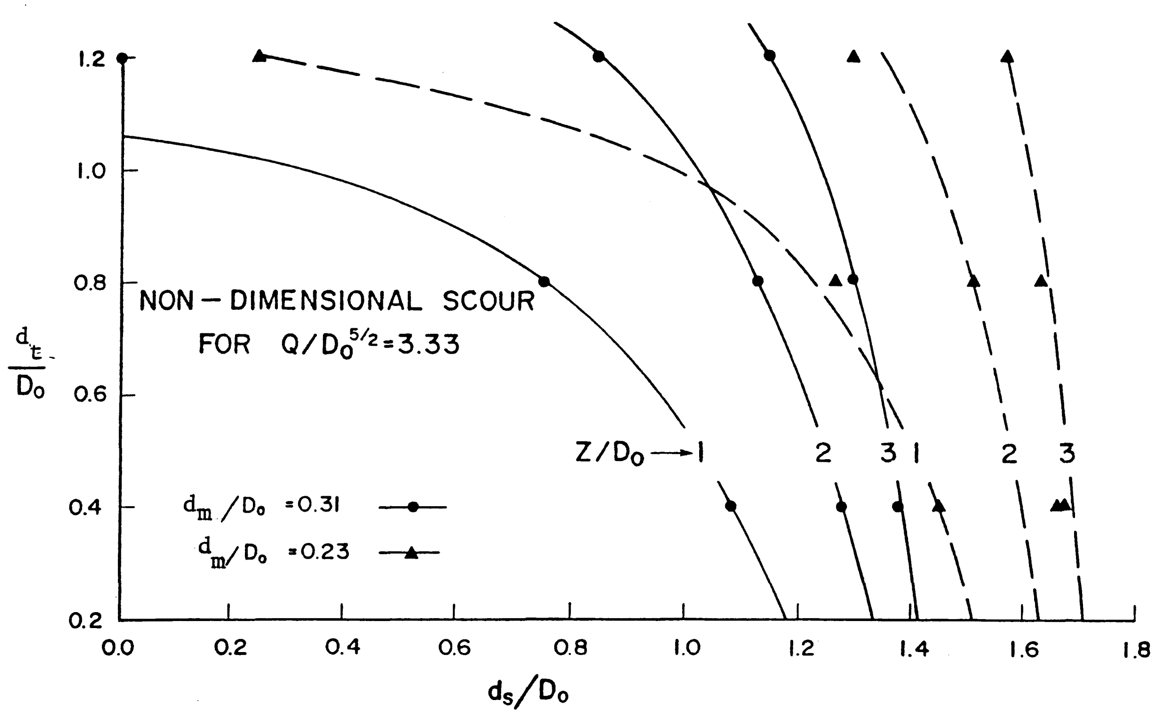

Figure 7. Scour in a Stone Basin Below an Overhanging Pipe - $Q/D_o^{5/2}$ = 3.33

An additional feature clearly indicated on the model was that the base of the rock basin must be depressed below the original bed level of the downstream channel. Formation of such a plunge pool is needed if movement of the stone and exposure of the subgrade is to be avoided. Only if the stone is large, the discharge small or the tailwater deep is it possible to avoid movement of the stone if the basin is not depressed. Naturally a deeper basin is required if the available stone is small in size. A depressed basin of this type conforms in principle to the free jet rock basin developed by the Bureau of Reclamation for jet discharge from valved outlet works. While it would be possible to allow development of a self formed scour hole, as for the vertical drop stone basin, the use of a pre-formed plunge pool reduces the total volume of stone required, and avoids the problem of formation of a large above grade mound of stone displaced from the scour hole.

10. Design

Details for the recommended basin are shown in Figures 8 and 9. Given design values for Q, D_o, Z, d_t and d_m, the depth required for the depressed basin d_s is determined from Figures 5. 6 or 7. The basin is excavated to accommodate a finished riprap surface according to the dimensions in Figures 8 and 9, based on a riprap layer thickness t of at least $2d_m$, plus a filter layer underneath. The filter layer is particularly important if the foundation is fine sand or silt, as this material may by sucked out through the voids in the riprap. The location of the deepest part of the basin, which is Y metres below the pipe invert, in which $Y = Z + d_s$, is X metres beyond the end of the pipe. The value of X may be determined from the nondimensional plot of X/D_o versus Y/D_o shown on Figure 10.

Figure 10 gives the free jet trajectory of the surface streamline for discharge from a horizontal circular pipe. It was found that a good correlation existed between this plot and the position of the bottom of the scour hole. The presence of tailwater had a minor effect, which was to flatten the lower part of the trajectory more than normal. This was allowed for in the design by locating X at the leading edge of the bottom of the rock basin rather than at the centre.

Hydrograph tests showed that there is a tendency to move some of the stones forward or backward in the bottom of the basin when the jet trajectory changes with the discharge as it varies from a low value up to the maximum and back again. To accommodate this natural tendency without disturbing the riprap layer which forms the basin, an extra layer 2 d_m in thickness should be placed in the bottom of the basin. This stone is placed with a horizontal surface at $Z + d_s - 2d_m$ below the pipe invert, and means that the total thickness of the stone in the bottom of the basin is 4 d_m.

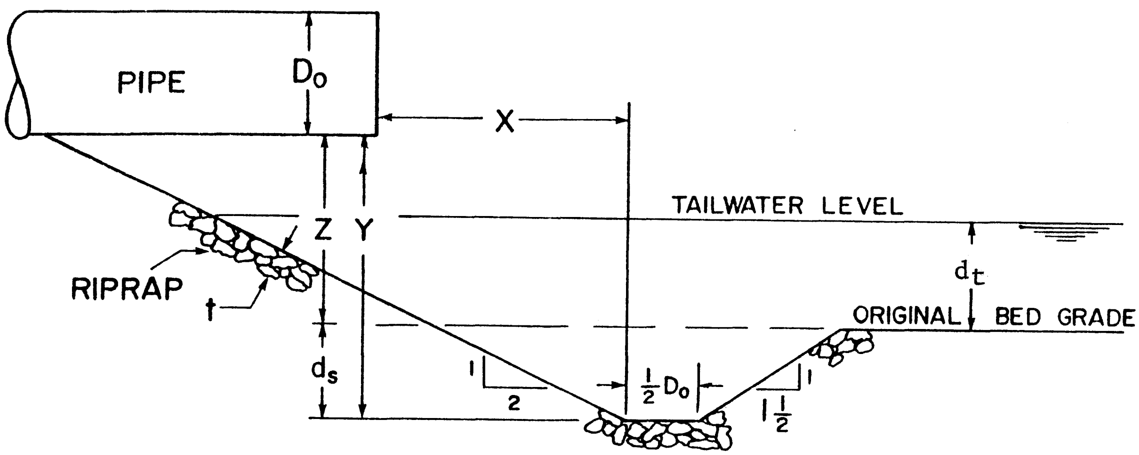

Figure 8. Elevation View of Rock Basin

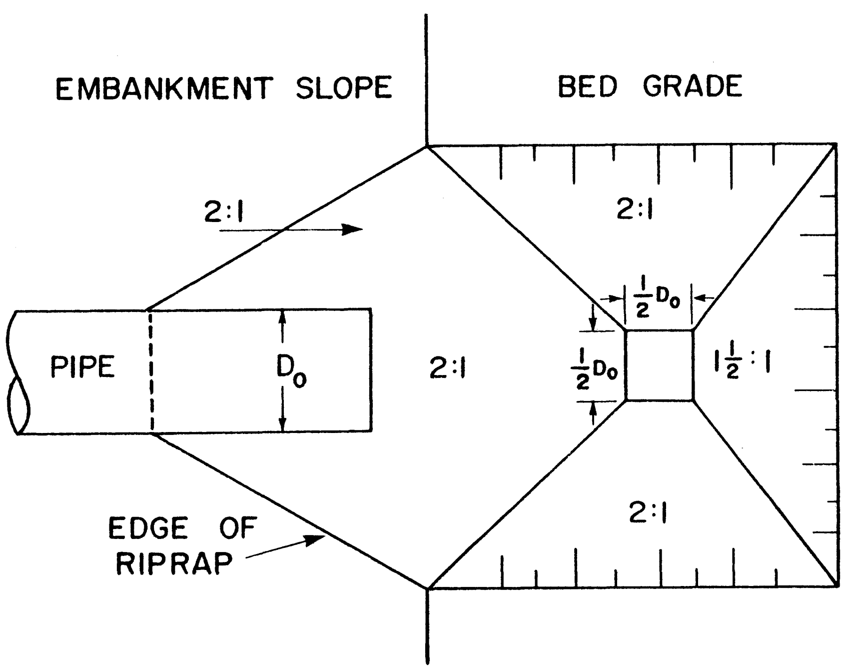

Figure 9. Plan View Layout for Rock Basin

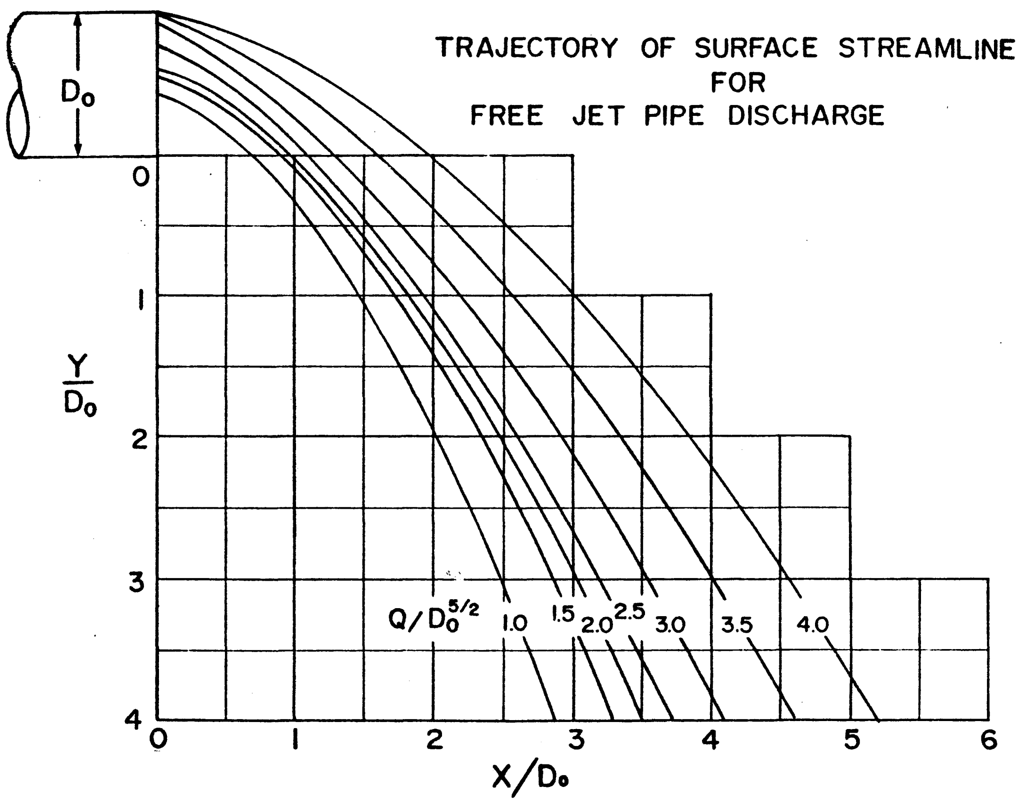

Figure 10. Trajectory of Surface Streamline from a Horizontal Pipe

E. ROCK SPURS

11. General

A spur is a low height earth or rock fill which extends partially into a stream from one bank. Spurs are used in river engineering for many purposes. For example, multiple spurs may be used to inhibit attack on a roadway running along the edge of a stream, or may be used to stabilize the position of the thalweg for navigation. A single spur may be used as shelter protection for boat docks, or may be used to direct the current to the opposite bank of the stream to enhance depth for a pumping plant intake.

In cases where the crest of the spurs is above the maximum flood stage, the discharge will always pass around the end of the spur and the nose of the spur is the only area subjected to significant attack by the flowing current. However, for reasons of economy of construction and to minimize nose scour and backwater effects, a low height spur may be used. Such a spur will be overtopped during flood stage and must be designed to resist the erosive forces associated with this overflow.

It is important to note that overtopping of the type considered is not accompanied by supercritical jet discharge on the downstream slope of the spur. In fact, if this were to occur the stability problem would be particularly severe and very flat downstream slopes would be required. When overtopping is produced by the natural increase in stage accompanying an increase in discharge, there will be a small drop in water elevation between the upstream and downstream side of the spur due to the backwater effect produced by the spur. This difference in water level will increase with increasing discharge until overtopping occurs. When overtopping first commences, the tailwater level will be slightly below the crest level of the spur but the downstream slope will be submerged. The difference in water level will tend to decrease as the spur becomes deeply submerged.

Normally the river flow will be subcritical through the opening at the end of the spur, but supercritical flow may occur on the spur crest under certain conditions. For example, when the elevation of the tailwater above the crest of the spur is less than two-thirds of the upstream head, the flow on the crest will be supercritical. This condition will occur when the spur is first overtopped, but the corresponding overflow discharge and velocity will be relatively small. The flow will become subcritical as the tailwater rises and submerges the spur. The greatest velocity will occur for the condition which produces the greatest difference in water levels across the spur.

12. Overflow Velocity and Stone Size

Single spurs subjected to overflow may be constructed entirely of rock fill, or if rock is in short supply, an earth fill with rock riprap on top may be used. The stone size must be sufficient to be stable under the influence of the overtopping flow.

Referring to the definition sketch on Figure 12, the maximum velocity, which occurs at the point of minimum depth, is located near the downstream edge of the crest. By analogy to the contracted area method of discharge measurement proposed by Kindsvater this velocity may be written as

[11] $V = [2g(h + \alpha_1 V_1^2 /2g - h_f - h_s + R)]^{1/2}$

in which V is the flow velocity in m/s, h is the depth of the upstream water level above the crest in m, V_1 is the velocity of the approach flow in m/s, α_1 is the velocity head correction factor for the approach flow, h_f is the head loss in m between the upstream point of measurement and the position where V is calculated h_s is the depth of the tailwater level above the crest in m, equal to (h_d-P) and R is the rise in water level (recovery in head) in the jet expansion region downstream from the crest.

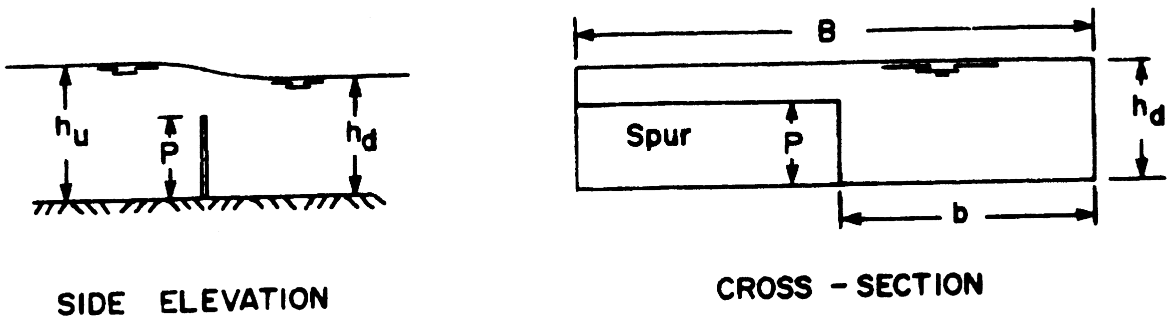

Figure 12. Definition Sketch for Flow Over a Submerged Spur

For practical design α_1 may be taken as unity and h_f may be neglected. These assumptions are compensating in nature so the error, if any, is small. Since there will be a separation zone on the downstream side of the spur, the eddy loss will be large and the recovery will also be small. If R is assumed to be zero, and H is substituted for $h + V_1^2$ /2g, then [11] may be written in a simplified form as

[12] $V = [2g(H - h_s)]^{1/2}$

Since for a given specific gravity, stone sizes required for stability depend upon the velocity squared (See Section 20 of Chapter 10, Culvert Hydraulics), that is

[13] $d_m = kV^2$

then [12] and [13] may be combined to yield

[14] $d_m = C\,(H - h_s)$

indicating a direct relationship between stone size and difference in water level.

Dick performed overflow tests on rockfill using four different sizes of stone ranging from 7 to 22 mm, and for a range of values for spur height, top width and side slopes. For a given set of conditions, the value of h_s was reduced until movement of the stone on top of the spur was observed. It was found that the spur height, top width and side slope had minimal effect, and the relationship in [14] was verified with C = 2/3 (for a stone specific gravity of 2.65). Hence, for design, with d_m, H and h_s all in m.

[15] $d_m = 0.667\ (H - h_s)$

15. Backwater Effect

Before [15] can be used, it is necessary to determine the backwater effect, $h_u - h_d$, produced by the spur. The downstream depth h_d would correspond to the value for the normal stage - discharge relationship for the downstream channel, and h_u is the upstream depth. After h_u is determined, H in [15] may be solved from

[16] $H = h_u + V_1^2/2g - P$

It may be deduced that

[17] $h_u = f(b, B, P, h_d, Q, g)$

in which b is the width of the opening between the end of the spur and the bank, B is the natural width of the channel, P is the spur height and Q is the total discharge. Rearranging [17] in non-dimensional form

[18] $h_u/h_d = f(b/B, P/B, h_u/P, \mathbf{F}_2)$

in which $\mathbf{F}_2$ is the downstream Froude number, given by

[19] $\mathbf{F}_2 = Q/(Bh_d\sqrt{g\,h_d})$

In [18] the term b/B may be replaced by the contraction ratio m, in which m = 1 - b/B. The term h_u/P is included in [18] because if $h_u/P \leq 1$, all the flow goes around the end of the spur, and if $h_u/P > 1$ then the flow is a combination of flow around the end and over the top. Thus, at $h_u/P = 1$ there will be a change in the character of the flow. Work by Oak and Smith at the University of Saskatchewan showed that, friction effects would be negligible and the flow pattern would be governed by the geometry of the opening, provided that b/B did not exceed 0.6 and P/B was not smaller than 0.1. The flow would contract laterally to about two-thirds of the width of the opening, but would decrease for shorter spurs or shallower flow, resulting in a decrease in the backwater.

By deleting P/B and omitting h_u/P for separate consideration, then [18] may be simplified to

[20] $h_u/h_d = f(m, \mathbf{F}_2)$

and written in explicit form as

[21] $h_u/h_d = 1 + C\, m^x\, \mathbf{F}_2{}^y$

Equation [21] shows that if there is no spur, for which m = 0, then $h_u/h_d = 1$, or if there is still water, for which $\mathbf{F}_2 = 0$, then again $h_u/h_d = 1$, thus explaining the need for the form of the equation. Equation [21] may be written as

[22] $h_u - h_d = C\, m^x\, \mathbf{F}_2{}^y\, h_d$

in which $h_u - h_d$ represents the increase in upstream water level due to the presence of the spur. Deductive reasoning suggests that the effect of m and $\mathbf{F}_2$ on h_u should be exponential. For example, if $\mathbf{F}_2$ is doubled (by doubling Q) but b, B and h_d are unchanged and there is no overflow, then the flow velocity through the contraction must be doubled and the velocity head (and head loss) quadrupled. The difference between h_u and h_d should then be quadrupled as well, suggesting the theoretical value for y should be 2. Similarly, if m is doubled, for example from 0.333 to 0.667, then again the flow velocity through the contraction would be doubled and the differential head quadrupled, suggesting x = 2 as well. The actual values of the exponents could be modified and must be determined by experiment.

Once overflow occurs, the rate of backwater rise will rapidly decrease with increasing depth because of the additional large area made available to pass the discharge. In this case, the value of C will change with h_d/P to reflect the reduced proportion of the total flow being passed through the opening. Thus, C value would be expected to decrease as h_d/P increases.

Flow conditions are extremely complex during simultaneous overflow and end flow. The flow regime over the weir is similar to the condition for a submerged weir, with an undulating surface jet and horizontal axis eddy under the jet. The flow at the end of the spur results in lateral jet contraction and a vertical axis eddy. These two eddies join and mix downstream from the end of the spur, producing locally a rough, unsteady, and non-level water surface.

Data plots were made for h_u/P versus h_d/P as a function of $\mathbf{F}_2$ for a range of h_d/P from 0.4 to 1.4. One plot was made for each b/B value - 0.4, 0.5 and 0.6. The best fit linear lines for all three plots in the region $h_u/P \leq 1$ was found by curve fitting, giving

[23] $h_u/h_d = 1 + 20m^{1.8}\, F_2{}^{1.8}$

For the region $h_u/P > 1$, the rate of increase in backwater decreases rapidly with increasing values of h_d/P. This occurs because h_u must approach h_d as the submergence of the spur becomes very large. In this non-linear region, the best fit curves are given by

[24] $h_u/h_d = 1 + Cm^{1.6} F_2^{1.5}$

with C being dependent upon h_d/P. The change in the exponents is a reflection of the change in the character of flow. It was found that C in [24] equaled 3.9 when $h_d/P = 1$, and decreased for values of $h_d/P > 1$ in accordance with

[25] $C = 3.9 (h_d/P)^{-2.4}$

and hence, for $h_d/P \geq 1$

[26] $h_u/h_d = 1 + 3.9 (h_d/P)^{-2.4} m^{1.6} \mathbf{F}_2^{1.5}$

Equations [23] and [26] are plotted on Figure 12, for the case of $b/B = 0.4$ ($m = 0.6$), along with the experimental data points. Similar plots were made for $b/B = 0.5$ and 0.6, and showed equally good agreement.

Figure 12 shows that for a contraction ratio of 0.6 and a Froude number of 0.2, an h_d/P ratio of 0.7 is sufficient to produce an h_u/P ratio of 1.0, indicating a large backwater of 0.3 P. However if h_d/P is increased to 1.3 the h_u/P ratio would only be 1.4, indicating a backwater of 0.1 P.

Anything that increases the area of flow around the end of the spur will reduce the velocity, velocity head, head loss, and backwater h_u-h_d. This effect is shown directly by [23], in that a short stub spur where m=0.1 (i.e. b/B=0.9) would produce a backwater equal to only 0.055 times the backwater that would be produced with m=0.5. However, there are other factors, not accounted for in [23] or [26], which also can reduce the backwater. These equations were developed for a rectangular plate spur on a rigid bed, with $P/B \geq 0.1$ and $b/B \leq 0.6$, and therefore the h_u-h_d values given by the equations are upper limiting values. On a sand bed river channel any bed scour in the contraction will increase the flow area and decrease the average velocity. A fill type spur would normally have a rounded nose and a side slope of 1.5:1 to 2:1. The rounded nose would reduce the lateral contraction of the jet flow through the opening at the end of the spur, and the side slope would result in an increase in surface width as the depth increases, further increasing the flow area. Finally for small values of P/B, such as for a 2m high spur on a 100m wide channel, friction effects dominate the flow pattern, greatly reducing the lateral contraction of the jet flow around the end of the spur. At the limit the contraction may be largely eliminated, which would be the equivalent of reducing the contraction ratio by one-third and the backwater h_u-h_d by one-half.

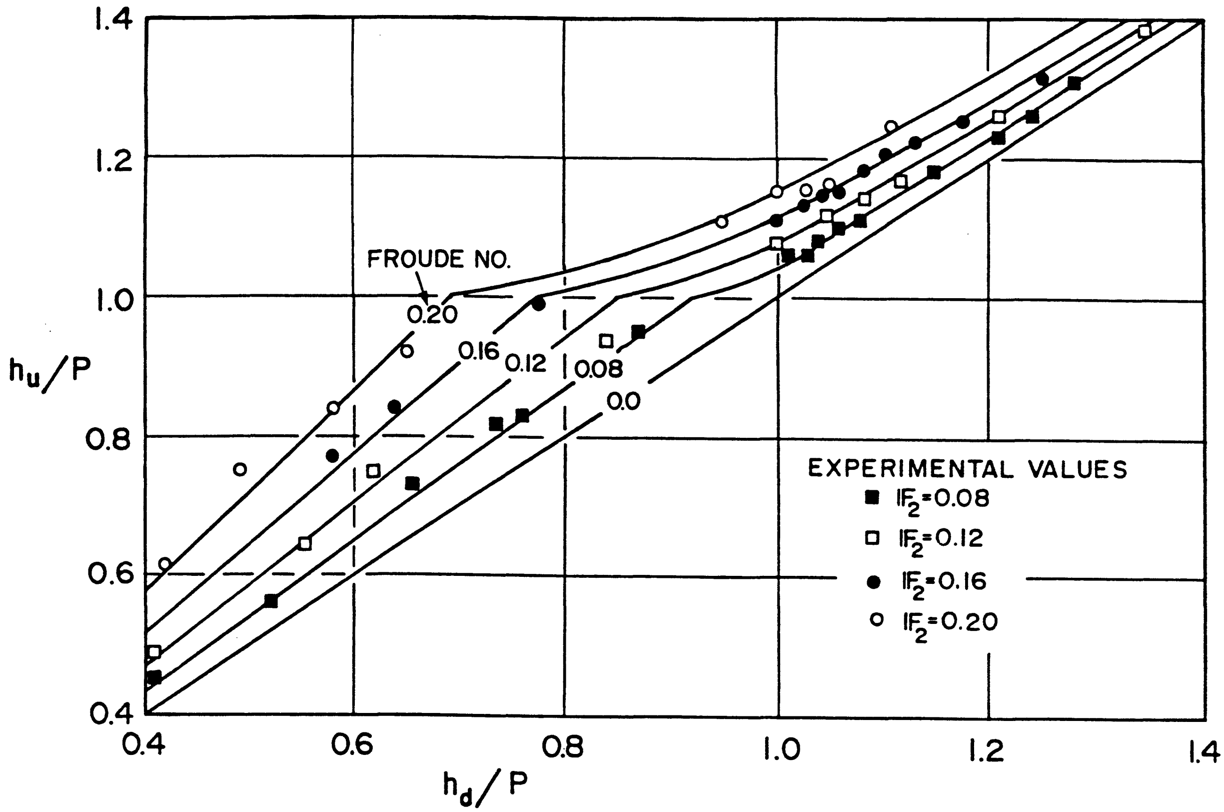

Figure 12. Data Plot for Rectangular Plate Spur with b/B = 0.4

F. GABIONS

14. Description and Application

Gabions are wire mesh containers for small stones. The containers may be box shaped, mattress shaped or sack shaped. The box shaped gabions may be stacked up to form a vertical wall for a weir or retaining wall. Mattress shaped gabions are used to line the bed or banks of a drainage channel, river or canal. Sack gabions may be used like riprap for river closures, spurs or bank protection.

Gabions were first used in Italy almost 100 years ago, and the design has been pioneered by Maccaferri. Modern day gabion mesh consists of a soft annealed 3 mm diameter steel wire, zinc coated for corrosion resistance, and with hexagonal openings. The wire is double twisted for extra strength. Stone sizes must be larger than the size of the hexagonal openings, but this allows use of stone sizes that would be much too small for stability if placed as loose dumped stone, such as described for the structures in Parts B, C, D and E. The hexagonal openings are approximately 83 mm by 114 mm. Typical stone sizes may range from 100 to 200 mm. The effective weight for stability becomes the weight of the whole unit, as opposed to the weight of an individual stone. In addition the gabion boxes or mattresses may be wired together in place, giving a further homogeneity to the structure. The flexibility of the structure allows it to follow foundation settlement without loss of strength.

When vertical walls are built up of box units, they are designed as gravity structures. The same type of load and stability analysis as used for gravity dams, as discussed in Chapter 1, may be applied. An example of a gabion vertical drop structure is shown in Figure 13.

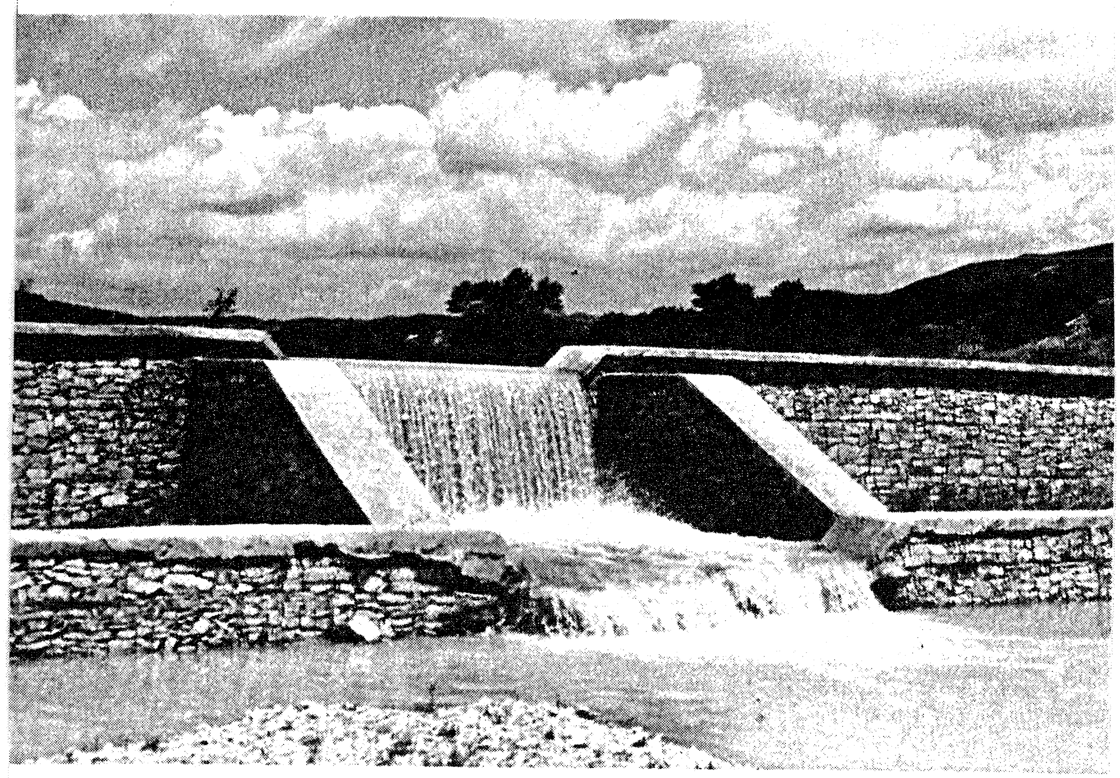

Figure 13. Gabion Box Unit Drop Structure

Gabions are resistant to wave attack as well as current, but when used for low height water retaining structures, the question of seepage, uplift and piping must be addressed. A porous water retaining stone structure, placed directly over an earth foundation, would result in progressive loss of foundation material due to high rates of seepage along the contact line, and could eventually produce failure of the structure. Hence, the requirements for water barriers and cutoffs are the same as for any rock fill dam. An upstream concrete cover or clay liner may be used for this purpose. If clay is used it will be necessary to have a filter between the clay liner and the stone filled gabion. The weakness of gabion structures is the dependency on the integrity of the wire. Abrasion due to moving sediment, or repeated stone movement due to wave action, could remove the zinc coating and allow the wire to corrode. Streams with large ice floes during spring breakup could result in wire breakage due to impact. However, breakage of a single wire is not serious as the double twist will prevent unraveling of the unit, but a large breach would allow the undersize stones to be exposed and removed. Gabions should be used with caution in these situations.

It is recommended that the gabion structures be inspected periodically for wire breakage. Repairs to exposed areas can be effected quite simply by looping a wire across the break and twisting it with ordinary pliers.

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PROBLEMS

1. A lagoon for precipitating fly ash at a large thermal plant consists of a rectangular plot surrounded by an earthen dike. The lagoon is 1000 m to a side and will contain water 3 m deep. The side slopes are 1 vertical to 3 horizontal, and the design over water wind velocity is 100 km/h. What size of stone riprap should be used on the side slope of the dike to resist wave action?

2. During flood stage a river discharges 2800 m^3/s in a channel which is 400 m wide and flow 3 m deep. A narrow earth fill has been placed along the river's edge to accommodate a bicycle path and hiking trail. The fill has a side slope of 1 vertical to 4 horizontal. What size of riprap should be used where the channel is straight, and where the channel has a bend?

3. A 3 m sheet pile wall is driven across a stream leaving 1.2 m projecting above the original natural stream bed. On the downstream side of the wall a stone basin is placed using a median stone size of 0.25 m. The stone basin is placed with the surface 0.3 m below the stream bed, and thick enough to permit a 0.45 m scour hole to develop in the stone. If the maximum flow over the wall will be 0.88 $m^3/s/m$, what minimum depth of tailwater above the stream bed is required?

4. An overhanging pipe 800 mm in diameter discharges 1.43 m^3/s into a receiving channel. The invert of the pipe is 1.6 m above the bed of the channel, and the tailwater depth will be 0.64 m above the bed of the channel. There is an abundance of field stone with a median diameter of 180 mm available near the site. Establish the dimensions for a stable rock basin below the pipe.

5. Discuss the design considerations in the use of box gabions for construction of a vertical drop structure.

CHAPTER VIII

FLEXIBLE CHANNEL LININGS FOR EROSION CONTROL

A. FLEXIBLE LININGS

1. Definition and Classification

If the material comprising the wetted perimeter of a channel is different than the material through which the channel is located or excavated, the channel is said to be lined. For example, the channel may be lined with clay, vinyl rubber sheets, concrete, asphalt, gravel, riprap, vegetation or many other materials. Lining is normally undertaken for one of two reasons - either to minimize seepage losses or to prevent erosion of the channel. In some cases, both of these objectives can be achieved with the same lining, such as with concrete or asphalt. Clay linings are used to reduce seepage losses, but will not prevent erosion. Gravel, riprap and vegetation are used to increase the erosion resistance but will not prevent seepage losses.

Linings may be classified as rigid or flexible. Rigid linings generally have a fixed shape and are inerodible, but they are relatively costly to construct and are subject to deterioration by cracking from movements associated with settlement, swelling pressures and freeze-thaw cycles. For this reason, rigid linings are not preferred for drainage ditches for lands and roads.

Flexible linings for erosion control will follow foundation movements due to settlement, swelling or freeze-thaw without loss of integrity. These linings are also porous and will allow seepage inflow or outflow as required. Costs for materials, quality control and construction are generally lower for flexible linings because often locally available materials and unskilled labor can be used. However, flexible linings have lower upper limits for erosion resistance than rigid linings.

Flexible linings may be further classified as temporary or permanent. Temporary linings are intended for short duration periods of only 1 or 2 years, and include such materials as straw, wood fiber, willow mats, sack cloth and netting. In some cases, some of these materials may be sprayed with asphalt to increase bonding and durability. More permanent linings are used where a service life of 10 to 30 years is required, and include materials such as gravel, riprap, wire-enclosed riprap (reinforcing mesh or gabion mattresses) or grassed channels. Gravel and riprap are relatively inert materials and can be used in continuous contact with water. They are suitable for bank and shore protection on rivers and lakes as well as land drainage channels which operate for extended periods each year. Stone structures have even been used on irrigation projects where flows of several months duration occur each summer. Grassed channels are suitable for short duration and infrequent flows only, because the vegetation requires a significant period of growing conditions in order to maintain the density and integrity of the stand.

2. Methods of Analysis

In the case of rigid linings, such as reinforced concrete, the size, shape and slope of the channel can be independently selected to suit the situation. Velocities and depths may then be calculated directly using an appropriate flow equation. Flow equations from Chezy, Hazen-Williams, Darcy-Weisbach and Manning have been used at various times. In fact, all of these equations would give identical results if the resistance coefficients were selected to be compatible. The Hazen-Williams equation is still popular in the water supply industry, but the Manning equation is used almost exclusively for open channel flow problems involving rivers, canals and drainage ditches. Manning's equation may be written as:

[1] $$V = \frac{R^{2/3} S^{1/2}}{n}$$

or

[2] $$Q = \frac{AR^{2/3} S^{1/2}}{n}$$

in which V is the velocity in m/s, R is the hydraulic radius in m, S is the channel slope, Q is the discharge in m^3/s, and n is Manning's flow resistance value.

In the case of flexible linings, which are erodible, the size, shape and slope of the channel cannot be independently selected. Thus, in addition to a flow equation, a stability equation is required. The purpose of the stability equation is to insure that the erosive forces on the bed and banks of the channel are less than the erosion resistance of the lining on the bed and banks. This condition places certain limitations on the size, shape and slope of the channel.

Channel stability analysis by the tractive force method, originally developed for coarse non-cohesive materials, has been extended to cover a broad range of conditions, including cohesive material, grassed channels and gabion mattresses. This method is adopted for purposes of the present stability analysis for flexible linings.

B. TRACTIVE FORCE METHOD

3. Historical Development

Most criteria for stable channel design originated from the need to construct irrigation canals and ditches which would remain basically in the same form as constructed (i.e. maintenance free). In this context, a stable channel was defined by Lane as "--an unlined earth channel (a) which carries water, (b) the banks and bed of which are not scoured objectionably by the moving water, and (c) in which objectionable deposits of sediment do not occur." In order to meet requirements for stability, there are limitations on the size, shape and slope of the channel. These limitations are a function of the characteristics of the material through which the channel is excavated, the magnitude of the discharge and the quantity of sediment being transported.

Much of the early work on stable channel design originated in India and Pakistan where extensive irrigation works were developed in the 19th century. Criteria for channel design were developed by Lindley and Lacey, culminating in the 'regime method' pioneered by Blench. The criteria were based on observations made on stable and unstable rivers and canals, and were largely empirical (or semi-empirical at best).

Studies on stable channel concepts were also carried out in Egypt by Leliavsky, in Europe by du Boys, Schoklitsch and others, and in the United States by Fortier and Scobey, Lane, Carlson, Vanoni, Simons and Albertson, and Leupold and Maddock. The Simons and Albertson method is a commonly used method in North America for channels excavated through alluvium consisting of gravels, sands, silts or clays. Like the regime method, the Simons and Albertson method is empirical, being based on data taken from 113 study reaches of canals in both India and the United States.

The shearing force which the flowing water exerts on the channel boundary is referred to as the tractive force. That this force could be the basis for canal design was suggested by Schoklitsch in 1937. In 1950, the U.S. Bureau of Reclamation undertook investigations on canal design. The results were reported by Lane in 1955, and this resulted in a major advance in adoption of the tractive force method as an option for the design of stable channels.

The tractive force method was a natural spin-off of the method of permissible velocities. The method of permissible velocities is based on the well known observation that silt or fine sand erodes more easily than gravel or pebbles, so steeper slopes and higher velocities can be used for coarser material. However, it was observed by Moritz in 1915 that even for similar material "-- small canals erode at a lower mean velocity than large canals." Accordingly, corrections for the effect of depth of flow on permissible velocity were introduced by Fortier and Scobey in 1926 and in Russian literature in 1936. These corrections basically account for the fact that for equal mean velocity the boundary shear will be larger in a channel with a shallower depth of flow.

The tractive force method was promoted by Lane in his landmark paper on "Design of Stable Channels" published in 1955. An attractive feature of the method is that it is rational. Shearing stresses can be calculated analytically, and for non-cohesive material particle stability also can be analyzed using a mathematical approach. The method has been extended to include cohesive materials, vegetation and mattresses, but for these materials considerable empiricism was required.

4. Shear Stress in a Channel

The basis of the tractive force method is that the de-stabilizing force acting on a channel boundary can be related directly to the shear stress on that boundary. It has been argued, particularly for non-cohesive material, that lift and drag forces on a particle are at least as important as shear forces in producing instability and erosion of the boundary. There is little doubt that lift, drag and shear are all present, that in turbulent flow, these forces are highly transient (varying continuously about some mean value), and that instability results from the collective action of all these forces. However, lift, drag and shear are each a function of the velocity squared, so even if instability is related to shear alone for purposes of analysis, the effect of the other forces is accounted for implicitly. This approach is a case of the ends justifying the means.

The shear force per unit of channel length acting on the perimeter of any channel, for the case of uniform flow at normal depth, is equal to the component of weight of water acting in the direction parallel to the bed of the channel. This may be expressed as

$$[3] \qquad F_f = \gamma A \sin\theta$$

in which F_f is the shear force, γ is the unit weight of water, A is the cross-sectional area of flow, and θ is the slope angle of the channel. This force is resisted around the perimeter of the channel, so the shear stress, τ, equals F_f divided by the wetted perimeter P. Further, since $\sin\theta = \tan\theta$ to 3 decimal places for flat slopes (i.e. less than 10%), $\sin\theta$ may be replaced by the channel slope S, and [3] becomes

$$[4] \qquad \tau = \gamma R S$$

in which R is the hydraulic radius (replacing A/P). In [4], τ is the average shear stress around the perimeter of the channel. This equation is exact and applies to all channels regardless of the composition of the boundary.

The actual shear stress at a point on the wetted perimeter may be more or less than the average value given in [4], depending on the location. For example, at the water's edge on the side slope where the depth and velocity approach zero, the shear stress also approaches zero. The maximum shear stress in the cross-section will normally occur at the bed in the center of the channel, approaching a value

$$[5] \qquad \tau_b = \gamma y S$$

in which τ_b is shear stress on the bed and y is the depth of flow. This stress is greater than the value shown in Equation 4 because the hydraulic radius is always less than the depth.

A typical distribution of shear stress around the perimeter of a trapezoidal channel is shown in Figure 1. It is to be noted that the shear stress is smaller on the sides than the bed, is lower at the corners, and is zero at the water's edge. This variation reflects the nature of the velocity distribution in the channel, in that the steepest velocity gradient (and highest shear) occurs at the center of the channel near the bed.

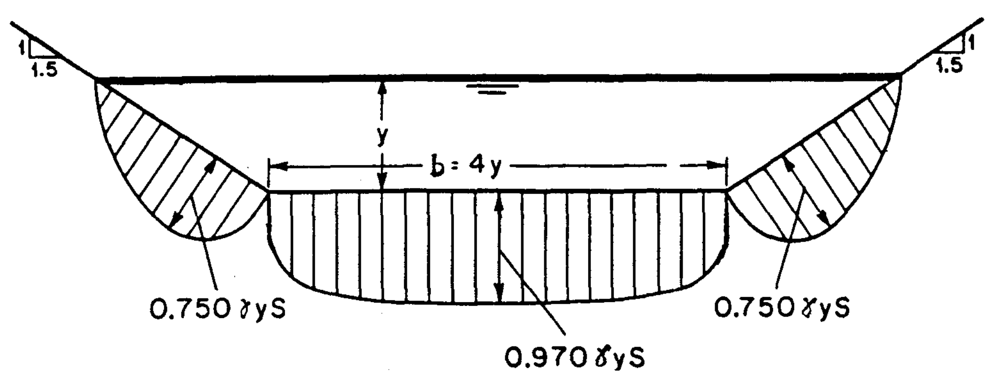

Figure 1. Shear Stress Distribution for a Trapezoidal Channel

The shear stress distribution is affected by the shape of the channel but not the size. As the width to depth ratio, b/y, decreases, the maximum shear stress decreases on the bed and increases on the sides. For example, t_b is 80% of γyS for a b/y = 1, 90% of γyS for b/y = 2, 97% of γyS for b/y = 4, and 100% of γyS for a b/y $\geq$ 8. Usually the bed is designed for a shear stress of 100% of γyS, as given by Equation 5. The effect of channel shape on the maximum shear stress on the sides is shown on Figure 2.

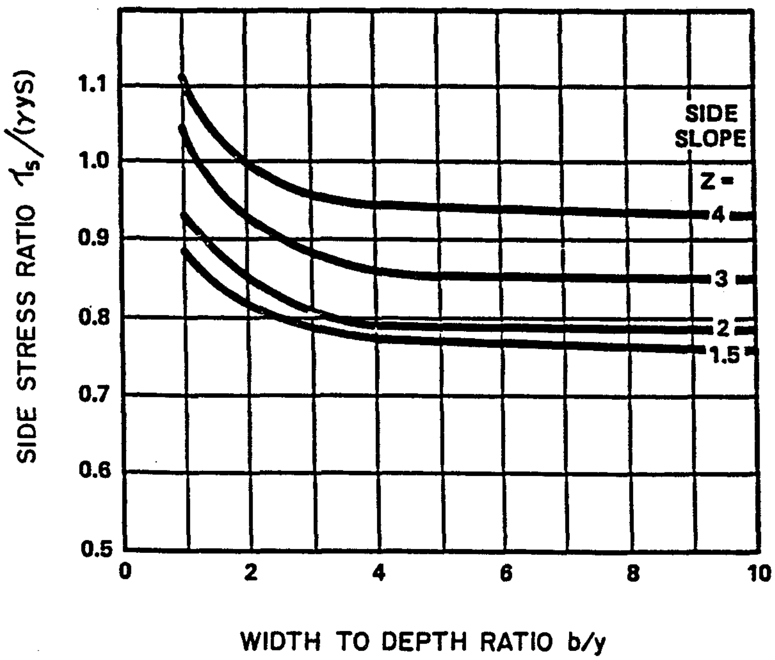

Figure 2. Side Shear Stress Ratio for Trapezoidal Channels

The shear stresses given by [5] for the bed and Figure 2 for the sides are values for straight channels. If there is a bend in the channel, the shear stress will be increased in the vicinity of the bend. The degree of increase will depend upon the sharpness of the bend, in that sharp bends produce stronger secondary flow, with higher flow velocity closer to the boundary. The higher velocity normally occurs at the inside of the bend at the start of the curve and the outside of the bend at the end of the curve. Lane recommended that the shear stress values for a bend be increased by using a multiplier dependent upon the sharpness of the bend, expressed in the ratio of central radius to channel width R_c/b, as given in Table 1.

Table 1
Ratio of Shear Stress in a Bend to the Shear Stress in a Straight Channel

Bend Sharpness, $\frac{R_c}{b}$	10	8	6	4	2
$\frac{\text{Shear stress (bend)}}{\text{Shear stress (straight)}}$	1.05	1.20	1.43	1.70	2.0

C. GRAVEL LININGS

5. Allowable Shear Stress on Bed

Flat gradient channels with subcritical flow may be lined with gravel size stones to prevent erosion. Steep channels will produce higher velocities and will require larger size stones, usually referred to as riprap. Riprap linings are covered in Part E of this chapter.

According to Shield's analysis, fully developed rough turbulent flow will occur at the boundary if the stone size exceeds about 5 mm. Thus, rough turbulent flow will occur for all gravel and riprap stone sizes. The critical tractive force on the bed, according to Shields, is given by

$$[6] \qquad \tau_c = 0.06\,\gamma\,(S_g - 1)\,d_m$$

in which τ_c is the bed shear stress at incipient motion of the bed particles, the number 0.06 is Shield's stress value, γ is the unit weight of water (9810 N/m^3), S_g is the relative density of the stone particles (usually 2.65 for most gravel and riprap), and d_m is the equivalent spherical diameter of the median size of the stones (the size for which the weight of all stones in the sample smaller than the median comprise 50% of the total weight of the sample). For $S_g = 2.65$ and $\gamma = 9810\ N/m^2$, [6] reduces to

$$[7] \qquad \tau_c = 971\,d_m$$

with τ_c in N/m^3 and d_m in m.

The concept of a particular threshold value for critical tractive force is somewhat ambiguous. In a cohesionless bed of randomly distributed particles of various sizes and shapes, subjected to turbulent bursts which vary both spatially and temporaly, some aperiodic movement of the smallest particles could be expected even for a very small value of the shear stress. A much larger value of the shear stress would be required to produce general continuous movement involving all particle sizes. Thus, the shear stress covers a range of values as the particle movement goes from negligible to general. The so-called critical value becomes a matter of definition. For example, Meyer-Peter and Mueller have recommended that the critical Shield's stress value in [6] should be taken as 0.048 instead of 0.06. French recommends a value of 0.056, but with the particle size taken as d_{75} (75% finer than). In experiments at the University of Saskatchewan on a riprap layer laid on a slope, actual failure did not occur until a Shield's stress value of 0.068 was reached. The failure occurred suddenly, and the value of 0.068 was repeated with remarkable consistency for 15 different tests using 3 different sizes of stone and 4 different slopes. On this basis, [7], corresponding to a Shield's stress value of 0.06, includes a small factor of safety.

The question of safety factor requires consideration. If the maximum discharge is known and the flow is controlled or regulated so as not to exceed this value, as in conveyance systems for irrigation or water supply, then the design is safe as long as the bed shear stress τ_b does not exceed the value of τ_c in [7]. However, if the peak discharge is frequency related, as in a drainage ditch for surface runoff, the question of additional safety arises. For example, if the ditch lining is designed for a flow peak corresponding to a 5% frequency, then an event with a lower frequency would be expected to produce some damage and maintenance would be required. The U.S. Federal Highway Administration recommend that the permissible shear stress on the bed for roadside channels should be

[8] $$\tau_p = 630\, d_m$$

The value of τ_p in [8] is about two-thirds of the value of τ_c in [7], so [8] requires a 50% increase in stone size d_m and in effect includes a factor of safety of 1.5 based on size. The same result could be achieved by using [7] but designing for a lower frequency flood peak.

Setting the maximum shear stress on the bed, γyS, equal to the allowable shear stress given by [7], yields the relationship

[9] $$d_m = 10\, yS$$

This equation shows that the required median stone size varies directly as the product of the flow depth and channel slope. Equation [9] is based on the assumption that $S_g = 2.65$. If $S_g < 2.65$, the stone size must be increased to compensate for loss of weight by using the multiplier $1.65/(S_g - 1)$.

6. Allowable Shear Stress on Sides

The stability of a stone on the side slope of a ditch or canal is presumed to be less than for the same stone on the bed because the stone on the side slope is subject to the additional de-stabilizing force due to the downslope component of the weight of the stone. Based on a theoretical stability analysis it can be shown that the ratio of the allowable shear stress on the sides to the allowable shear stress on the bed, called the tractive force ratio K, is given by

[10] $$K = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}}$$

in which θ is the side slope angle and ϕ is the angle repose for the material. Equation [10] is shown plotted on Figure 3.

Equation [10] shows that as $\theta \to 0$, which is the condition for the bed, then $K = 1$ (i.e., allowable stress is equal to the allowable stress on the bed). When $\theta \to \phi$, then theoretically the side slope is at incipient instability even in the dry state, and $K = 0$ (i.e., no shear stress due to fluid flow is permitted). However, [10] is based on stability analysis for a single stone, neglecting the stabilizing effect of the inter-particle contact force between the stones on the side slope. Thus, provided the riprap layer on the side slope is properly anchored at the toe to resist the downslope weight component, for example with riprap placed across the bed, then [10] is conservative.

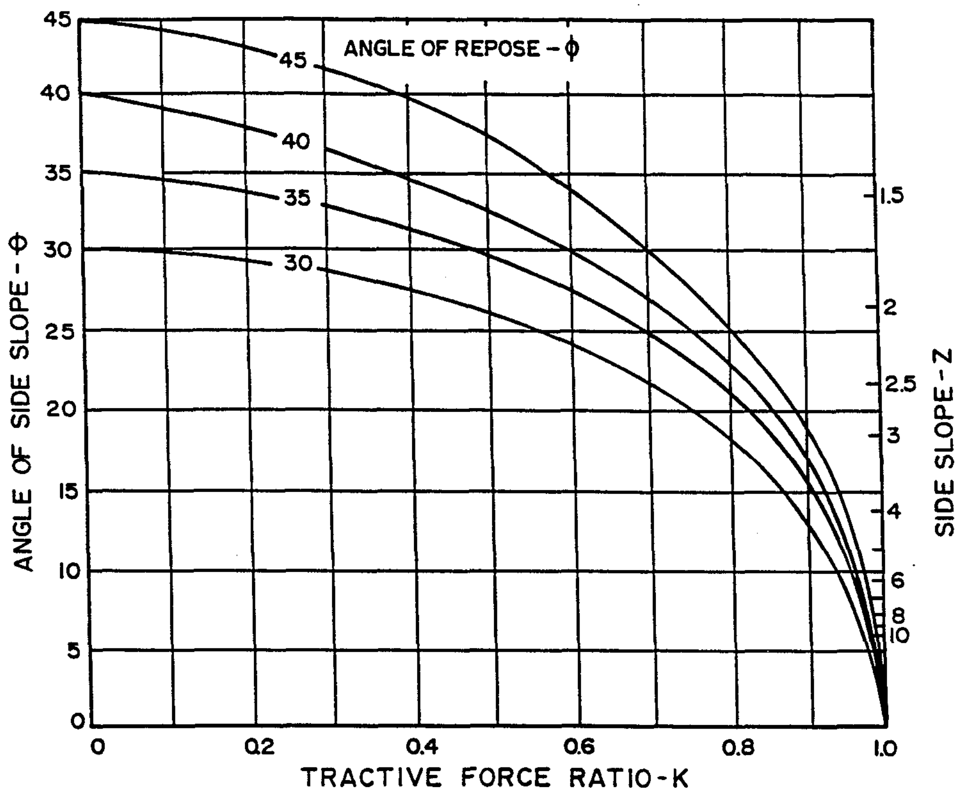

Figure 3 . Tractive Force Ratio

The value of K in Figure 3 represents the ratio of the stability of a stone on the side slope to the same size stone on the bed, and it is always less than unity. In the limit, if the side slope equals the angle of repose, K = 0 (i.e., incipient failure). Since the shear stress on the sides is less the γyS, as shown by Figure 2, it does not necessarily follow that larger size stone is needed on the side slopes. For example, for $b/y \geq 4$ and $z = 3$, Figure 2 shows that the side shear stress will be 0.86 times the bed shear stress, and for $\phi = 35$ degrees, which is typical, Figure 3 shows that the shearing resistance on the side is 0.86 times the shearing resistance on the bed (i.e., K = 0.86). Thus, the same stone size may be used for both the bed and the banks. In general, stone suitable for the bed will also be suitable for the sides if the side slopes are 3H : 1V or flatter.

In order to use Figure 3 a value must be selected for the angle of repose, otherwise known as the angle of internal friction. The coefficient of static friction, μ, is related to the friction angle by

[11] $$\mu = \tan \phi$$

so in theory ϕ could be determined from static friction tests. In practice, it is easier to measure the angle of repose. Figure 4 gives the angle of repose as a function of angularity and size of the particles. Angularity has the effect of increasing the angle of repose by a few degrees, obviously due to the greater interlocking effect of angular shapes as compared to rounded shapes. Figure 4 also suggests that the angle of repose is affected by grain size. Modelling theory indicates that if two samples have similarity of particle shape, gradation and porosity, then the angle of repose should be the same. Experiments have shown that the friction angle for cohesionless material will be increased by reducing the porosity (by compaction), or by using a wider size gradation, either of which will increase the number of intergranular contact points. These two effects are not accounted for in Figure 4. In addition, it seems unlikely that the particle shape for the fine sand (d_m = 0.4 mm) is the same as for the rock (d_m = 400 mm). The low ϕ value for fine sand, approaching 30 degrees, may reflect more porosity (due to air-cushioning effect) and more uniformity of size. In any case, for the linings considered here, minimum particle size will be at least 20 mm, so minimum friction angles will be about 35 degrees, possibly increasing to 40 degrees for larger stone sizes.

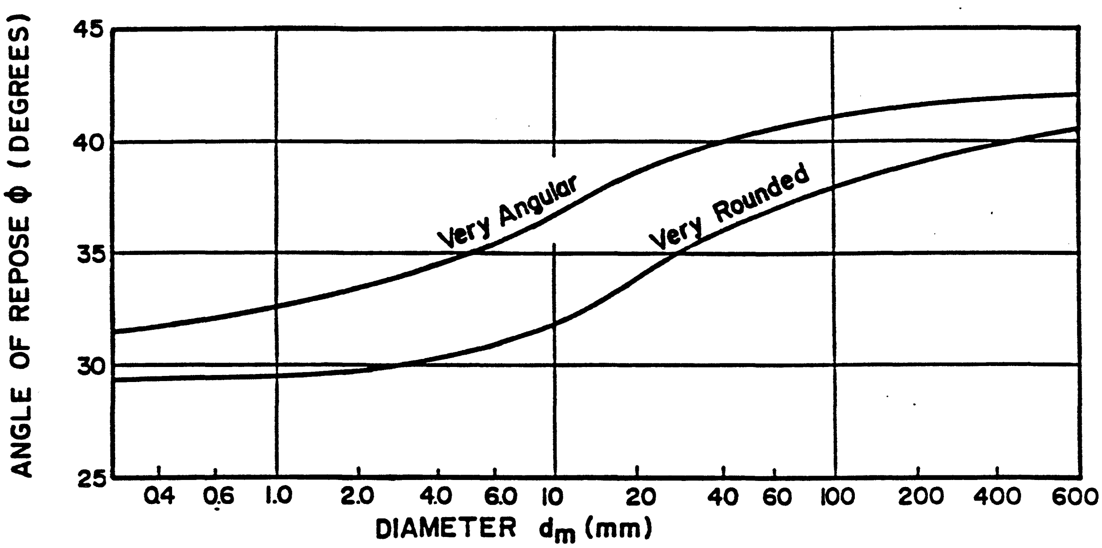

Figure 4 . Angle of Repose (Non-Cohesive Material)

7. Manning's n Values

The resistance to open channel flow over a plane boundary consisting of stable, large size, cohesionless particles is often expressed in terms of Manning's n, as a function of stone size, in the form

[12] $$n = C\, d_m^{1/6}$$

in which C is presumed to be a constant. However, different researchers have determined values for C which show significant differences. It is reasonable that C should be affected by the particle shape (rounded or angular) and by the gradation of the sample (uniform or graded). There is some evidence that C may also be affected by differences in Reynolds number and Froude number, and there is strong evidence that C depends upon the ratio of the flow depth to particle diameter ratio y/d_m, referred to as the relative roughness. Finally, in calculating n from Manning's equation using measured data on channel discharge, flow depth and slope, the question of the datum for the depth arises. For example, in the case of shallow flow over large riprap, significant differences in calculated n values can occur, depending on whether the channel bed is taken to be at the tops of the stones or at some lower level, such as $d_m/2$ below the surface. Often the questions of gradation, Froude number, Reynolds number and bed level datum are not reported with the data, so it is difficult to rationalize the differences in n values.

In S.I. units (with d_m in m) the values for C in [12] range from 0.039 by Keulegan to 0.053 by Straub. The best fit to Lane's data on San Luis Valley channels, for rounded material and y/d_m values ranging from 2.4 to 24, gives C = 0.046, but the n values varied by $\pm$ 0.005, reflecting the variability in channel conditions. Straub's C for flume data was 0.049. His high value of 0.053 was for angular Missouri River chalk rock excavated by cutter dredge and used for the diversion channel closure at Fort Randall Dam. Tests were done at the University of Saskatchewan in 1973 using 3 stone sizes - 15 mm, 19 mm and 23.5 mm. The stones included a mixture of rounded and broken stone. It was assumed that the bed level datum was at $d_m/4$ below the tops of the stones. The ratio for y/d_m varied between 2 and 5. For these conditions, the C value was 0.049. This value is recommended for design purposes, thus

[13] $$n = 0.049\, d_m^{1/6}$$

is used to calculate n values.

It should be pointed out that the assumption of a high n value leads to a greater calculated shear stress on the bed, because the flow depth used in [5] is increased. Therefore, it is conservative to use a higher n value. Simons has recommended lower n values, but also recommended lower allowable shear stresses. These are offsetting assumptions, so the net result, in terms of stone sizes required for stability, is basically unchanged.

In using Manning's equation, it is implicitly assumed that the flow is uniform (i.e., at normal depth). At the entrance to the lined channel the flow may very well occur at a greater depth. For example, if the channel is hydraulically steep, the flow may pass through critical depth at the entrance to the channel, and subsequently accelerate down the slope until normal depth is reached. The water surface profile between critical depth and normal depth would be an S2 profile. The critical region for riprap stability begins at the downstream end of the S2 profile because that is where the maximum velocity is reached. Thus, the assumption of uniform flow for design purposes is conservative as far as the upper reach of the lined channel is concerned. However, it should be noted that the flow rapidly approaches the uniform flow depth on steep rough channels, often in a length of only 20 times the depth, so design for uniform flow is normally required.

8. Design Procedure

Equations [2], [9] and [13] must be simultaneously satisfied, and will give a unique solution relating the discharge, channel size, channel slope and stone size for the channel lining. Usually, the solution is achieved quite readily using a simple trial and error procedure.

The layer for the lining must be at least 1.5 d_m in thickness in order to allow some degree of overlapping. If the stone sizes in the sample have a wide variation, a thicker layer, up to 2 d_m, may be necessary. From a practical point of view, the layer thickness should not be less than about 0.15 m, even if $d_m < 0.10$ m would be satisfactory. A filter layer should be placed under the stone layer to prevent foundation material from being washed out or sucked through the voids in the rock layer. The filter layer may consist of a graded sandy gravel and should have a thickness of d_m or 0.1 m, whichever is greater. The filter layer is intended to constitute a transition between the fine particles in the foundation and the coarse particles in the stone lining, and it must be designed to prevent foundation material from being washed out or sucked through the voids in the rock layer. Recently, geotextile products have been widely used as a substitute for granular material for the filter layer. These products, known as filter cloth or filter fabric, allow water to pass through, but not soil particles. Filter fabric is dependable, easy to install, and eliminates problems of quality control associated with construction of granular filters.

It should be noted that prediction of performance for stone linings, both riprap and gabion mattresses, is less precise than for rigid linings such as concrete. Fixed boundary hydraulics is a fairly exact science. Loose boundary hydraulics introduces questions about how shear strength and Manning's n are affected by sample gradation, by Froude and Reynolds numbers, by relative roughness, and by channel slope. In the design procedure it is assumed that shear strength and Manning's n can each be expressed as a function of the median size of the stone alone. This procedure is an obvious over-simplification of the problem, but it gives designs that will be satisfactory in the majority of cases. It has been stated that in the area of loose boundary hydraulics the engineer must be an artisan as well as a scientist. This cautionary note is introduced because the engineer should be aware that a 100% success rate is unlikely. Fortunately, the failures are usually amenable to simple remediation - just add more rock or bigger rocks as the situation requires.

Example 1:

A channel is excavated through a silty sand alluvium on a slope of 0.0008. It has a bottom width of 8 m and side slopes of 3H:1V, and it is intended to discharge 39 m^3/s flowing at a depth of 2 m (based on n = 0.025). The channel begins to erode quite severely as soon as it is put into operation. Design a gravel lining to control this erosion.

The channel was improperly designed for this discharge and soil condition, and severe erosion would be a normal expectation. The flow velocity, at 1.4 m/s, is much too large for a silty sand. The channel should have been designed using a width of 20 m and a slope of 0.0002, producing a flow velocity of 0.75 m/s. A gravel lining will be essential in order to control the erosion in the existing channel. From [9]

$$d_m \quad = \quad 10\,yS \;=\; 10 \times 2 \times 0.0008 \;=\; 0.016\text{m}$$

Since a minimum layer thickness of 0.15 m is recommended, d_m can be increased without increasing the required volume of gravel, so d_m = 25 mm is recommended. The use of oversize material will also accommodate bends in the canal. According to Table 1, a bend sharpness of $R_c/b = 4$ would be permissible for 25 mm material. A gravel having an equivalent spherical diameter of the median size of 25 mm will be expected to contain a range of stone sizes, such that the shortest triaxial dimension of the smallest stone may be 10 mm and the longest dimension of the largest stone may be 60 mm.

The n value for this stone, from [13], is

$$n \;=\; 0.049\, d_m^{\;1/6} \;=\; 0.049 \times 0.025^{1/6} \;=\; 0.0265$$

Since this value is slightly larger than the design value of 0.025, the flow depth in the canal will increase slightly above the original design value of 2 m. It can be shown, using [2] (Manning) that the new depth will be 2.07 m.

D. GRASSED CHANNELS

9. Class of Vegetation

Vegetation is classified in 5 categories, A to E inclusive, according to retardance rating, with Class A having very high retardance and Class E having very low retardance. The retardance refers to the ability of the vegetation to impede the passage of the flow and resist erosion of the boundary.

Erosion resistance is best achieved with a well rooted high density grass, and is greater for tall grasses than short grasses. Root binding of the sod contributes to erosion resistance, but the above ground vegetation is equally important. The vegetation is bent under the influence of the flowing water, and a dense stand of tall grass will form a formidable protective layer on the surface, greatly assisting in the prevention of erosion. The minimum requirements of the vegetal cover to qualify for each of the 5 classes is shown in Table 2.

Table 2
Ratings for Grassed Channels

Class	Retardance Rating	Vegetal Cover Requirement	Permissible Shear Stress * (N/m^2)	Riprap Equivalent ** (mm)
A	Very High	Good Stand ≥ 75 cm	270	278
B	High	Good Stand ≥ 30 cm Fair Stand ≥ 75 cm	150	154
C	Moderate	Good Stand ≥ 15 cm Fair Stand ≥ 30 cm	75	77
D	Low	Good Stand ≥ 5 cm Fair Stand ≥ 15 cm	45	46
E	Very Low	Good Stand < 5 cm Fair Stand < 15 cm	25	26

* Must be decreased by one-third to obtain a factor of safety of 1.5.
** From [7]

10. Canadian Prairie Grass Types and Retardance Rating

Generally a Class A rating is not common and a Class E rating should be avoided. A Class A rating can be attained by a good stand of yellow sweet clover at maturity (90 cm). Earlier in the season, the rating may be Class B (30 cm) or Class C (15 cm).

Good stands of alsike clover, alfalfa, birdsfoot treefoil, blue grama, fescue, Kentucky bluegrass, white clover or Western Parks grass mixture can each be given a Class B rating at 30 cm height, or Class C rating at 15 cm height.

Alkali grass and rye grass generally qualify for Class D rating only, and all of the grasses named in the previous paragraph would have to be given a Class D rating if they were mowed to a 5 cm height. If the grasses are mowed for hay and the stubble is burned off, the rating is reduced to Class E.

It is evident that the retardance rating may vary throughout the year, and even vary from year to year depending on the climatic conditions. The suitability of the grasses is dependent on soil type, topography, climate, other environmental conditions, and the time of year and frequency of use of the drainage channel. The quality of the growth is often quite site specific and may be difficult to estimate in advance. In this respect there is no substitute for local experience. Clover, which is a deep rooted, drought resistant perennial, is often a good choice.

11. Allowable Shear Stress

Table 2 gives the allowable shear stress for each class of vegetation. The values must be decreased by one-third if a factor of safety of 1.5 is desired. Exceedence of these values would be expected to produce erosion in some parts of the channel. It should be noted that the grass on the side slopes is equally resistant to erosion as it is on the bed, so, unlike the case for gravel and riprap, the allowable shear stress on the sides does not have to be reduced. Generally, flat side slopes (3H:1V or flatter) are preferred, as these are easier to seed, mow and drive across, are more stable, and give more uniform growth.

As a matter of interest, the equivalent riprap size, calculated from [7] is shown for each class of vegetation in Table 2. Thus, 77 mm riprap would be equivalent to a good stand of grass at 15 cm height.

12. Manning's n Values

The U.S. Soil Conservation Service in the 1940's conducted field tests on erosion resistance and flow resistance of various classes of grass vegetation. It was found that the flow resistance, expressed in terms of Manning's n, varied over a wide range even for one type of grass. The n values were observed to decrease as the flow depth increased. This result was an obvious relative roughness effect, since if the grass height was greater than the flow depth, the flow resistance would be great and Manning's n values would be high. If the flow depth was much greater than the grass height, the reverse effect would be true, and Manning's n values would be much lower. It was also observed that the flow resistance was affected by the velocity, in that higher velocity flow was accompanied by lower values of Manning's n. This effect was due to the change in the boundary roughness as the velocity increased. The grass would be increasingly bent or flattened in the direction of flow as the velocity increased, effectively making the channel smoother and the flow resistance lower.

It was determined that a unique relationship existed between n and the VR product. This relationship can be expressed mathematically in the form

[14] $n = C/(VR)^{1/2}$

in which C is different for each class of vegetation. Unfortunately C is not even constant for one class of vegetation as VR changes, so the equation is not particularly useful except to show the reciprocal type relationship between n and VR. The U.S. Soil Conservation Service published charts of n vs VR for each of the 5 classes of vegetation.

Noting that $V = R^{2/3} S^{1/2}/n$, [14] can be written as

[15] $n = C^2/(R^{5/3} S^{1/2})$

In this form the velocity is eliminated from the equation and n can be expressed as a function of R and S instead of R and V. This transformation was the basis for the U.S. Department of Transport 1988 circular, in which 5 charts of n vs R and S were prepared. These charts are reproduced, converted to S.I. units, in Figures 5 to 9. Equation [15] cannot be used in practice because of the undetermined variation in C. This variation is accounted for in Figures 5 to 9. In essence, the relationship is purely empirical, but examination of the curves shows that n decreases as either R or S increases, and that the effect of R is more pronounced than the effect of S. (The exponent of R is larger than the exponent of S in [15].) As an example, in Figure 5, if R is increased 10-fold from 0.3 to 3.0 on the S = 0.03 line, n decreases from 0.300 to 0.046, whereas if S is increased 10-fold from 0.001 to 0.010, on the R = 1 line, n decreases from 0.190 to 0.084.

13. Design Procedure

Manning's equation, the n vs R and S relationship, and the stability requirement must be satisfied simultaneously. The stability requirement is that the shear stress should be equal to or less than the permissible value (Table 2). Trial and error may be required for the solution, depending upon which values are initially specified.

The performance of the design is less certain than for stone structures or other rigid boundary structures because of the variation in vegetal quality from one area to another, from one season to another, or due to the difference in grasses even in the same class. Estimates may have to be made of the magnitude of the discharge to be expected in different seasons of the year. If summer rains produce the largest discharges, a Class B or C rating may be appropriate. If spring snowmelt is the governing condition, Class D or E rating may have to be applied. If the discharge is expected to occur several times a year, more conservatism is required. This could be achieved by reducing the permissible shear stress by one-third (equivalent to a factor of safety of 1.5), or reducing the class rating of the vegetation.

Example 2:

A floodway bypass channel is to be designed to carry a regulated maximum discharge of 50 m^3/s. Use of the channel will be infrequent, about once every 2 years, and the maximum flow will occur only once every 10 years. The channel will have a bottom width of 10 m and side slopes of 5H:1V to permit haying operations in the channel. The channel will be seeded with a grass mixture and a Class C rating is assigned. What slope should be used for the channel?

Assume a flow depth of 2 m, for which A = 40 m^2, P = 30.4 m and R = 1.32 m. Then from Figure 7, for R = 1.32 m, if S = 0.001 then n = 0.056, from which Q = 40 x $1.32^{2/3}$ x $0.001^{1/2}$/0.056 = 27.2 m^2/s. If S = 0.003 then n = 0.046, from which Q = 57.3 m^3/s. By interpolation, the required S for 50 m^3/s becomes 0.0025. The shear stress γyS for this design would be 9810 x 2 x 0.0025 = 49 N/m^2. The permissible shear stress is 75 N/m^2 (Table 2), so the channel with 2 m depth and 0.0025 slope is satisfactory. If the shear stress had been too large, the procedure would have to be repeated using a larger depth, giving a flatter slope and a lower velocity.

Example 3:

A roadside ditch with 2 m bottom width and side slopes of 3H:1V has a slope of 0.05. What maximum discharge will grass with a Class C rating withstand?

The maximum allowable flow depth, using a permissible shear stress of 75 N/m^2 (zero factor of safety), as given by $\tau_b/(\gamma S)$, is 75/(9810 x 0.05) = 0.153 m, from which

A = 0.376 m^2

P = 2.97 m

R = 0.127 m.

From Figure 7, for R = 0.127 m and S = 0.050 m, n = 0.10, from which

Q = 0.376 x $0.127^{2/3}$ x $0.05^{1/2}$/0.10 = 0.213 m^3/s.

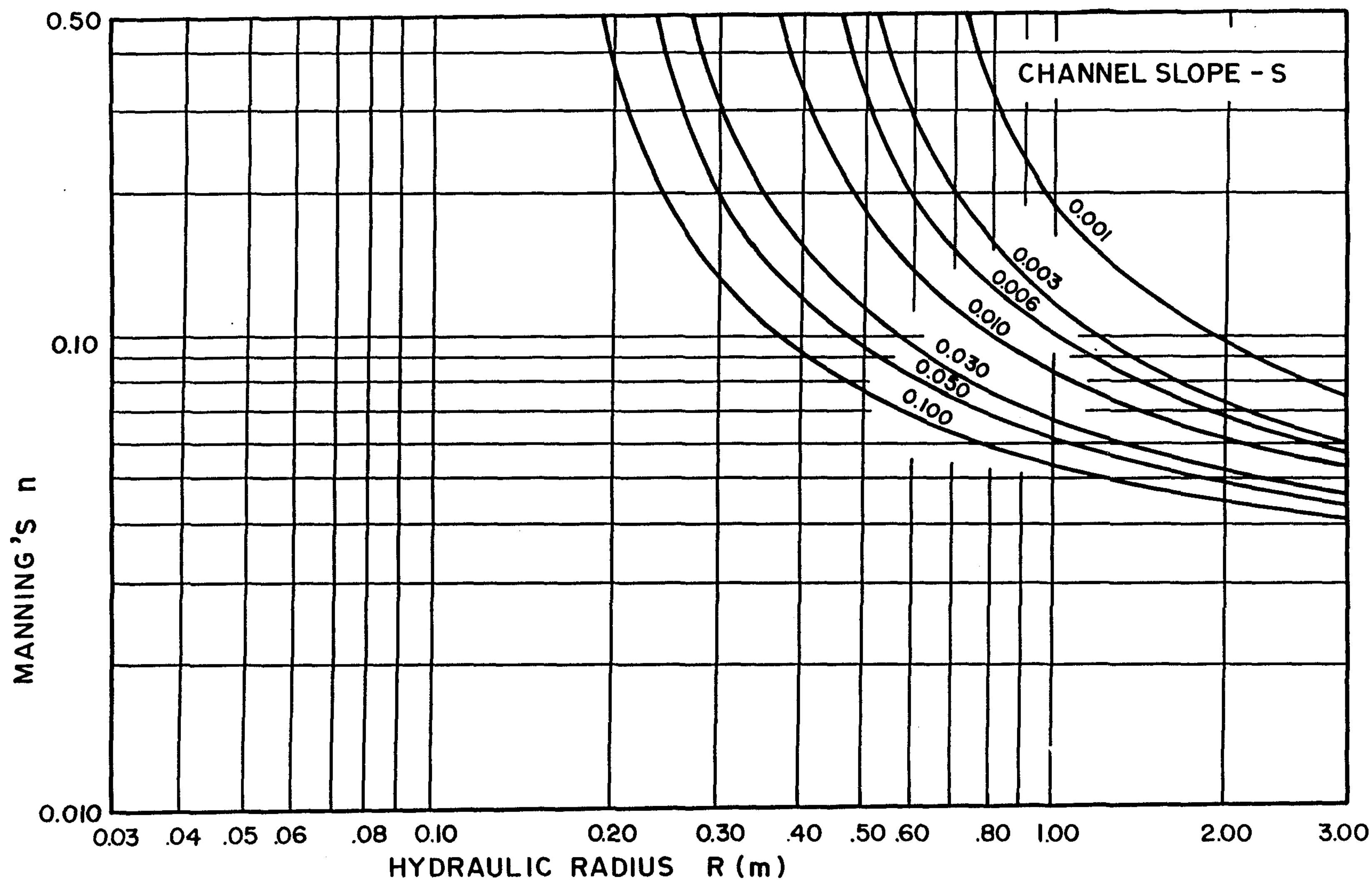

Figure 5. **Manning's n vs. Slope and Hydraulic Radius**
Class A - Very High Retardance

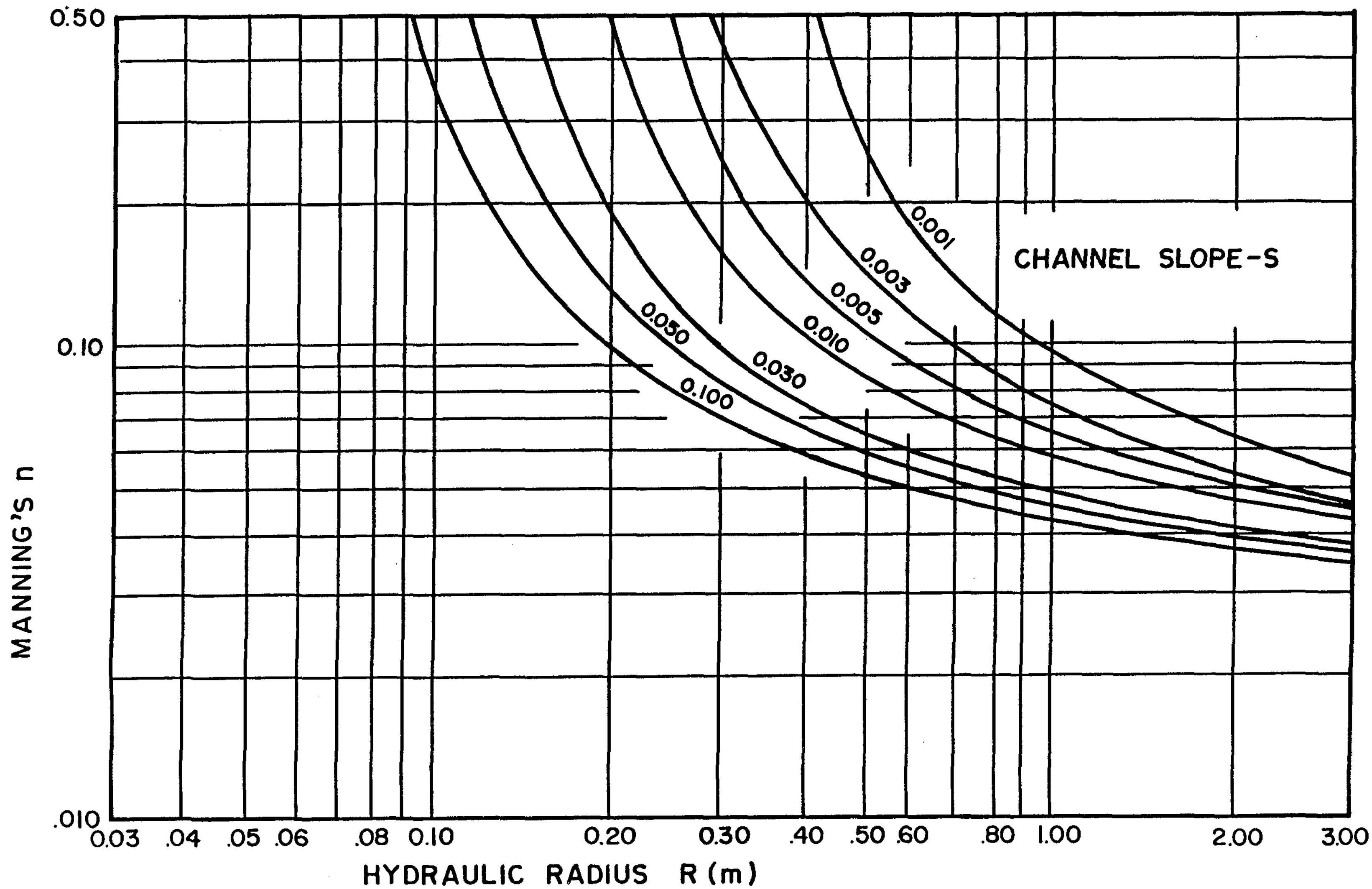

Figure 6. **Manning's n vs. Slope and Hydraulic Radius**
Class B - High Retardance

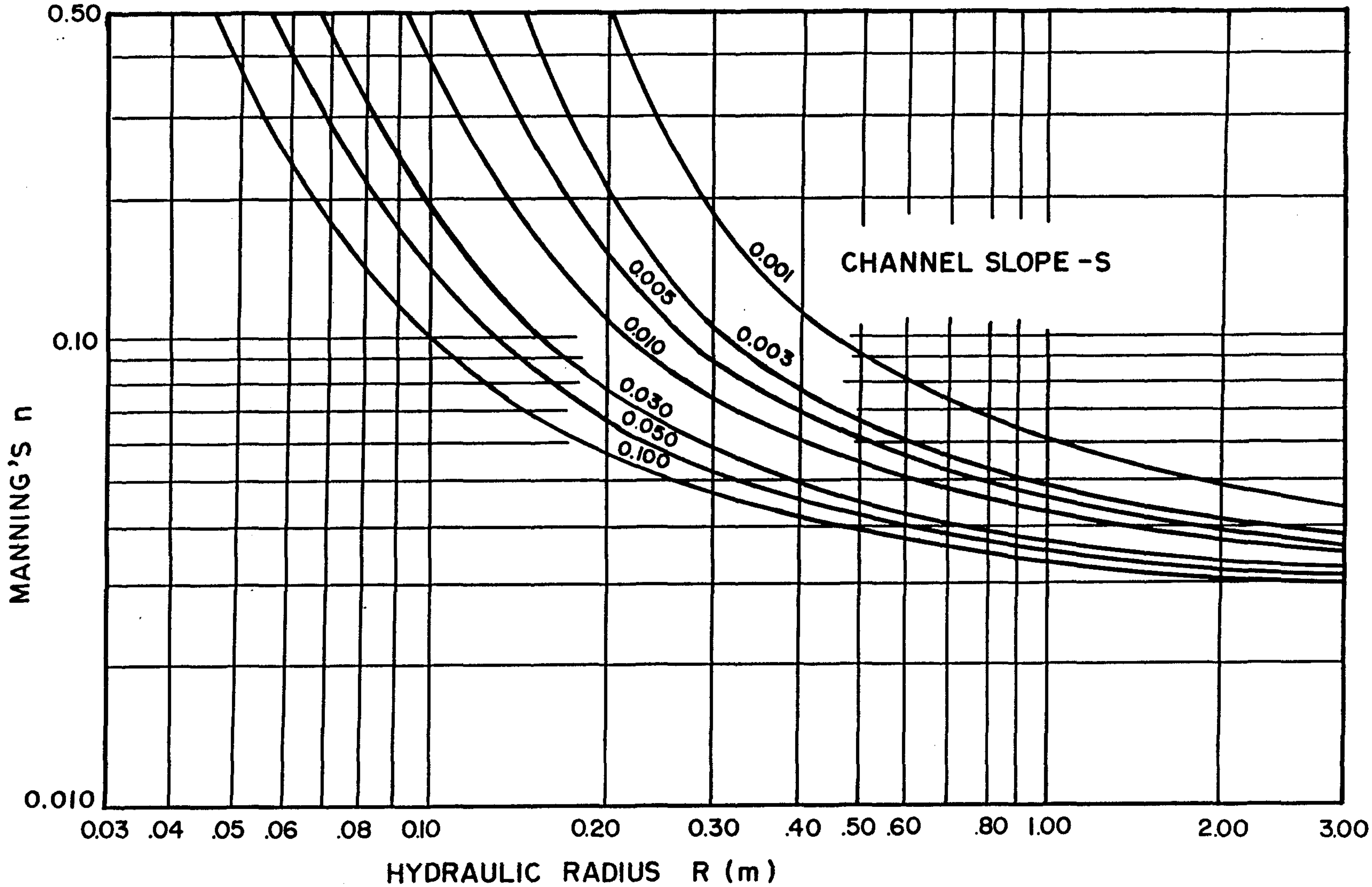

Figure 7. Manning's n vs. Slope and Hydraulic Radius
Class C - Moderate Retardance

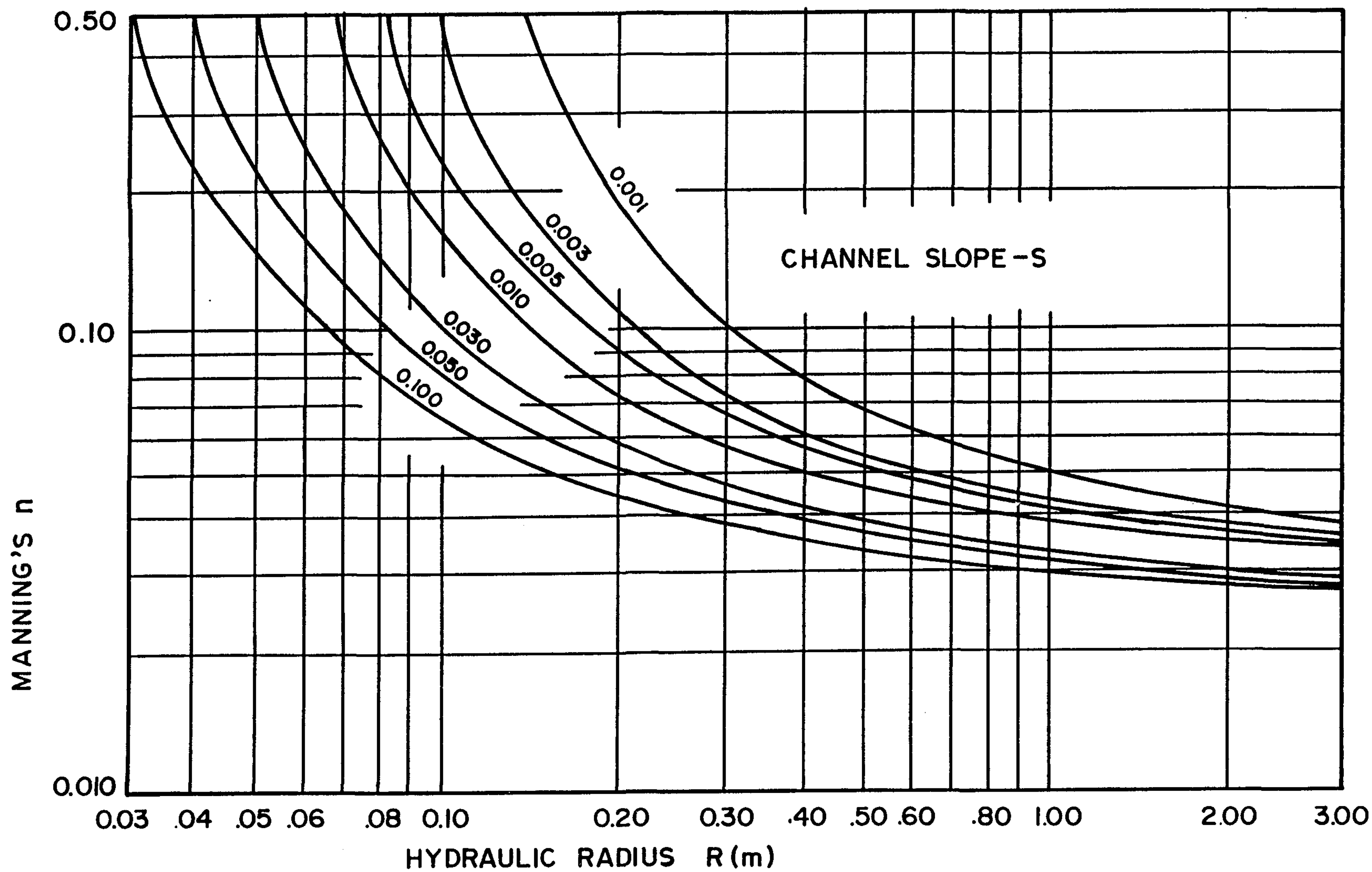

Figure 8, **Manning's n vs. Slope and Hydraulic Radius Class D - Low Retardance**

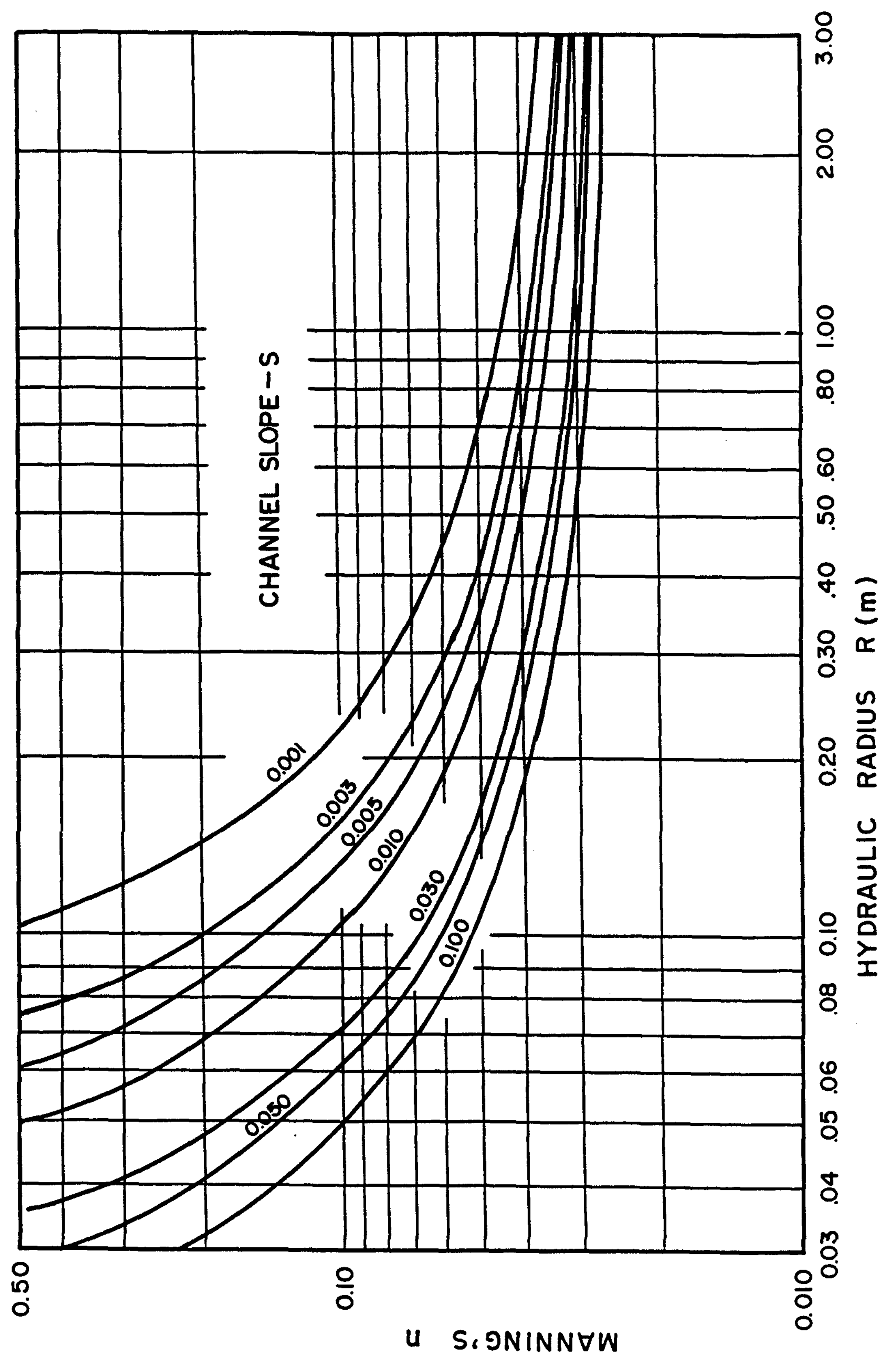

Figure 9. Manning's n vs. Slope and Hydraulic Radius
Class E - Very Low Retardance

E. RIPRAP LININGS FOR STEEP GRADIENTS

14. Application

Steep gradient channels are channels on which the flow becomes supercritical. Such conditions can occur in roadside ditches where highway grades up to 6%, or in extreme cases 8%, are used. Steep gradients may also be used on overflow channels from lagoons, tailings ponds or stock watering dams, and in fact can be used as a substitute for reinforced concrete drop structures along a canal. Riprap sized material is invariably required for these structures. While slopes up to 20% or 25% are technically possible for an overflow channel, stone sizes become very large and the problems of piping of the foundation material under the riprap require special consideration.

15. Theory

There is a question as to the applicability of the tractive force method, developed for gravel sized material on flat gradients, to riprap on steep gradients. On flat gradients, the ratio of the flow depth to particle size is very large, even up to 100. The bed elevation is readily defined and the water surface, for subcritical flow, is smooth and level transversely, so the depth, y, can be accurately defined and easily measured. Use of [9] ($d_m = 10$ ys), is appropriate. On steep gradients where shallow high velocity flow may occur and large stone size is required for stability, the stone size may become as large as the flow depth. The effective value of the bed elevation becomes a subject of debate. When the water surface reaches the tops of the stone, there may already be a significant flow down the slope through the stones so the tops of the stones is not an appropriate datum for the bed elevation. Furthermore, for shallow flow over a rough bed, the water surface tends to be very rough and difficult to define or measure, so [9] cannot be directly verified by experiment.

For shallow high velocity flow, it is satisfactory to let the hydraulic radius equal the flow depth. In this case, Manning's uniform flow equation may be written as

[16] $$q = y^{5/3} S^{1/2}/n$$

If it is assumed that $n = 0.049\, d_m^{1/6}$, as in [13], and $y = d_m/(10S)$, as in [9], then d_m may be expressed in terms of q and S as

[17] $$d_m = 1.73\, q^{2/3} S^{7/9}$$

This equation represents the same degree of stability as [9], and in fact would give the identical result as [9] for a gravel lined canal. It remains to verify that [17] can be applied to riprap linings on steep gradients. The advantage of [17] is that q and S can be readily measured without reference to the flow depth.

It should be noted that Stephenson in 1979 proposed an equation which an be reduced to

[18] $d_m = 1.62\, q^{2/3}\, S^{7/9}$

which is remarkably similar to [17], and purports to give the "threshold" flow at which initial movement of the riprap will occur.

16. Verification of Theory

An experimental program was undertaken to determine the magnitude of the unit discharge which would cause "failure" of stone protection of a specified size on a specified slope. Basically, the program was intended to determine a design value for K in the equation

[19] $d_m = Kq^{2/3}S^{7/9}$

Tests were run in a parallel-sided, glass-walled flume 305 mm wide. A base layer of uniform sand (d_m = 0.5 mm) was placed in the flume at the specified slope. The slope length was 4 m. Since typical flow depths were about 0.030 m, the slope length was more than sufficient to produce uniform flow over the length of the slope except for a very short length at the top where S2 water surface profile developed. Four different slopes were tested, namely 4%, 5%, 6% and 7%. Three stone samples were prepared, consisting of semi-rounded stones with d_m values of 0.0152m, 0.0195m and 0.0235m. The stone sizes were determined by weighing and ranking a sample of 100 stones in each size class. The 0.0152m and 0.0235 m stone samples were quite uniform, having been sieved through a narrow size range. The 0.0195 m stone sample was a mixture of the two previous sizes and was considered graded. The stone had a relative density of 2.65 as determined by a standard weighing and water displacement method.

Although initial tests were run with stone layer thicknesses on the sand bed of both $2d_m$ and $1.5d_m$, it was found that the $2d_m$ layer was only marginally more stable. Thereafter, all tests were conducted with a layer thickness of $1.5d_m$. In all cases, the layer thickness was established by drawing a line on the flume sidewall at a distance above the slope corresponding to the desired layer thickness. The stones were subsequently placed on the slope so that approximately one-quarter of the median stone height was above the line in accordance with the recommendation of Smith and Strang for the effective position of the top of a stone bed. The stone layer was extended horizontally downstream, from the end of the slope to a length of six times the tailwater depth (i.e., L) for the corresponding design discharge. A schematic illustration of the riprap slope is shown in Figure 10.

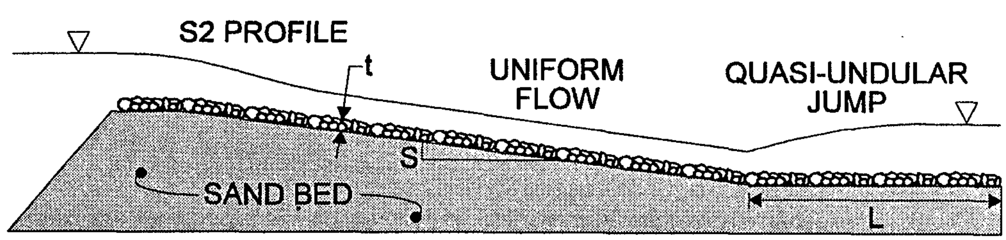

Figure 10. Schematic View of Riprap Slope

In a typical test, the discharge was increased in small increments, with a time lapse of at least 60 minutes between increments. The stone layer was carefully observed through the glass sidewalls. It was found that any adjustments in the stone layer in response to each increase in discharge occurred soon after the change in discharge. The lack of further activity was verified during initial testing in which each discharge was run continuously for up to 24 hours.

As the discharge was incrementally increased, it was observed that a few stones would initially wobble in place, then move slightly, and then move a short distance to a more stable location in the stone bed. Following that, more stones moved a greater distance, then the sand bed became temporarily exposed in local areas, and finally the continuity of the surface layer was disrupted and ceased to provide protection for the sand base, which severely eroded. The change in bed response was progressive in that the discharge at ultimate failure was at least 5 times as great as the discharge which produced the first observed movement (i.e., wobble) of a stone.

To appreciate the following discussion, it is necessary to introduce the concept of "initial failure" as opposed to "ultimate failure". When the stone layer became thin, to the point of exposing the sand bed in a local area, the stone would shift from the upstream edge to the downstream edge of the exposed area, which migrated to the top of the slope. This exposure was considered an initial failure, although the adjusted layer remained stable and required a further increase in discharge of about 50% before ultimate failure occurred. This failure process has been observed by many researchers, and has been likened the process to yield strength and ultimate strength in structural design. Stephanson noted that complete collapse of the slope would not usually occur until flow rates reached 120% to 180% of the threshold flow from [19].

In the research by Smith and Murray, the initial failure was selected to correspond with the observation where there had been sufficient movement of stone that the sand bed was temporarily exposed in a local area. The corresponding K values in [19] were calculated for the measured discharge, slope and stone size, and are shown in Table 3 for the entire test range. Several factors emerge. The K value is very nearly constant, and it is smaller than the value of 1.73 in [17], corresponding to Shields' criterion for first movement. It is apparent, therefore, that the first movement criterion is conservative relative to initial failure in a riprap channel. In a river channel, there may be continuous movement at K=1.73 because sediment continuously enters the reach from an upstream source. In a riprap channel, once there is a minor adjustment of some smaller stones exposed on the surface, no further movement occurs. A final observation is that the performance of the graded samples (i.e., Test Nos. 6 and 7) was basically the same as the more uniform samples. It appears that the presence of larger stones in the graded sample compensates for the presence of smaller stones by a sheltering action, so there is no loss in stability.

For the tests in Table 3, it was observed that initial movement, defined as the condition where a stone was first transported to the base of the slope, occurred at a K value between 1.7 and 1.8. This movement was confined to a small number of small stones exposed on the surface (i.e., in a vulnerable position). Ultimate failure, defined as a mass downslope movement of the riprap layer, generally occurred at a K value between 1.1 and 1.3.

Table 3
K Values for Initial Failure
(Semi-rounded stone, S_g = 2.65)

Test No.	Stone Size, d_m (m)	Channel Slope, S (m/m)	Unit Discharge, q (m^2/s)	K
1	0.0152	0.04	0.0461	1.445
2	0.0152	0.05	0.0332	1.512
3	0.0235	0.05	0.0648	1.497
4	0.0235	0.06	0.0607	1.357
5	0.0235	0.07	0.0461	1.446
6	0.0195	0.05	0.0509	1.459
7	0.0195	0.07	0.0340	1.469
			Average	1.455

For normal design, it is recommended that a K value of 1.73 be used, as in [17]. This value includes a safety factor of 1.2 relative to initial failure (viz., 1.73/1.455 = 1.19). For those in structural engineering or geotechnical engineering where a safety factor of 1.2 may seem dangerously low, it should be pointed out that the value of 1.2 is based on size (e.g., diameter), whereas stability is based on a force analysis (i.e., weight). Thus, the safety factor based on force would be $(1.19)^3$ or about 1.7. In addition, initial failure does not result in actual collapse of the slope. With respect to ultimate failure, and based on a force analysis, the safety factor would fall between 2 and 3.

It is of interest to note that stone paved channels have been used as an alternative to concrete for major spillways. If the spillway is to pass the PMF, it is normal practice to reduce the required factor of safety for maximum discharge, a design philosophy which is justified in consideration of the extremely rare occurrence and short peak duration of the PMF. For example, the K in [17] corresponded to 1.25 at the peak of the PMF for the rock armoured spillway for Wadi Khasab Dam in Aman (d_m = 1250 mm, q = 5.1 m^2/s, S = 0.25).

It had been anticipated that failure would occur at a larger value of K as the slope was steepened, due to the increasing downslope component of the submerged weight of the stone (neglected in [17]). This effect is not apparent in the results of Table 3, although the downslope component should be 75% greater for the 7% slope than the 4% slope. In order to magnify the presumed effect, an additional test using a 20% slope was undertaken, for which the downslope weight component is 390% greater than for the 4% slope. It was observed that [17] with K=1.73 applied equally well to the 20% slope. The anticipated increase in K value did not occur. There appear to be two mitigating factors at work. First, on steep gradients, which require large stone sizes in a thick layer, a significant percentage of the total flow passes through the layer, thus reducing the depth of the surface flow. Second, layer stability must be considered as opposed to individual stone stability. Many researchers regard the downslope component of weight of an individual stone as a de-stabilizing force. Undoubtedly, this is true when considering a single stone on a plane surface. However, in a stone layer, this force is resisted at the contact points with other stones in the layer. An individual stone in the layer cannot slide or roll downslope unless the whole layer also moves downslope, or unless the stone is first lifted out of the layer so it may move downslope on the surface. The lift force on the stone is resisted not only by the component of stone weight normal to the surface, but also by the friction force developed at the contact points with other stones in the layer. This friction force is related to the contact point forces between the stones, and therefore increases as the slope increases. The neglect of this friction force in free-body analysis was first pointed out by Wittler and Abt, who also found that no adjustment of coefficients or exponents in the stability equation are required for slopes from 2% to 20%.

17. Throughflow

It should be noted that the discharge in [17] is the total discharge, including the flow through the stone layer. The latter discharge becomes significant when the layer consists of large size stone in a thick layer on a steep gradient. To compute the throughflow velocity in the stone layer, recourse may be made to Forchheimer's equation for non-Darcy flow through rockfill given as

[20] $$i = av + bv^2$$

where i is the hydraulic gradient (equal to the channel slope), v is the seepage velocity in m/s, and a and b are the Forchheimer coefficients. Coefficients a and b have been determined by experiment for material sizes up to 84 mm. It is assumed that the values can be used for larger sizes as well, and, as recommended by Kells, may be calculated from

[21] $$a = 135.8\, d_m^{-1.627}$$

and

[22] $$b = 6275\, d_m^{-1.420}$$

with d_m in mm, and a and b having units of s/m and s^2/m respectively. The throughflow discharge is calculated by multiplying the velocity, from [20], by the cross-sectional area of the stone bed.

The basis for the Forchheimer equation is that for flow through fine grained material (i.e. laminar flow), v is very small and bv^2 becomes negligible. For flow through large void spaces in a stone layer the flow becomes turbulent, and bv^2 becomes dominant. Thus, the form of [20] accommodates the transition from laminar flow to turbulent flow as the grain size of the layer increases. Application of [20] to flow on steep gradients, such as 20%, shows that for a layer thickness of 1.5 d_m, the discharge through the layer may be of the order of 30% to 40% of the design discharge.

18. Filter Layer

The large throughflow discharge in the stone layer draws attention to a very important feature of the design. It is essential to have a filter layer, sometimes referred to as a bedding layer, under the riprap in order to avoid erosion of the subgrade material. This feature is particularly important if there is fine sand or silt in the subgrade. The material would be eroded at the surface by the throughflow discharge, and transported through the voids in the riprap to the bottom of the slope. The upslope stone layer would subside as the foundation material is removed. This type of action was graphically illustrated on the model with 30 mm stone on a 20% slope over a sand bed with no filter. Long before the design discharge was reached the layer had subsided due to loss of upslope subgrade material. The eroded material formed a large sand dune in the downstream channel in a low velocity area. The test was discontinued when the slope had flattened to 10%, although the slope was still not stable. This type of failure was avoided on the model by using a simulated filter fabric under the riprap. The fabric prevented transport of the upslope subgrade material through the stone layer.

Commercial filter fabrics (geotextiles) are increasingly being used as a substitute for a graded sand and gravel filter. These fabrics have the advantage of quick installation, and their use eliminates the problems of quality control which often occur with sand and gravel filters. However, it is not recommended that large size stone, particularly quarried rock, be dumped directly on the filter fabric, as there is a danger that the fabric may be punctured or torn by a sharp edge. It is preferable to prepare a base layer of 5 mm to 25 mm gravel, in a 100 mm layer, which is placed on top of the filter fabric before the riprap is dumped and spread.

19. Design

The riprap structure may be designed for two-dimensional flow if parallel vertical sidewalls are used. Such sidewalls could be constructed with driven steel or timber sheet piling, or stacked up rock-filled gabions. However, a channel with slopeing sides would be less costly and more typical of actual construction. In this case the flow would be three-dimensional. Details for such a structure are shown in Figure 11.

Given a stone size and channel slope, the design unit discharge may be determined from [17]. The approximate channel width required for the total discharge may be determined from

[23] $$B = Q/q$$

in which B is the bed width of the sloping channel. The width in [23] will be somewhat wider than necessary since the discharge over the sideslopes is not included. If the channel is narrow, this extra discharge may be significant, and an adjustment may be desirable (i.e., a decrease in B) after a trial design. The total discharge must also satisfy Manning's equation

[24] $$Q = A\,R^{2/3}\,S^{1/2}/n$$

where the area A and hydraulic radius R can be calculated after the flow depth is determined from [9], and with n given by [13].

In Figure 11, the stone slope extends to a height P above the bed level of the approach channel, forming a triangular rock weir at the top of the slope. The upstream slope of the weir may be set at 2 horizontal to 1 vertical, or just flatter than the angle of repose. This is not critical for stability because flow velocities are upslope. The weir height is determined as being whatever value is necessary to prevent excessive drawdown in the channel approaching the drop structure. Since the channel will have sloping sides, the weir is trapezodial in front elevation. The discharge equation for a trapezoidal weir is

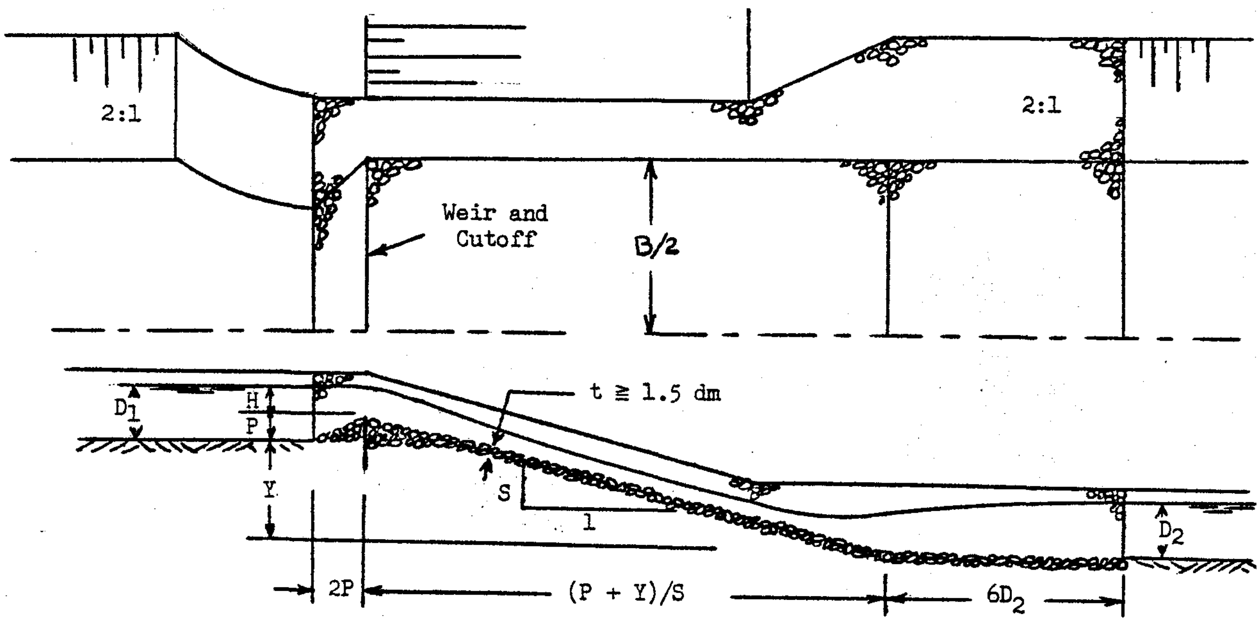

Figure 11. Stone Paved Sloping Channel

[25] $Q = C_d(2/3)\sqrt{2g}BH^{3/2} + C_d(8/15)\sqrt{2g}H^{5/2}\tan(\theta/2)$

in which C_d is the weir discharge coefficient, B is the length of the weir crest (equal to the bottom width of the trapezoidal channel), and $\theta/2$ is the angle of the side slope relative to the vertical. The portion of the expression containing $H^{3/2}$ represents the discharge passing over the rectangular portion of the weir, and the portion containing $H^{5/2}$represents the discharge over the sloping sides at each end (considered as a triangular weir). Because of the flat slopes upstream and downstream from the weir the flow will essentially be critical flow at the crest of the weir. In this case [25] can be reduced to

[26] $Q = 1.70\ BH^{3/2} + 1.27\ H^{5/2} \tan(\theta/2)$

This equation is sufficiently accurate for a determination of the head H, given Q, B and θ. The recommended side slopes for the channel are 2 horizontal to 1 vertical, for which tan $(\theta/2)$ will have a value of 2.

If seepage through the weir must be prevented, it will be necessary to have some type of water barrier in the center of the weir. This may be a concrete wall, a creosoted timber wall, or timber or steel sheet piling.

Usually the stone will be graded to some degree, so the median diameter, or 50% size by weight, should be used for d_m. Since field stone cannot be sieved,the median diameter of the stone must be estimated by other means. Measurement of the triaxial dimensions of stones in a representative sample is often used in the field. Field stones are rarely spherical, so care should be taken to insure that individual stone sizes are not overestimated. For example, an ellipsoidal stone with triaxial dimensions of 0.4 m x 0.2 m x 0.1 m has the same volume as a 0.2 m sphere, hence very often the equivalent spherical diameter of a stone may be only half of the maximum dimension. Furthermore, in a graded sample, stone sizes may range from one-quarter of the median diameter up to twice the median diameter. In this case the maximum dimension of the largest stone may be four times the median diameter of the sample.

Graded stone is somewhat superior to uniform stone of the same median diameter. By virtue of decreased void spaces, a graded sample has greater density (giving increased unit weight), greater interlocking effect between particles (giving increased shear strength), and decreased porosity (giving a better filter effect between the flowing water and the base material under the stone).

At the end of the slope the stone should be placed for a length of 6 D_2 into the downstream channel. This is required to accommodate the transition from supercritical flow down the slope back to sub critical flow in the canal. Because the slope is both flat and rough the Froude number for the flow is small, usually 2 or less, and a normal hydraulic jump with a turbulent top roller does not occur. The flow simply expands vertically, through critical depth, from the depth on the slope to the downstream depth D_2.

Example 4:

A discharge of 2 m^3/s must be passed down a roadside ditch with a slope of 6%. The ditch bottom has a width of 1.5 m and side slopes of 3H:1V will be used. Design a riprap lining for this ditch.

A trial and error procedure is required. Assuming y = 0.40 m, then

$$d_m = 10ys = 10 \times 0.4 \times .06 = 0.240 \text{ m}$$

and

$$n = 0.049 \times 0.24^{1/6} = 0.0386$$

The area wetted perimeter and hydraulic radius will be 1.08 m^2, 4.03 m and 0.268 m respectively, from which

$$Q = AR^{2/3}\,S^{1/2}/n = 1.08 \times 0.268^{2/3} \times 0.06^{\,1/2}/0.0386 = 2.99 m^3/s$$

This discharge exceeds the design value, therefore y will be less than 0.4 m.

Assuming y = 0.32 m, then

$$\begin{aligned}
d_m &= 10 \times 0.32 \times 0.16 = 0.192 \text{ m}\\
n &= 0.049 \times 0.192^{1/16} = 0.0372\\
A &= 0.32 \times 1.5 + 3 \times 0.32^2 = 0.787 \text{ m}^2\\
P &= 1.5 + 2\sqrt{10}\,0.32 = 3.52\\
R &= 0.787/3.52 = 0.224 \text{ m}\\
Q &= 0.787 \times 0.224^{2/3} \times 0.06^{1/2}/0.0372 = 1.91 \text{ m}^3/\text{s}
\end{aligned}$$

This discharge is close to the assumed value. Thus, 200 mm riprap in a 300 mm layer should be specified, underlain by either a 200 mm granular filter layer or filter fabric. The actual flow depth will be 0.33 m and the velocity will be 2.43 m/s.

F. GABION MATTRESSES

20. General

Gabions are steel wire mesh containers filled with rock. Tubular shaped wire enclosures were originally used for bank and pier protection as a substitute for riprap. They are often referred to as rock sausages. The word gabion usually refers to rectangular basket shaped enclosures which can be stacked up to form various shapes which may serve as retaining walls, dams, weirs and drop structures. Gabion mattresses, frequently referred to as Reno mattresses, are mattress shaped enclosures which are thinner than gabions and cover a larger area. They are usually intended for bed and bank protection in channels.

Wire mattresses are shipped flat and subsequently unfolded at the site. They are equipped with diaphragms and dividers which divide the mattress into smaller sections. The dividers run parallel to the flow and the diaphragms are placed perpendicular to the flow. The mattresses are placed in the channel, the ends, sides and dividers are bent up and tied with wire ties. The diaphragms are placed and wired, creating flat box-like units. The boxes are filled with stone, and a wire mesh lid is placed on top and wired to the ends, sides, dividers and diaphragms, so the stone is completely enclosed.

Mattresses are available in widths of 1 or 2 m, lengths of 3, 4, 5 or 6 m, and thicknesses of 0.15 m, 0.17m, 0.20 m, 0.23 m, 0.25 m and 0.30 m. If greater thickness is needed, normal gabions can be used, for which thicknesses of 0.5m and 1.0 m are available. Recently, Maccaferri has produced a new design known as the gabion mat. It is similar to the mattress except it is supplied in rolls. When unrolled, a mat will cover an area 30 m long and 2 or 3 m wide. The ends, sides and dividers are already attached to the base, and when the diaphragms are wired on, at 3 m intervals, 1 m by 3 m sections are produced. Usually the time required to install mats is less than required for mattresses.

The advantage of mattresses over riprap is that smaller size stone can be used, the total volume of stone required is greatly reduced, and additional strength is derived from the steel wire mesh. The stability is related to the stability of the whole unit, rather than stability for an individual stone, as in the case of riprap.

21. Stone Size and Mattress Thickness

Movement of stone within the sections of a mattress is initially prevented by the restraining effect of the wire mesh. Experiments on mattresses performed at Colorado State University, reported in 1984, have shown that the first movement of stone within the mattress will not occur until the shear stress is twice as great as the stress which would produce movement in the same size of stone in riprap. Thus, for equal shear stress, the required stone size at first movement will be reduced to half, or

[27] $$d_m = 5\, yS$$

and minimum mattress thickness t must not be less than

[28] $$t = 7.5\, y\, S$$

This thickness will be 1.5 d_m with d_m as given in [27], thus meeting the same minimum thickness specification as for riprap.

The studies at Colorado State University showed that some movement of the stone within the mattress can be permitted without loss of ability to protect the base material. The movement is accompanied by a rearrangement of the stone within the section, in that the layer thickness becomes thinner at the upstream end of the section and thicker near the downstream end. In effect, the downstream portion of the mattress will bulge above the original level at which the stone was placed. According to the tests, the protection is still adequate as long as the minimum depth of stone remaining at the upstream end of the section is not less than d_m. This limit will be reached when $yS = d_m/4$ at which point the layer thickness would be reduced to 5yS at the upstream end of the section and increased to 10yS near the downstream end. The appearance of the rearrangement of the stone within the mattress is illustrated in Figure 12. It is important that the space between the wire mesh diaphragms, which are perpendicular to the direction of the flow, not exceed 1 m, otherwise a greater displacement of stone within a section will occur when $d_m = 4yS$. Also, the mattresses should be tightly packed when the stone is installed.

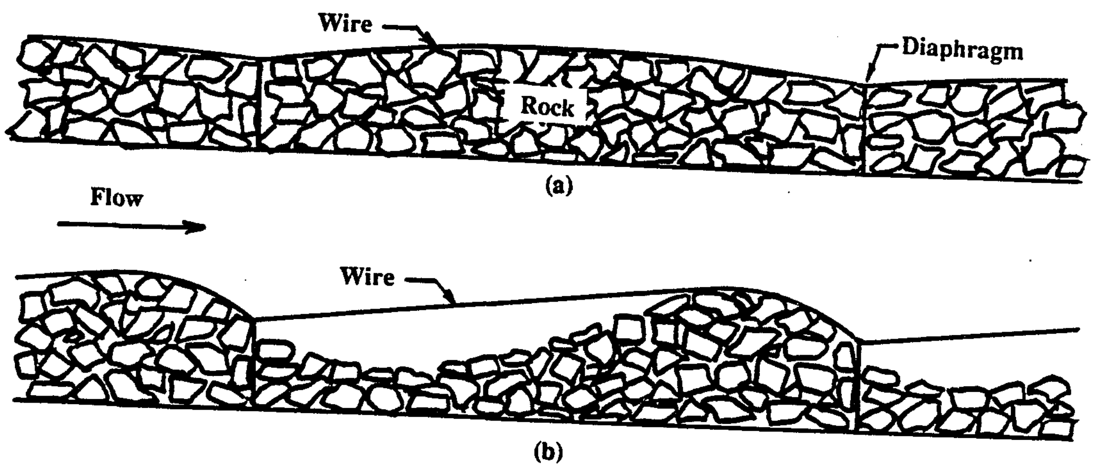

Figure 12. Disposition of Stone in a Mattress Section: a) $d_m = 5yS$ (design condition); b) $d_m = 4yS$

When $d_m = 4yS$, the flow depth would be increased by 25%. An increase in the depth of this magnitude would correspond to an increase in discharge in excess of 40%, which shows that a design based on [27] would have considerable overload capacity before serious problems occurred.

It should be noted that minimum stone sizes are governed to some extent by the size of the openings in the wire mesh. The diamond shaped mesh openings are 60 mm by 80 mm for mattress thicknesses up to 0.30 m, so stone with a size range of 75 mm to 150 mm, with $d_m = 100$ mm (approximately) is desirable.

The need for placement of a granular filter layer and/or filter fabric under the stone is just as important for gabion mattresses as it is for riprap.

Example 5:

Find the maximum slope which can be used to discharge 2 m^3/s in a roadside ditch with a 1.5 m bottom width and side slopes of 3H:1V, given that the available stone size d_m = 100 mm.

The mattress thickness of 150 mm should be selected, giving $t = 1.5\ d_m$. Manning's n will be given by $n = 0.049 \times 0.1^{1/6} = 0.033$. For no movement of stone inside the mattress, $d_m = 5\ y\ S$, or $yS = 0.1/5 = 0.02$. Also, $Q = AR^{2/3}S^{1/2}/n$, or $AR^{2/3}S^{1/2} = 0.066$. Since A and R are both functions of y, then $AR^{2/3}S^{1/2}=0.066$ and $yS = 0.02$ may be solved for y and S, by trial and error, giving $y = 0.300$ m and $S = 0.0666$. In example 4, using riprap for the same conditions, the permissible slope was only 6%, even though the size of stone and thickness of the layer were each twice as great as required for the mattress design.

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Problems

1. Why is it possible to use a higher mean velocity for a larger channel than a smaller channel, without producing erosion, even though the material through which the channel is excavated is the same for both?

2. Why must flatter slopes be used for larger channels than smaller channels where the channels are excavated through erodible material and are not lined?

3. Why is the shear stress at the corner of a rectangular open channel smaller than the shear stress on the bed at the centre of the channel?

4. Why does the n value for a grassed channel decrease as the discharge increases?

5. What are the advantages of a gravel or riprap lining over use of vegetal cover (i.e., a grassed channel).

6. Prove from [9], [13] and [16] that $d_m=1.73\ q^{2/3}\ S^{7/9}$, relating stone size with discharge and channel slope.

7. An emergency overflow spillway for a small stockwatering dam is to be vegetated, and is expected to meet a Class D retardance rating during infrequent use during the spring of the year. The channel will be wide and shallow, with a width of 10m and a maximum design depth of 0.3 m on a 1% slope. Will the shear stress be within the allowable value? What discharge can be passed to satisfy these conditions?

8. The bed grade of a canal designed for 4.75 m^3/s must be lowered 0.9 m very near a gravel deposit containing an abundance of 100 mm (median size) material. What width would have to be used for a stone paved channel if a 4% slope is to be used? (Assume 2-dimensional flow.) What will be the Froude number for the flow at the base of the slope?

9. Stone with a median equivalent spherical diameter d_m = 1250 mm, placed in a 2000 mm layer on a slope of 20%, is to be used for a major spillway. What design unit discharge should be specified? What percentage of this discharge would be passing through the stone layer itself?

CHAPTER IX

CONVEYANCE AND CONTROL STRUCTURES

A. GENERAL

1. Conveyance Structures

Conveyance structures are required to connect reaches of a canal where the continuity is interrupted by topographical features or obstructions. For example, at a ravine the flow must be carried over or across the depression in some manner. At a highway, or other obstruction, the flow must be carried over the top or underneath. These problems may be encountered in the layout of any canal system, whether it be for power, irrigation, or water supply. Structures used to overcome these obstacles include earth fills, flumes, and syphons. Drop structures may also be classified as conveyance structures because they connect two reaches of a canal separated by a change in elevation. Because of their importance, however, drop structures were covered separately in Chapter 6. Although the types of structure vary widely, they are all classified as conveyance structures because they are intended to convey the water between two reaches of a canal.

2. Control Structures

Control structures are structures required to regulate the discharge or water level in a canal or a canal distribution system. As the name implies, there must be some means of control, so control structures generally have some type of gate, stop logs, or flash boards. A distinguishing feature between a control structure and a conveyance structure is that the latter has no means of flow regulation. Frequently, however, these two types of structure may be combined. For example, if both a drop structure and a check structure are required in the same vicinity of the canal, it may be economical to combine these into a single structure, a check-drop,, in order to avoid duplication of wingwalls, abutment walls, and floor slabs.

Control structures include headgates, wasteways, turnouts, and checks. The headgate, which is the first regulating structure on a canal, has already been discussed under Diversion Works in Chapter 5. Wasteways are installed where excess canal flows must be discharged to waste. Checks are built where it is necessary to raise canal water surfaces above natural levels. Headgates and wasteways may be found on all types of water resource projects. Turnouts and checks are used exclusively on irrigation projects.

B. EARTH FILLS FOR CROSSINGS

3. Single Bank Fill

An earth fill may be placed across the depression so as to create a small reservoir or pool which connects the two reaches of the canal. This method, called a single bank fill, is illustrated in Figure 1. The method is simple and gives the lowest head loss. It is suitable if the flooded area is small and the fill is low in height. It cannot be used if the natural runoff in the drainage course is large, because it is not economical to build a spillway to protect the fill against overtopping.

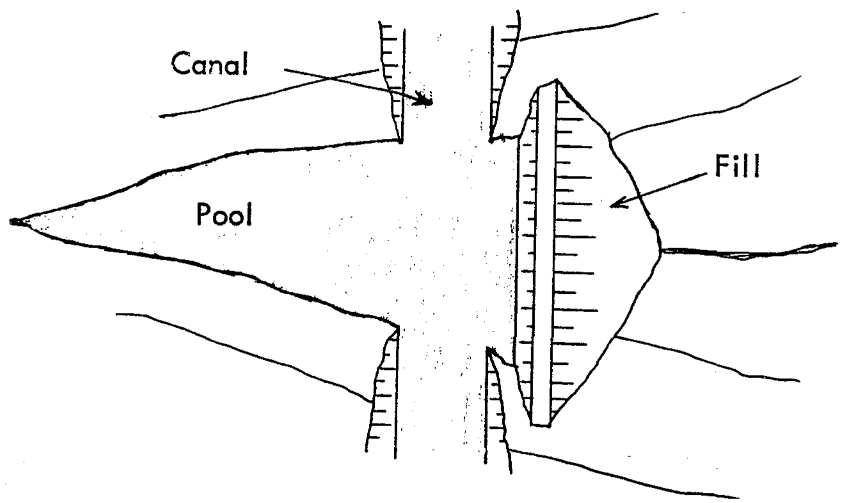

Figure 1. Single Bank Fill

4. Double Bank Fill

If conditions are suitable for using a fill for the crossing, but the grade of the depression is such that a large area would be flooded by the single bank fill method, a double bank fill may be used. In this case the canal goes straight across the depression with the same cross section and grade as in the normal reach, except that it is constructed entirely in fill. It is suitable for a shallow depression where the natural flow in the drainage course can be conveniently handled by a culvert running under the canal. The method is illustrated in Figure 2. It is evident that great care must be taken in the design and construction of a double bank fill in order to insure the stability of the cross section. Since the flow is supported above the adjacent ground level and water table, outward seepage may have to be controlled by lining the section with clay or plastic membranes.

C. FLUME

5. Description

The flume is a free flow structure which is used to convey water straight across a depression or along the side of a steep sidehill where a canal cannot be built. It consists of a short inlet and outlet transition connected by an artificial trough built on or above ground level. A ground level flume built along a sidehill is called a bench flume. The elevated flume requires substructure support in the form of trestle work. The elevated flume is often used where a major watercourse must be crossed, because the large opening under the flume will accommodate large flood flows in the same way that floods are carried under a highway bridge.

Flumes may be constructed of concrete, timber, or sheet metal. Concrete is more permanent, but also more expensive. It is frequently used for bench flumes where the dead weight is not a consideration in substructure design. Timber is suitable for smaller size flumes which must be supported. Concrete and timber flumes are usually rectangular. Semicircular shapes can be constructed with wood staves, as shown in Figure 3. The flow area is a half circle, which is the most efficient cross section, so that actually the section is a little greater than a half circle to allow for freeboard. Creosoting is recommended to increase the useful life of timber and wood stave flumes. Galvanized smooth sheet metal, or half sections of corrugated iron pipe, can also be used for supported flumes.

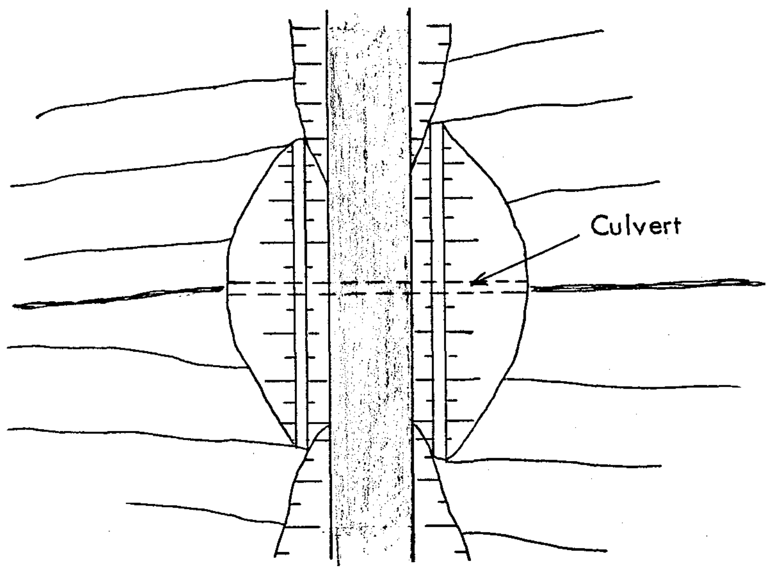

Figure 2. Double Bank Fill

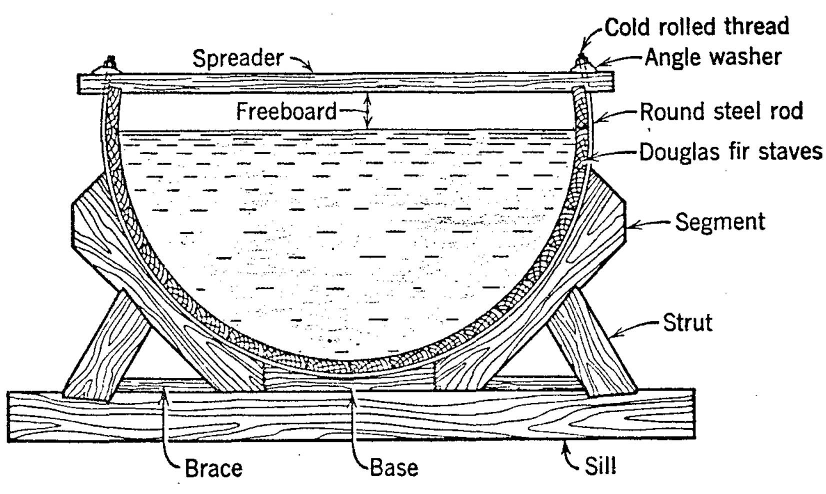

Figure 3. Wood Stave Flume Cross Section

The transitions are usually constructed of reinforced concrete. The inlet transition is used to change the size and shape of the flow cross section between the canal and the flume, and the outlet transition to effect the change from the flume back to the canal. The flume velocity is purposely greater than the canal velocity in order to reduce the size and cost of the flume. The transitions must contain the region of accelerating and

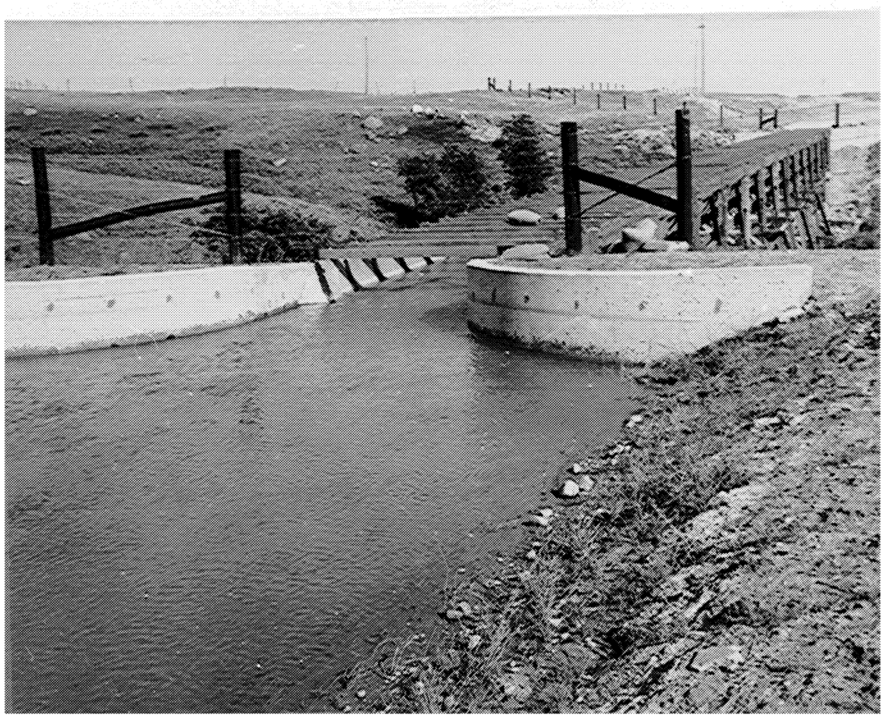

Figure 4. Flume Inlet Transition, Elbow, Saskatchewan

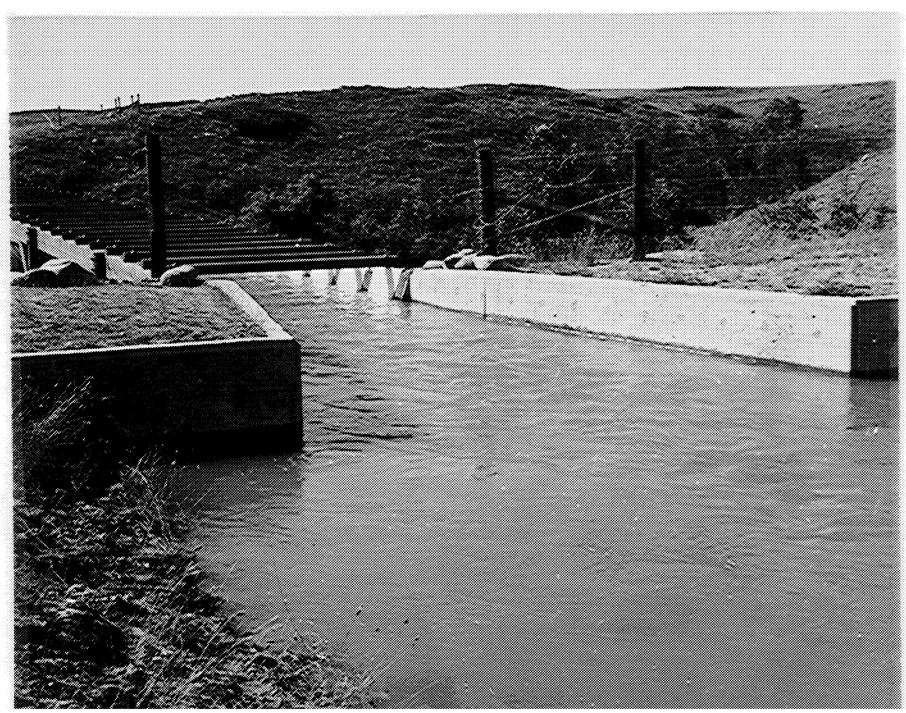

Figure 5. Flume Outlet Transition, Elbow, Saskatchewan

decelerating flow to prevent excessive scour of the canal. Typical reinforced concrete flume transitions for a smooth steel semi-circular flume are shown in Figures 4 and 5.

6. Hydraulic Design

The first requirement of a flume is low head loss. This is true regardless of whether the water is being conveyed for power, irrigation, or water supply. In the first case the loss in head at the flume results in a corresponding loss of power at the plant. In the case of irrigation or water supply the loss in head may have to be made up by subsequent pumping. In order to minimize the head loss the flume should have a well designed inlet and outlet transition, and the flume proper should be smooth and straight. Whenever possible the flume should be designed as a most efficient hydraulic section. For a rectangular section this means that $b/d = 2$, in which b is the flume width and d is the flow depth in the flume. For a given discharge and flume velocity, both the head loss and the amount of construction material are a minimum for the most efficient cross section.

The head loss also depends upon the flow velocity selected for the flume. A low design velocity would give a correspondingly low head loss, but the flume would be more costly because of the large cross-sectional area required. A higher design velocity would decrease the size and cost of the flume, but would increase the head loss. For this reason the selected design must be a compromise. The ratio of V_2/V_1 is called the fluming ratio, where V_1 is the flow velocity in the canal and V_2 is the corresponding velocity in the flume at the same discharge. The fluming ratio is rarely less than 2 nor more than 5. The higher values result in a decrease in the flume cost, but an increase in the head loss and increase in the cost of the transitions at each end. Values of 3 or 4 are common in design.

The second requirement for a flume is that the water surface be smooth and predictable. This feature is also related to the economy of the design, as it affects the freeboard requirement. In order to minimize cost, a freeboard allowance of only 20% of the depth is requently recommended. If the water surface is rough or wavy, the risk of overtopping is increased. Even if the wave heights are less than the freeboard, overtopping could still result during a strong cross wind. In order to obtain a smooth plane water surface in the flume, a streamlined inlet is required. In addition, it is necessary to avoid flow near critical depth, otherwise an undular water surface may develop. A value of $\mathbf{F}_2 \leq 0.7$ should be considered as the maximum for the Froude number for flow in the flume. This value is sufficiently large to permit a fluming ratio up to 5, when desired, although it may be necessary to use a b/d value less than 2 in order to satisfy this requirement.

The flume slope should be set equal to the friction slope at design discharge, as calculated by Manning's equation. In this way the flume will flow at normal depth throughout its length. It is very important that the elevation of the inlet and outlet transitions be properly set with respect to the floor elevation of the flume, in order that both the canal and flume flow at normal depth at design discharge. If this is not done, either the upstream canal or the flume itself may be overtopped before the design discharge is reached.

7. Inlet Transition

In some early designs proposed by Hinds it was common to use a gradually converging warped wall transition to connect the canal and flume sections. Although these inlets are hydraulically efficient, the hydraulic and structural design is complicated and construction is difficult. It is now recognized that a short simple convergence with vertical sidewalls throughout is equally efficient and less costly. A recommended design is shown in Figures 6 and 7.

The transition consists of straight vertical walls converging at an angle of 33° 41' (1 in 1 1/2) to a simple curve which is tangent to the flume sidewall. The tangent distance for the simple curve is 0.5 (B - b), in which B represents the average water width in the approach channel and b the flume width. The transition length is given by

[1] $$L = 1.25\,(B - b)$$

This inlet produces a smooth water surface which is level transversely, and which may be calculated by simultaneous application of the continuity and energy equations at successive sections through the transition. The head loss for the inlet is given by

[2] $$h_L = 0.06\,(1 - b/B)\,V_2^2/2g$$

The floor of the inlet transition is constructed as a plane surface, but not necessarily level. Normally the elevation of the floor of the transition is lower at the entrance to the flume than it is at the canal. The magnitude of this downslope depends upon the cross-section selected for the flume. A narrow deep cross-section will result in more slope than is required for a wider shallower section. The change in the elevation of the floor of the flume may be determined from

[3] $$Z_1 + D + V_1^2/2g = Z_2 + d + V_2^2/2g + h_L$$

in which Z_1 and Z_2 are the floor elevations at the beginning and end of the inlet transition, D and d are the canal depth and flume depth respectively, V_1 and V_2 are the velocities in the canal and the flume, and h_L is the inlet transition head loss.

8. Outlet Transition

The outlet is intended to contain the high velocity and minimize head losses while the flow is expanded from the flume back to the canal. The reduction in velocity is accompanied by a rise in the water surface. This conversion of velocity head back to elevation head is called "recovery". As decelerating flow is inherently unstable, a very gradual transition must be used if the structure is to operate as intended. If flow separation is to be avoided the outlet transition must be considerably longer than required for the inlet transition. Even with a gradual expansion it is not possible to get the efficiency of the outlet as good as for the inlet.

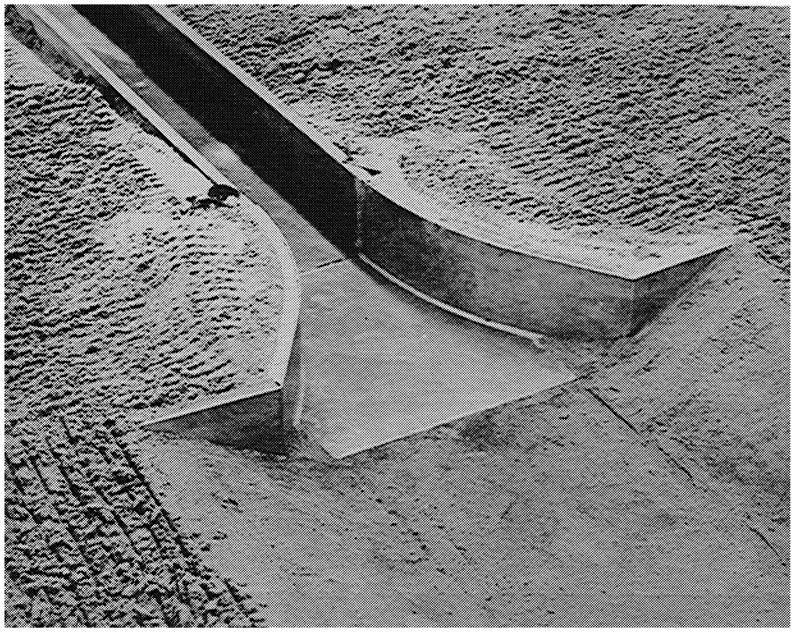

Figure 6. Model of Flume Inlet

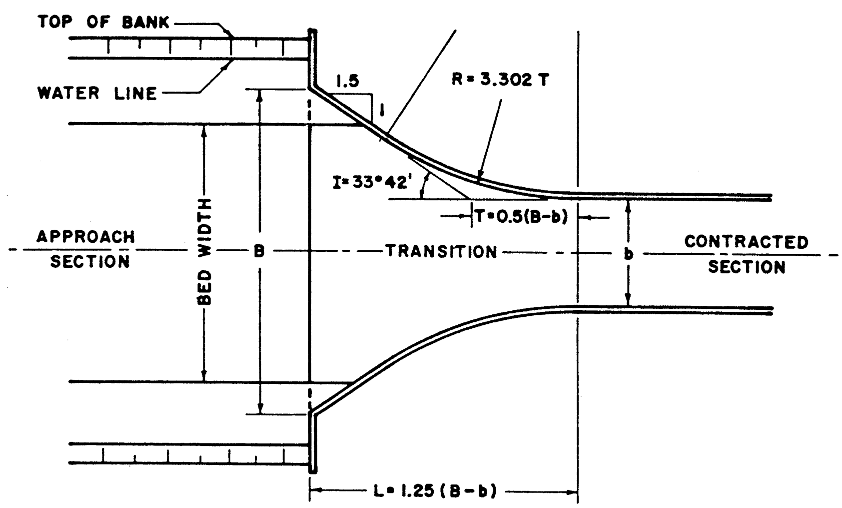

Figure 7. Definition Sketch for Flume Inlet

If separation occurs, the flow may pass through the outlet as a concentrated jet, and the maximum velocity at the outlet may be several times greater than the average canal velocity. The consequent scour may endanger the structure and most of the velocity head will be dissipated in the canal downstream. In this respect it is to be noted that canal scour and high head loss go together. Figure 8 shows the type of separation flow pattern which will develop with a B/b = 3 and 1 in 4 rate of sidewall divergence.

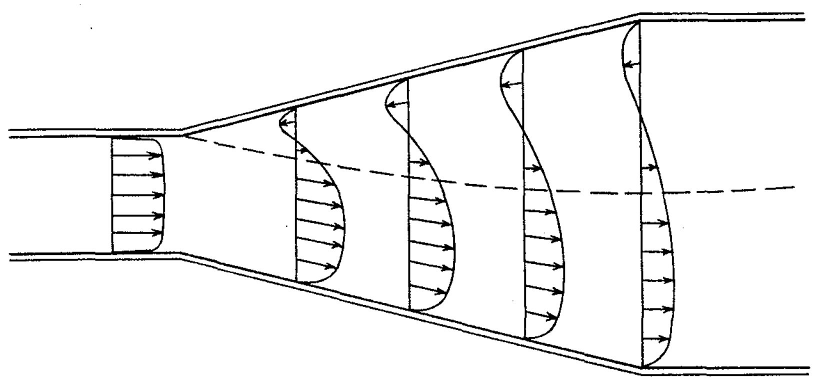

Figure 8. Plan View Velocity Distribution for a Diverging Transition With Flow Separation

Experiments have shown that separation can be avoided in an outlet with straight sidewalls diverging at 1 laterally to 10 longitudinally. At this rate of divergence, however, a very long transition would be required to complete the expansion out to the average width B. The transition length would be four times the length of the inlet transition. A considerable saving in cost can be effected by terminating the transition at a width less than B. This can be justified because most of the velocity reduction and head recovery occurs near the beginning of the transition, and the benefit due to each added increment of length decreases. Since the cost of each added increment increases (due to increasing width), it is good economics to foreshorten the transition. Of course the average velocity at the outlet will be greater than the average canal velocity, but this condition can be resisted with riprap if necessary.

It is recommended that an outlet width of 0.667B be used. The transition length will then be

[4] $$L = 5(0.667B - b)$$

For a B/b ratio of 3, the outlet transition length by [4] would be twice the inlet transition length given by [1]. A further economy could be effected by starting the outlet transition before the far bank of the depression is reached. By substituting part of the transition for part of the flume, the overall length of the flume would be reduced although the length of the outlet transition would be unchanged. This procedure is illustrated in Figure 9.

The head loss for the outlet transition can be calculated from

[5] $$h_L = 0.10\,(1 - b/B)\,V_2^2/2g$$

The floor of the outlet transition is constructed as a plane surface, but again there may be a change in elevation. This may be calculated from

[6] $$Z_2 + d + V_2^2/2g = Z_3 + D + V_3^2/2g + h_L$$

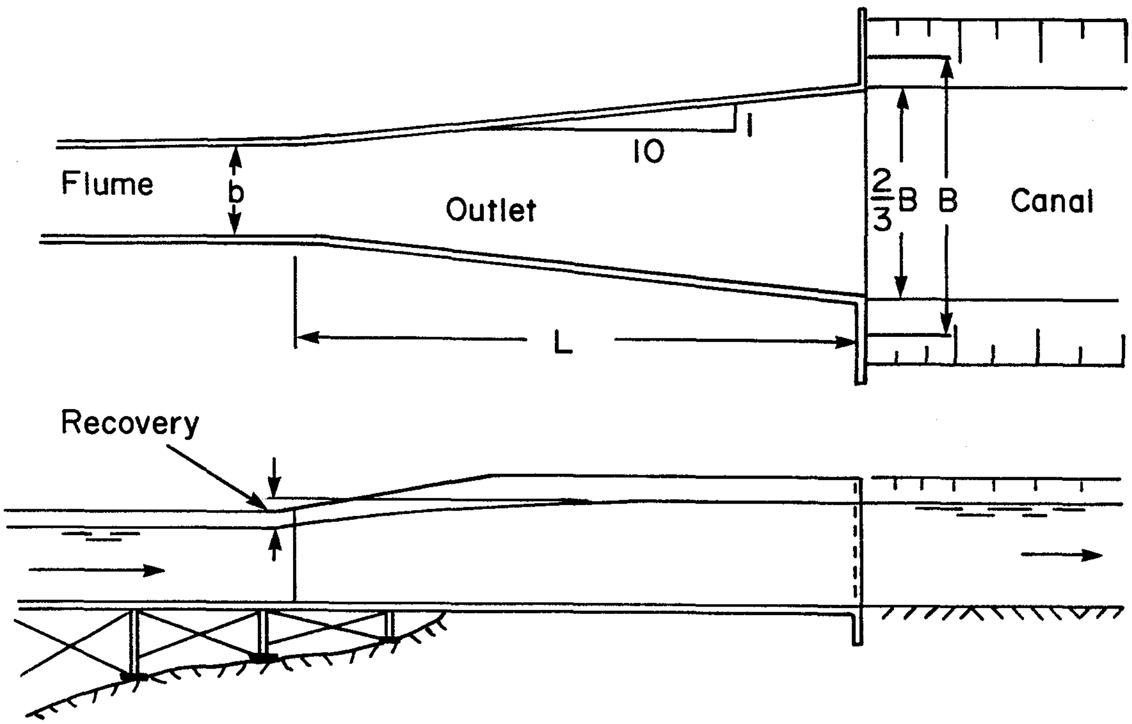

Figure 9. Definition Sketch for a Flume Outlet Transition

in which subscript 3 refers to conditions in the downstream canal. In [6] the values on the left hand side of the equation apply to conditions at the start of the transition and values on the right to the end of the transition, with h_L being the outlet transition head loss. Normally Z_3 will exceed Z_2, indicating an upslope in the transition.

<u>Example 1:</u>

Given $Q = 6.80\ m^3/s$, $b = 2.44$ m, and $d = 1.22$ m for a rectangular flume, and $B = 7.3$ m and $D = 1.37$ m for the canal, calculate the slope of the floor in the inlet transition.

$$V_1 = 6.80/(1.37 \times 7.3) = 0.680 \text{ m/s}$$

$$V_2 = 6.80/(1.22 \times 2.44) = 2.284 \text{ m/s}$$

and the head loss in the inlet, from [2], is

$$h_L = 0.06\ (1 - 2.44/7.3)\ 2.284^2/2g = 0.011 \text{ m}$$

Writing the energy equation between a point in the canal and a point in the flume

$$Z_1 + D + V_1^2/2g = Z_2 + d + V_2^2/2g + h_L \text{ or}$$

$$Z_1 + 1.37 + 0.024 = Z_2 + 1.22 + 0.266 + 0.011$$

Hence the drop in floor elevation

$$Z_1 - Z_2 = 1.497 - 1.394 = 0.103 \text{ m}$$

The length of the transition, from [1], is

$$L = 1.25\ (7.3 - 2.44) = 6.075 \text{ m}$$

and the floor slope = 0.103/6.075 = 0.0170 (dropping)

9. Partial Capacity

As with all hydraulic structure designs, it is desirable to check the hydraulic performance at less than design discharge. This is necessary to insure that an unexpected condition will not develop. Usually it is the sufficient to check the design at one-third and two-thirds design capacity.

At design discharge the water surface elevation at the end of the flume will be lower than the water surface elevation in the downstream canal by the amount of the recovery. This difference will decrease as the discharge decreases because the velocities and the recovery are reduced. Normally this will not cause a problem because the downstream canal depth will also decrease, and the flow depth in the flume will be smaller than at design flow. However, if the downstream depth is maintained by a downstream check, or perhaps held up by some temporary channel blockage, then the depth at the end of the flume will be greater than it was at design discharge. Freeboard allowance must be checked for this possibility.

Further upstream the depth in the flume will be shallower because an M1 water surface profile will form. If the flume is long enough the depth at the upstream end will correspond to the uniform flow depth for that discharge. When [2] and [3] are applied to solve for D in the upstream canal, it may be found that the canal depth is less than normal for that discharge, producing an M2 drawdown curve in the upstream canal. This condition may occur if there is a significant downslope in the floor of the inlet transition, such as would occur if the cross-section selected for the flume is narrow and deep. If the drawdown condition results in scouring velocities in the upstream canal, it may be necessary to riprap the canal in the area approaching the flume. Alternatively a wider and shallower cross-section for the flume may be used. This will reduce or even eliminate the downslope in the floor of the inlet transition, but care must be taken not to exceed the Froude number limitation in the flume.

D. SYPHON

10. Description

The syphon is an enclosed flow structure used to carry water across a depression or under a highway or railway. It consists of an inlet and outlet transition connected by a barrel, as indicated in Figure 10. A syphon is usually more economical than an earth fill or a flume if the depression is wide and deep.

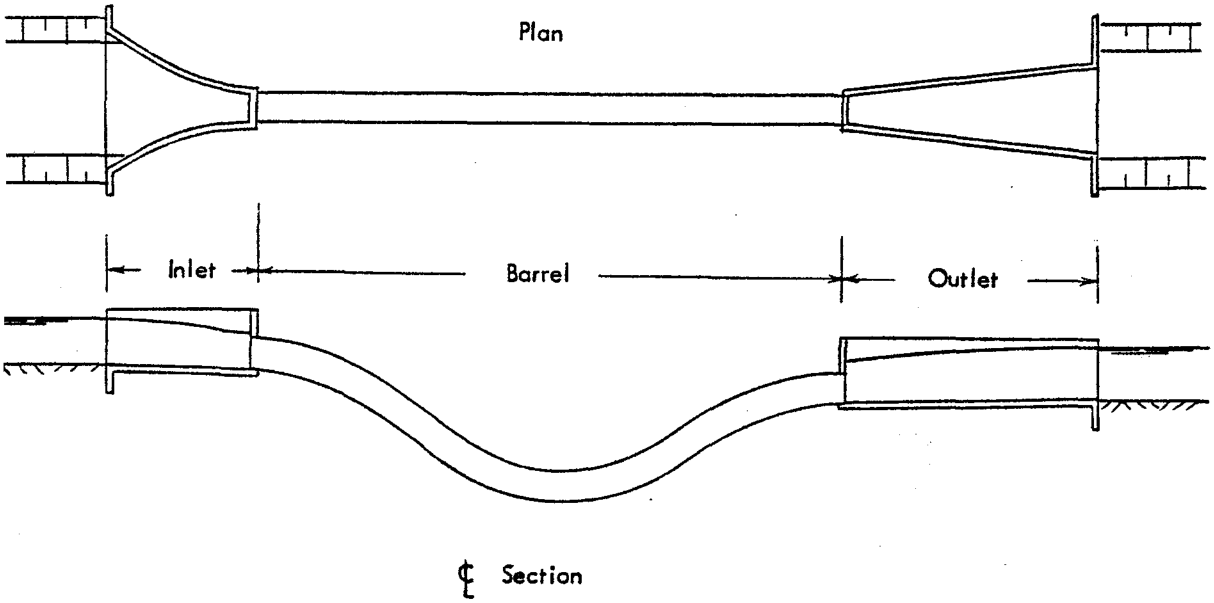

Figure 10. Definition Sketch for an Inverted Syphon

The term syphon is actually a misnomer because the barrel is completely below the hydraulic grade line and pressures are positive throughout. There is no syphonic action involved. The term "inverted syphon" is somewhat more descriptive, but generally the structures have come to be known simply as syphons.

The barrel of the syphon may be precast concrete, cast in place concrete, steel, or wood stave. If the syphon must pass under a railway the external load may be greater than the internal hydrostatic load. Reinforced concrete is usually best suited to this case. If cast-in-place concrete is used the barrel may be square or rectangular. If necessary a multi-barrel unit may be used. Steel is suited for high head installations as tension cracks in concrete may result in leakage, although precast prestressed concrete pipe may also be used. Concrete pipes are usually buried below ground because they will not corrode or rot. Steel or wood stave is usually supported above ground on cradles.

Creosoted wood stave syphons are still used, although precast concrete is now in common use. Wood stave syphons are easy to build using unskilled labour and are competitive in areas where the fir staves may be obtained cheaply. The tongue and groove wood staves are held under compression in a circular section by steel rods or hoops at 10 to 20 cm on centres. The tension in the rods may be adjusted by tightening a nut at the connecting shoe. The barrel must be supported by concrete cradles at intervals of 2 to 4 m. An example of a wood stave syphon is shown in Figure 11.

11. Hydraulic Design

Barrel velocities must be limited in order to insure that the inlet and outlet transition, designed for subcritical velocity, will operate as intended. If the barrel is square a permissible velocity is $V_b = 0.8\sqrt{gb}$, in which b is the inside dimension of the square barrel. This is somewhat higher than allowable flume velocities, but is permitted because the flow is enclosed and freeboard considerations do not apply. High velocities

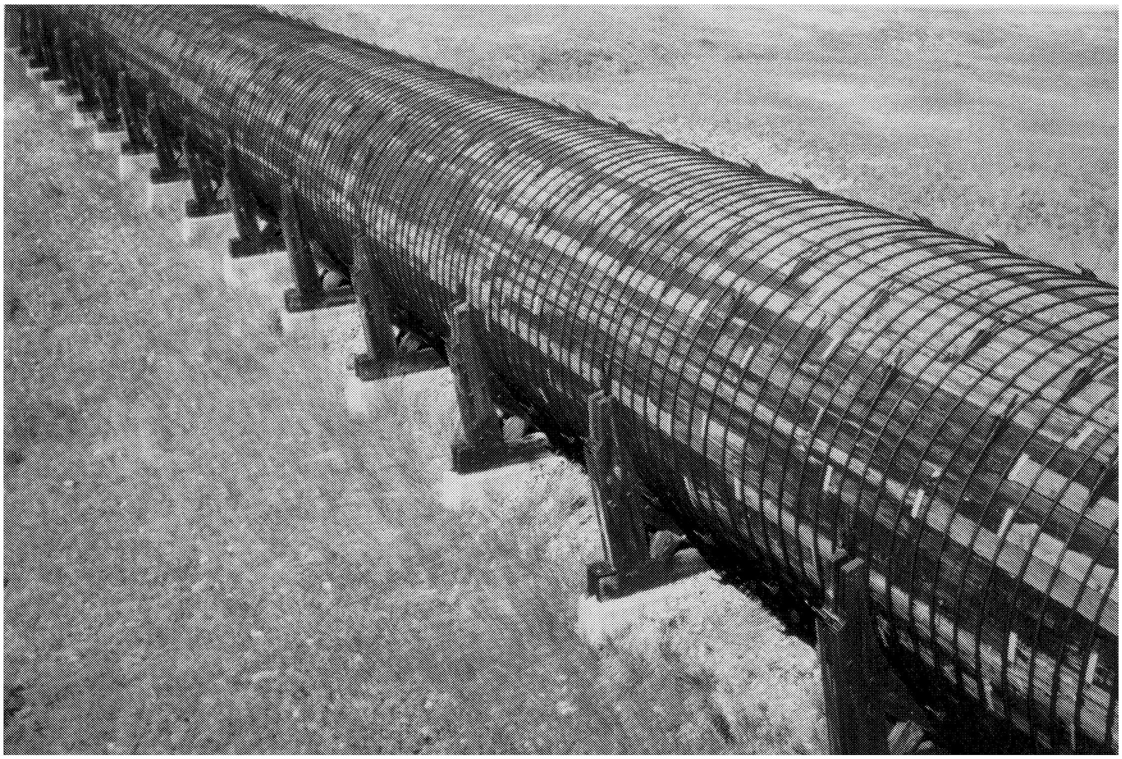

Figure 11. Wood Stave Syphon, Forty Mile Coulee, Alberta

will reduce barrel costs, but if it is necessary to minimize barrel friction loss, velocities much lower than this may have to be used.

Inlet and outlet transitions for the square barrel syphon are designed in exactly the same way as the flume transitions discussed in Sections 7 and 8.

If the barrel is circular, a square to round transition 0.5 D_o in length should be used to connect the inlet transition to the barrel. The side dimension of the square should equal the barrel diameter (i.e. $b = D_o$). At the outlet a round to square transition $1D_o$ in length should be used to connect the barrel to the outlet transition. In this case barrel velocities up to $V_b = \sqrt{gD_o}$ are permissible.

In laying out the syphon it is important that the elevations of the inlet and outlet structure be set to correctly allow for the head loss across the system. If the drop in elevation is more than necessary, the canal depth at the inlet will be lowered and a drawdown condition will develop. This drawdown may result in excessive channel scour. If the drop provided is too small, the syphon will head up and the canal banks may be overtopped near the inlet unless the discharge is reduced to less than the design value. The required drop in elevation of the canal bed and canal water surface across the syphon may be computed from

$$\Delta EL = (K_e + K_o + 2gn^2L/R^{4/3})\, V_b^2/2g \qquad [7]$$

in which K_e and K_o are the inlet and outlet loss coefficients. By analogy to the case for flume transitions, the sum of K_e and K_o may be taken as 0.16 (1 - b/B) for a well designed inlet and outlet. Manning's n is about 0.012 for concrete or wood stave pipe. Frequently the drop in elevation calculated by [7] is increased by 10% for design to make sure the syphon capacity will meet the requirement.

12. Hydraulic Jump

The head loss across a syphon is usually much larger than across a flume. The syphon, often used for a deep depression, may have a greater length, and if the higher permissible velocity is used to save cost the hydraulic radius will be smaller. These factors will increase the inlet, outlet and barrel losses, giving a large drop in elevation. There are many syphon designs with elevation drops as much as 2 or 3 m.

At partial capacity the head loss across a flume may not be much different than at full capacity, since for uniform flow at normal depth the friction loss in the sloping flume is the same at any discharge. However, the head loss across a syphon decreases markedly at partial capacity. The area available for flow in the syphon barrel is constant, and hence the velocity reduces in proportion to the discharge. As shown by [7], at half capacity the value of ΔEL would be only one quarter of the value at the design discharge. Since the tailwater level at the outlet will also be reduced, the required upstream water level needed to pass the discharge may be lower than the bed elevation of the canal. In this case the flow would pass through critical depth at a control point in the inlet transition (usually at the entrance), supercritical flow would enter the inlet of the syphon, flowing partly full, and a hydraulic jump would occur inside the syphon barrel. The jump would be forced at the position where the syphon barrel flows full.

The consequences of this hydraulic condition are twofold. First, there will be a significant drawdown in the canal approaching the syphon. If this cannot be permitted, either due to excessive velocity or insufficient depth to meet upstream demands, a check structure will be required at the syphon inlet. Radial gates are commonly used for this purpose, and they may be installed in a parallel sided section immediately preceding the inlet transition. The gates will be partially closed at reduced discharge, producing a supercritical jet through the transition and into the syphon barrel up to the position of the hydraulic jump.

The second factor is that since the hydraulic jump occurs in a closed section, the air entrained by the jump will be trapped momentarily in the barrel. Air bubbles rise to the crown of the pipe and are acted upon by two forces. The drag force due to the flowing current tends to move the bubbles down the slope further into the pipe. The buoyant force (actually a differential hydrostatic pressure) tends to move the bubbles up the slope toward the inlet. When the bubbles are small drag forces dominate. However, the air bubbles collecting at the crown coalesce and grow into larger bubbles. A point may be reached where a large flat air bubble several metres long may remain stationary at the crown of the pipe as the forces are in balance. This stationary bubble is joined by other smaller bubbles moving downstream, increasing the volume, until buoyant forces dominate and the entire bubble rapidly moves up the slope and vents to atmosphere. This condition is known as blowback, and is accompanied by violent surging in the hydraulic jump. Jumps in enclosed sections tend to be more unsteady than free jumps, even without blowback, because of the loss of freedom for vertical oscillation of the surface in the region of full flow. The result is that the syphon barrel near the inlet is subjected to repeated pounding by dynamic forces. Wood stave syphons actually flex under the influence of these forces. Extra barrel strength is needed in this area.

If the distance to the bottom of the syphon is short, some of the entrained air may be carried beyond the bottom of the syphon and escape to the outlet. Since drag and buoyancy act in the same direction in the rising part of the syphon, any air bubbles which collect along the crown of the pipe travel rapidly up the slope to the outlet. This blowby condition may be accompanied by a waterspout at the outlet, and water slapping against the crown of the pipe behind the exiting bubble.

Usually the blowback condition at the inlet is simply tolerated, but it can be mitigated with air vents. A series of holes may be drilled in the pipe along the crown in the vicinity of and just downstream from the calculated jump location. These may be connected to a common header running along the top of the pipe to a single standpipe just behind the inlet portal. Any air bubbles which collect at the crown escape through the holes and are vented to atmosphere through the header. This procedure minimizes blowback and makes the jump less unsteady. The air vents must be spaced about one half a barrel diameter apart and have a size of 0.05 diameters.

13. <u>Hydraulic Jump Location</u>

In order to define the region near the syphon inlet where extra barrel strength is recommended to offset dynamic forces associated with an enclosed hydraulic jump, or to locate the position of an air vent system, the jump position must be determined.

The definition sketch for a hydraulic jump in a sloping pipe is shown in Figure 12. In this figure, V_1 and d_1 represent the velocity and depth of the inflow jet, D is the pipe diameter, V_2 is the downstream velocity (equal to $Q/(0.785D^2)$ when the pipe is flowing full), h is the height of the hydraulic grade line (HGL) above the center of the pipe at the end of the jump and h_j represents the vertical height between the HGL at the end of the jump and the intersection of the toe of the jump at the inflow water surface. A feature of the hydraulic jump in a sloping pipe is that, except at small flows, the surface roller reaches the crown of the pipe upstream from the end of the jump. Thus, much of the return flow in the surface roller occurs in contact with the crown. Since the HGL continues to rise as the jet expands and the velocity head decreases, the pipe becomes pressurized downstream, reaching a pressure head of h at the end of the jump.

Although the momentum equation can be written for this flow regime, showing the balance of pressure forces and momentum forces, this exercise does not lead to a useful solution of the problem except for a level pipe. It is necessary to determine h_j to locate the position of the start of the jump (at the toe). As was the case for the jump on a continuously sloping chute, discussed in Chapter 6, Part E, Reservoir Inlet Structures, h_j can be determined only by experiment. It is apparent that h_j will be affected by any change in the discharge, pipe diameter, pipe slope or inflow depth, as in

[8] $h_j = f(Q, D, d, S)$

or, non-dimensionally as,

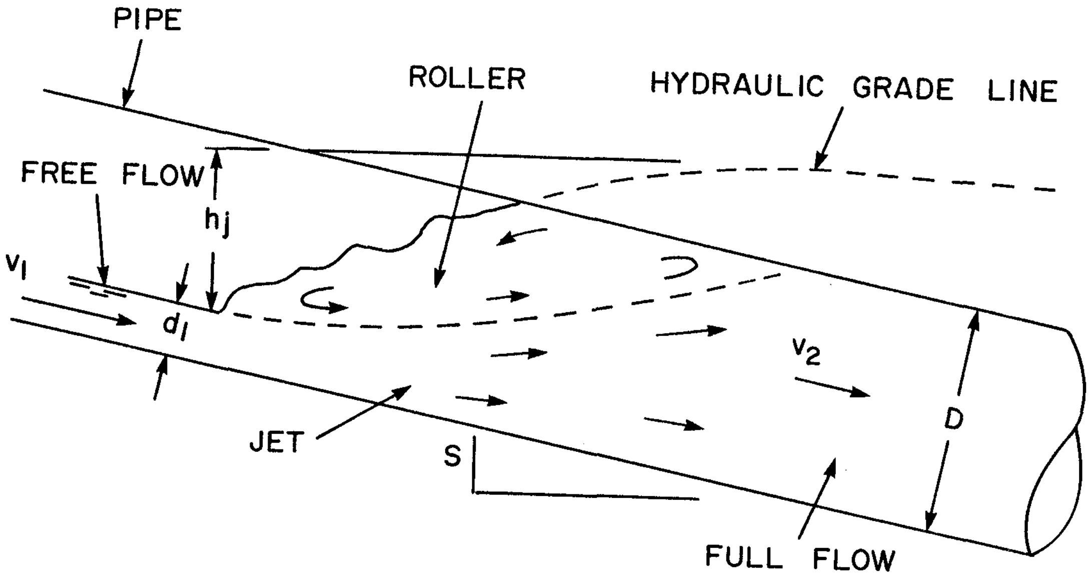

Figure 12. Definition Sketch for Hydraulic Jump in a Sloping Pipe

[9] $h_j/D = f(d_1/D, \mathbf{F}_1, S)$

in which $\mathbf{F}_1$ is the Froude number for the inflow

[10] $\mathbf{F}_1 = V_1/\sqrt{gD_{AV}}$

The value D_{AV} is referred to as the average depth, which is smaller than d_1, and is equal to the flow area A_1 divided by the top width (i.e., surface width T_1). Hence, [10] may be written as

[11] $\mathbf{F}_1 = Q\sqrt{T_1}/(\sqrt{g}A_1{}^{3/2})$

If the jump is close to the inlet at the upstream end of the pipe, d_1 may be greater than the uniform flow depth in the pipe (normal depth) given by Manning's equation. In this case, d_1 would have to be determined by evaluating the S2 water surface profile, starting with critical flow at the inlet. Once d_1 is determined, values of A_1 and T_1 may be determined from Table 1.

Non-dimensional charts for the relationship expressed in [9] were determined by Smith and Haid, and are shown in Figures 13, 14 and 15. These figures apply to d_1/D ratios of 0.2, 0.3 and 0.4 respectively. Since it is unlikely that the d_1/D ratio in practice will agree exactly with these 3 values, interpolation would be required, and extrapolation to d_1/D values of 0.1 and 0.5 could be done if required.

TABLE 1
AREA AND SURFACE WIDTH OF CIRCULAR SECTION FLOWING PARTLY FULL

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{T}{D}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{T}{D}$
0.01	0.0013	0.1990	0.51	0.4027	0.9998
0.02	0.0037	0.2800	0.52	0.4127	0.9992
0.03	0.0069	0.3412	0.53	0.4227	0.9982
0.04	0.0105	0.3919	0.54	0.4327	0.9968
0.05	0.0147	0.4359	0.55	0.4426	0.9950
0.06	0.0192	0.4750	0.56	0.4526	0.9928
0.07	0.0242	0.5103	0.57	0.4625	0.9902
0.08	0.0294	0.5426	0.58	0.4724	0.9871
0.09	0.0350	0.5724	0.59	0.4822	0.9837
0.10	0.0409	0.6000	0.60	0.4920	0.9798
0.11	0.0470	0.6258	0.61	0.5018	0.9755
0.12	0.0534	0.6499	0.62	0.5115	0.9708
0.13	0.0600	0.6726	0.63	0.5212	0.9656
0.14	0.0668	0.6940	0.64	0.5308	0.9600
0.15	0.0739	0.7142	0.65	0.5404	0.9539
0.16	0.0811	0.7332	0.66	0.5499	0.9474
0.17	0.0885	0.7513	0.67	0.5594	0.9404
0.18	0.0961	0.7684	0.68	0.5687	0.9330
0.19	0.1039	0.7846	0.69	0.5780	0.9250
0.20	0.1118	0.8000	0.70	0.5872	0.9165
0.21	0.1199	0.8146	0.71	0.5964	0.9075
0.22	0.1281	0.8285	0.72	0.6054	0.8980
0.23	0.1365	0.8417	0.73	0.6143	0.8879
0.24	0.1449	0.8542	0.74	0.6231	0.8773
0.25	0.1535	0.8660	0.75	0.6319	0.8660
0.26	0.1623	0.8773	0.76	0.6405	0.8542
0.27	0.1711	0.8879	0.77	0.6489	0.8417
0.28	0.1800	0.8980	0.78	0.6573	0.8285
0.29	0.1890	0.9075	0.79	0.6655	0.8146
0.30	0.1982	0.9165	0.80	0.6736	0.8000
0.31	0.2074	0.9250	0.81	0.6815	0.7846
0.32	0.2167	0.9330	0.82	0.6893	0.7684
0.33	0.2260	0.9404	0.83	0.6969	0.7513
0.34	0.2355	0.9474	0.84	0.7043	0.7332
0.35	0.2450	0.9539	0.85	0.7115	0.7142
0.36	0.2546	0.9600	0.86	0.7186	0.6940
0.37	0.2642	0.9656	0.87	0.7254	0.6726
0.38	0.2739	0.9708	0.88	0.7320	0.6499
0.39	0.2836	0.9755	0.89	0.7384	0.6258
0.40	0.2934	0.9798	0.90	0.7445	0.6000
0.41	0.3032	0.9837	0.91	0.7504	0.5724
0.42	0.3130	0.9871	0.92	0.7560	0.5426
0.43	0.3229	0.9902	0.93	0.7612	0.5103
0.44	0.3328	0.9928	0.94	0.7662	0.4750
0.45	0.3428	0.9950	0.95	0.7707	0.4359
0.46	0.3527	0.9968	0.96	0.7749	0.3919
0.47	0.3627	0.9982	0.97	0.7785	0.3412
0.48	0.3727	0.9992	0.98	0.7817	0.2800
0.49	0.3827	0.9998	0.99	0.7841	0.1990
0.50	0.3927	1.0000	1.00	0.7854	......

In design, it would first be necessary to locate the HGL, starting with the tailwater level at the outlet and working upstream through the outlet structure and syphon barrel. The toe of the jump will occur where the water surface of the inflow jet is h_j below the HGL.

It should be noted that when the inflow depth is shallow, say for a smaller discharge, the jump may be completed as a free surface jump. This condition would apply to most of the plot on Figure 13, for example, because the sum of the jet depth d_1 and jump height h_j would be less than the pipe diameter (i.e., $d_1/D + h_j/D < 1$).

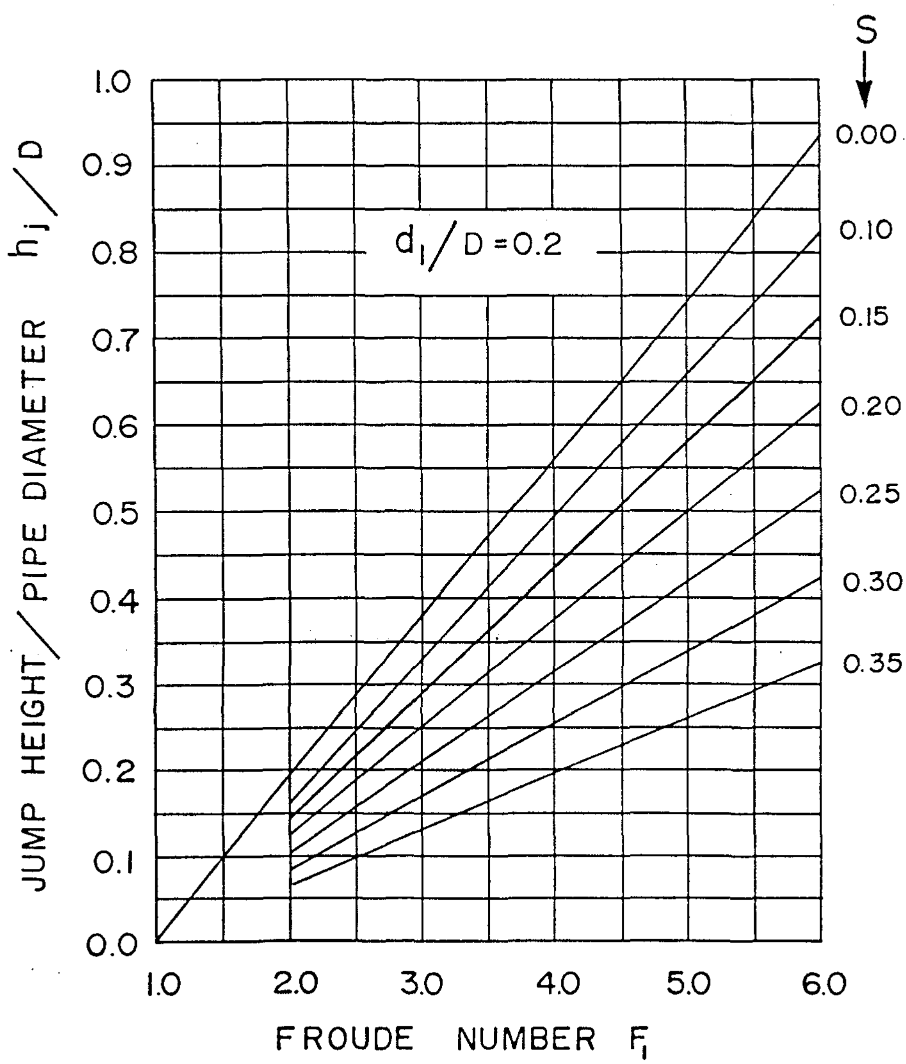

Figure 13. Design Curves for h_j/D vs. F_1 and S for $d_1/D = 0.2$

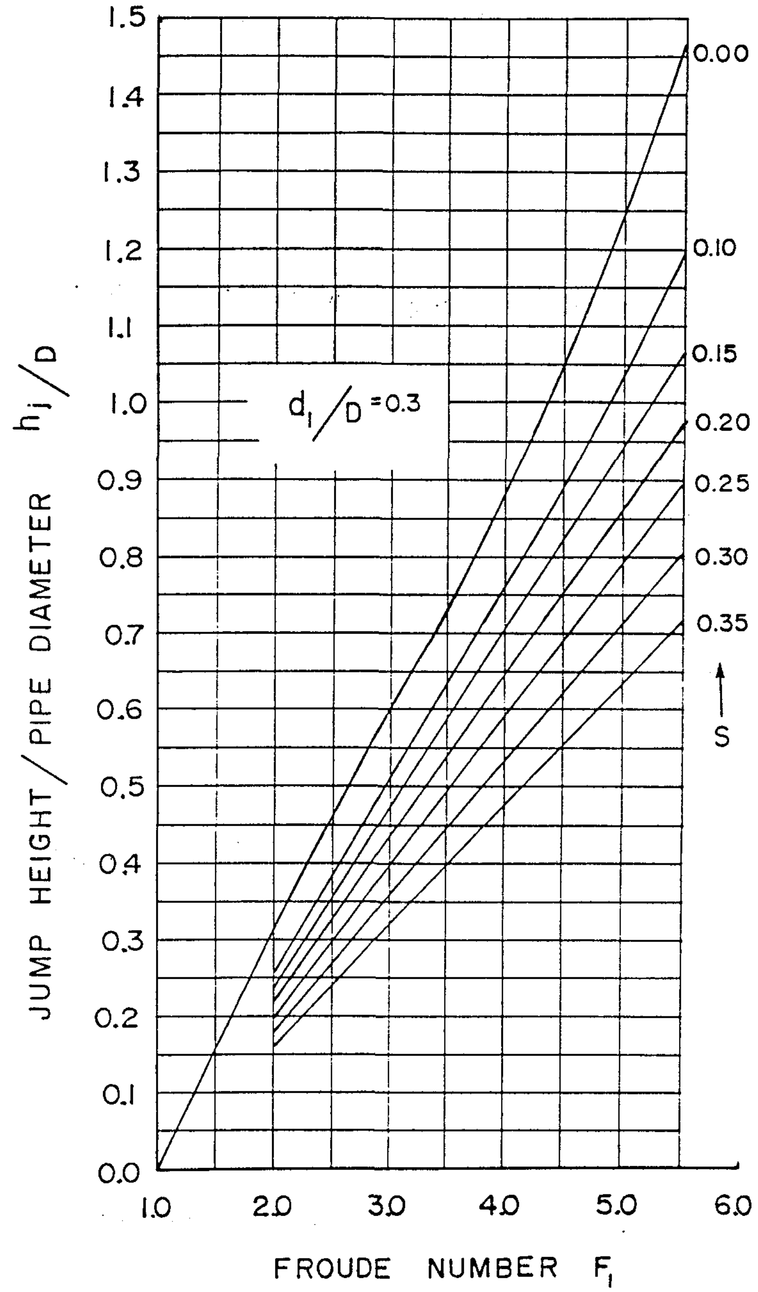

Figure 14. Design Curves for h_j/D vs. F_1 and S for $d_1/D = 0.3$

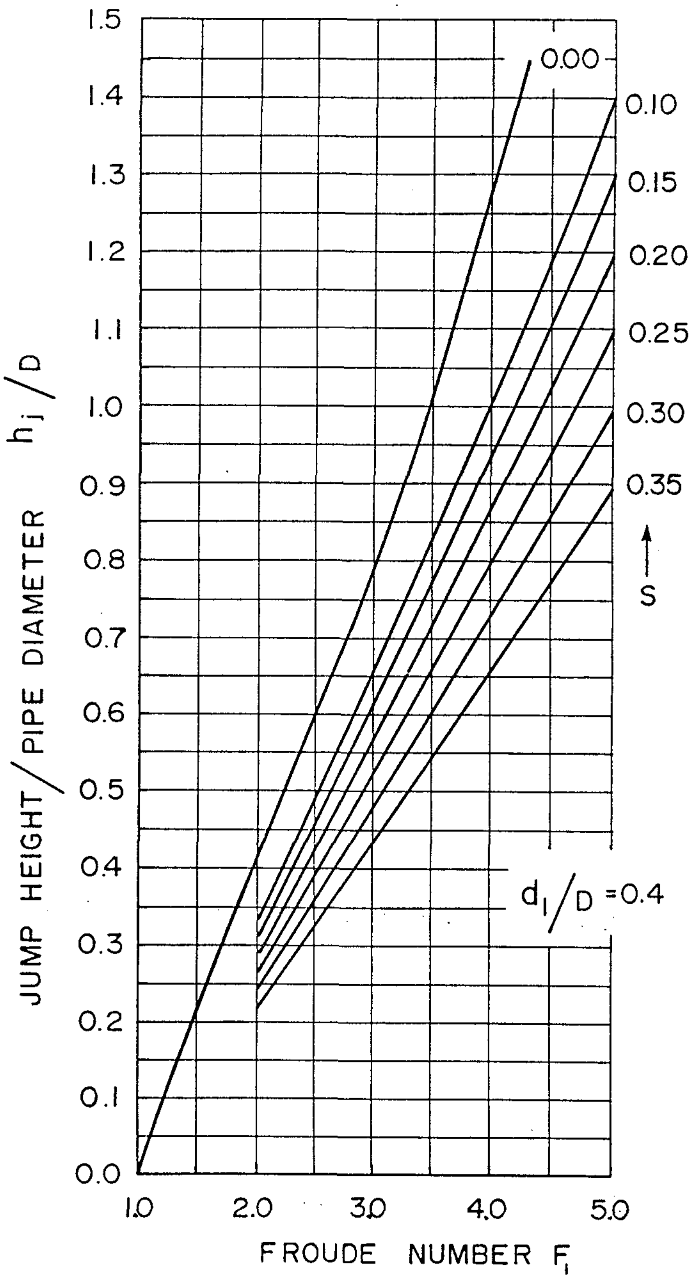

Figure 15. Design Curves for h_j/D vs. F_1 and S for $d_1/D = 0.4$

E. WASTEWAY

14. Purpose

A wasteway is a structure designed to dispose of excess canal discharge. Excess canal discharge may result where considerable runoff may enter the canal following a cloudburst. Since the canal cannot be regulated in advance to compensate for unforeseen runoff of this type, an overload may result. On irrigation systems a block of users may shut down after a rain and when the canal is running at full supply, and an overload may result further downstream. There is also the possibility of a failure in the canal or a canal structure so that the canal will have to be suddenly closed upstream to prevent further damage. The flow above the shutoff point would have to be diverted.

A wasteway must always be located at the dead end of a canal system, and must be capable of discharging the full canal flow at that point. On a long canal it is desirable to have other wasteways at intervals along the canal. In particular it is desirable to have a wasteway immediately upstream from a syphon or enclosed flow structure. The head on such a structure varies as the square of the discharge so it cannot take much overload without overtopping the banks.

15. Alternative Designs

Since a wasteway may have to operate on short notice at any time, there should be some automatic discharge feature incorporated into the design. Many different methods have been used. A rectangular box inlet may be used for the wasteway inlet. The box would project into the canal with the crest elevation set at the normal canal operating level. Alternatively a fixed side weir, called a lateral spillway, may be installed in one of the canal banks parallel to the canal. A serious disadvantage of either of these methods is the limited capacity. A relatively small percentage of the canal flow is discharged to waste. In fact in order for the water level to rise above the weir crest, the residual flow in the canal must exceed the normal canal discharge, otherwise there would be no head on the wasteway weir crest.

The use of automatic float controlled gates is one of the most convenient controls for a wasteway. The gates may be silled at the bed level of the canal, and when fully open will pass the entire canal discharge to waste. A radial gate is usually used in order to keep the opening and closing torque to a low value. Gate motion is actuated by a system of floats and counterweights, and is completely automatic, independent of any external power source.

A schematic of the Armco automatic gate is shown in Figure 16. The upstream canal water level can be controlled at any desired elevation as determined by the setting of the level control located in the upstream canal. The level control is a telescoping pipe with a funnel shaped inlet which may be raised or lowered by a D-handle. The pipe is connected to a float well with an overflow pipe located at mid-height. The weight of the gate and half submerged float are exactly balanced by a counterweight, and the system is in equilibrium with no water flowing into the funnel of the level control. If the upstream canal level rises, the level in the float well will also rise. Due to increased buoyant force on the float, the gate will open and the float will rise until it again reaches the half submerged position. The increased gate opening prevents a further rise in the upstream level. If the level drops below the funnel the float well will drain through the overflow pipe and the gate will return to a closed position. The float well is equipped with a separate low level drain to facilitate cleanout and allow draining to prevent damage due to freezing in winter.

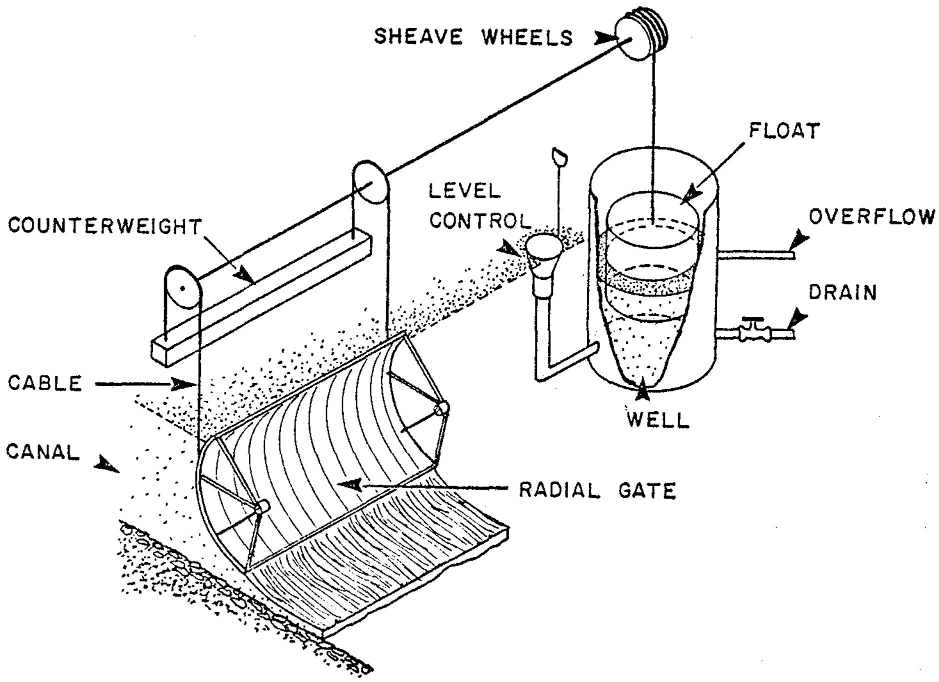

Figure 16. Automatic Gate for Upstream Level Control

A conduit placed over the canal bank, called a syphon spillway, may also be used for a wasteway. The barrel is usually rectangular in cross section. A typical design is shown in Figure 17. The syphon may be designed to prime with only a small head rise in the canal. Once primed the syphon will operate at full capacity until the canal water level returns to normal. The syphon is then broken by air admission and the discharge ceases. Unlike the fixed crest overflow structures, the syphon discharge capacity is largely independent of the depth in the canal. The unit discharge may be calculated by the classical discharge equation

[12] $$q = CB\sqrt{2gh}$$

The United States Bureau of Reclamation has determined values of the coefficient C ranging from 0.6 to 0.9.

The nappe vent shown in Figure 17 is a vertical pipe, open at each end, to connect the crown and invert of the barrel downstream from the crest. It is instrumental in insuring rapid priming of the syphon. Upon initial water level rise in the canal, weir flow occurs over the crest. The nappe separates from the inside curve and springs to the outside curve of the barrel, sealing off the air space in the top of the syphon. As the nappe plunges into the pool standing in the vertical leg of the barrel, air is entrained and discharged to the outlet. The development of subatmospheric pressure under the nappe assists in evacuation of the air from the crown of the syphon by drawing it through the nappe vent. Without the nappe vent the nappe could cling to the inside curve, reducing its capacity to entrain air and requiring a greater canal depth to prime the syphon.

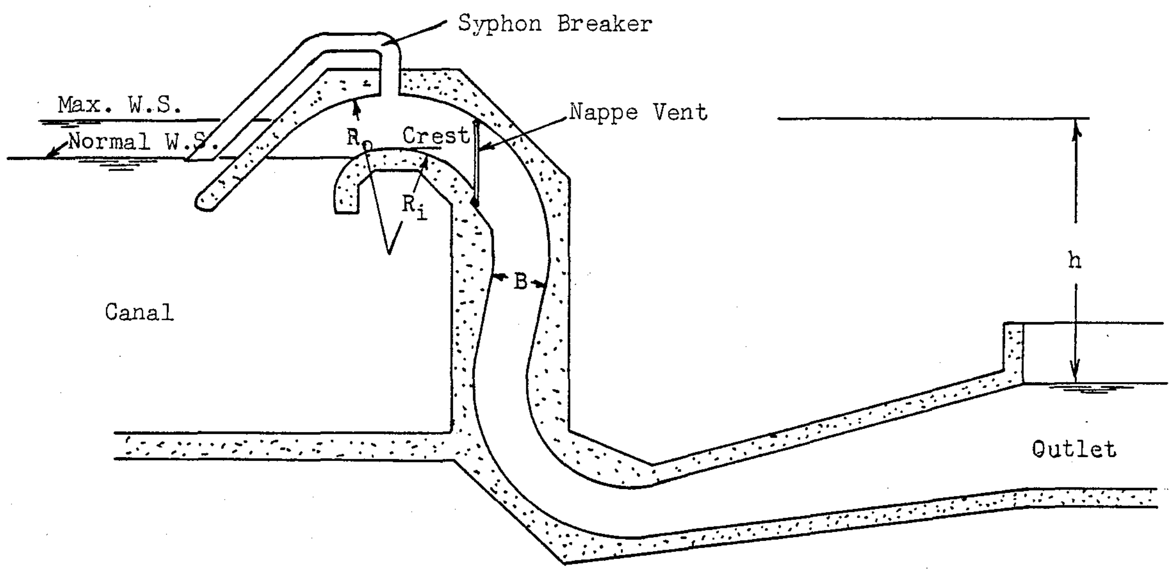

Figure 17. Syphon Spillway

Syphonic action depends upon the development of negative pressure in the barrel. The point of lowest pressure in the system is at the crest of the upper bend. Although this is not the point of highest elevation, it is the point of greatest velocity head. The velocity at this point may be calculated in a manner similar to that used for flow around vertical bends, as discussed in Chapter 3, that is

[13] $$v_i = q/(R_i \ln (R_o/R_i))$$

Because this type of structure is a low head low capacity structure, and discharges for short periods at infrequent intervals, it is permissible to design for quite low pressures. Crest pressures of - 4 m have been used on some designs.

The disadvantage of a syphon is the rather complicated design and construction, and the fact that since the discharge increases very rapidly there is little time for the tailwater to develop. This could complicate the design of the energy dissipating device at the outlet.

F. TURNOUT

16. <u>Description</u>

A gravity irrigation distribution system is laid out similar, in some respects, to a city waterworks distribution system. Main laterals branch off from the main canal, sub-laterals branch off the laterals, farm laterals branch off the sub-laterals, and finally the flow is diverted from the farm lateral to the field. At each point of diversion a regulating structure must be installed. This is necessary because the demand for water varies from area to area as well as from time to time. It is not possible to secure the required degree of control with only a few main structures.

Diversions from the main canal are made through a lateral turnout. Since these are still relatively large structures, designed for perhaps 10 m^3/s, they are constructed of reinforced concrete and are usually equipped with radial gates for easy operation. Most lateral turnouts are open or free flow structures.

Sub-lateral turnouts, which are designed for smaller flows, often less than 3 m^3/s, may be constructed in timber or reinforced concrete. Control may be by radial gates, slide gates, or stop logs. In smaller sizes slide gates are often more economical than radial gates, and they can be hand operated without difficulty. Sub-lateral turnouts may also be designed as enclosed flow structures (i.e., conduits). Usually conduit type installations must be multi-barrel in order to get the necessary discharge capacity and still keep the head loss small. Multi-barrel conduits may be concrete box sections or several corrugated steel pipes, each with their own gate.

Farm turnouts, which are designed for 0.2 or 0.3 m^3/s, may be small timber structures with stoplog control or corrugated steel pipes with hand operated slide gates at the inlet. Pipes are most common because of the ease of installation. A typical pipe size is 500 mm diameter.

Turnouts are low head structures; however, there will always be some energy to be dissipated. In the large concrete turnouts a hydraulic jump stilling basin may be required. In small pipe turnouts adequate protection can be obtained with riprap on the bed and banks of the ditch near the outlet.

G. CHECK

17. <u>Description</u>

An irrigation check is a structure built in a canal for the purpose of maintaining the upstream water level. The check is intended to maintain the upstream level at a time when canal flows are small and the natural depth would be too shallow to divert the required flow through the turnouts along the canal. This situation is common during the early years of a project when it is not yet operating at full capacity, and will occur anytime when the canal flow is reduced during periods of lower demand.

Just as there are many sizes of turnout, so also are there many sizes of check. Simple and inexpensive timber, metal, or canvas dams can be used to check up the water levels in a farm lateral. Small metal or canvas dams which are portable can be removed and transported to other locations on the ditch. A simple portable check for a concrete lined ditch is shown in Figure 18. It consists of a trapezoidal shaped metal leaf with centre pivot and is operated like a butterfly valve.

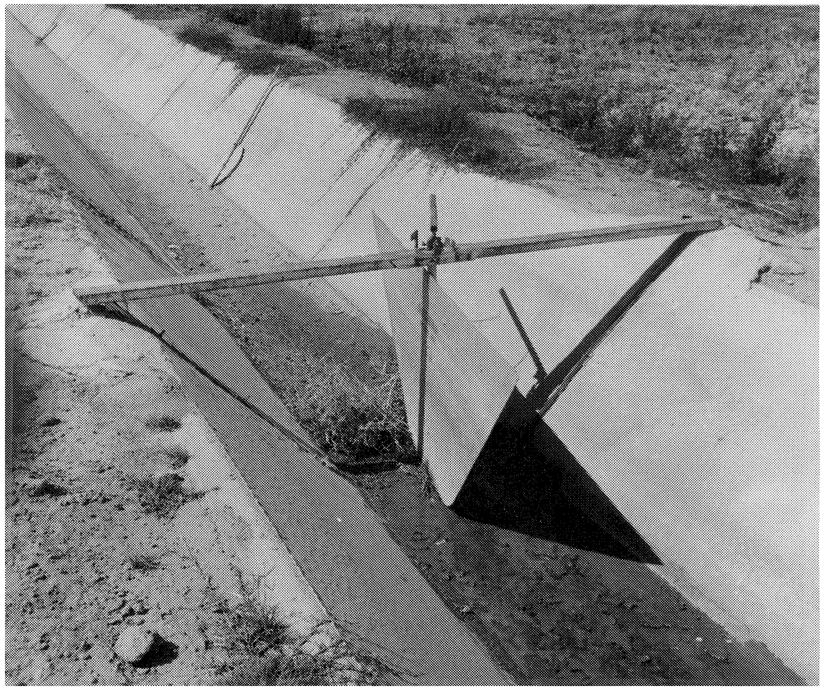

Figure 18. Small Butterfly Ditch Check

Large checks on main canals are usually reinforced concrete structures equipped with radial gates for easy operation. A typical structure is shown in Figure 19, as used on the South Saskatchewan Irrigation Project. This structure consists of a central wide span with a single radial gate, flanked by two narrow spans with stoplog control. The use of both stoplogs and a radial gate in the same structure is intended to take advantage of the desirable features of each. The gate permits rapid easy adjustment for large changes in discharge, and the overflow sections help maintain the desired water level due to small changes in discharge during the operators absence.

Basically the check consists of an entrance section, some means of control, and a stilling basin. The control may be by stop logs or gates, or in some cases a combination of both. The overflow type of check utilizes stop logs or bottom hinged gates for control, and the underflow type utilizes either slide or radial gates.

In a simple check the control is normally wide open when the canal flow is at design discharge. Velocities through the structure are low. If a small drop is incorporated into the design, the upstream level must be maintained above the downstream level, even at full capacity flow. A check with a small drop included may be designed as a breastwall check.

The net width W of the check between sidewalls should not be too small because excessive lateral contraction has a tendency to produce cross currents and eddies at the outlet, increasing the erosive effect. At full capacity flow, the velocity through the check should not be appreciably greater than in the canal. A width of $2.3\sqrt{Q}$, or a width equal to the average width of the canal, whichever is smaller, will be adequate.

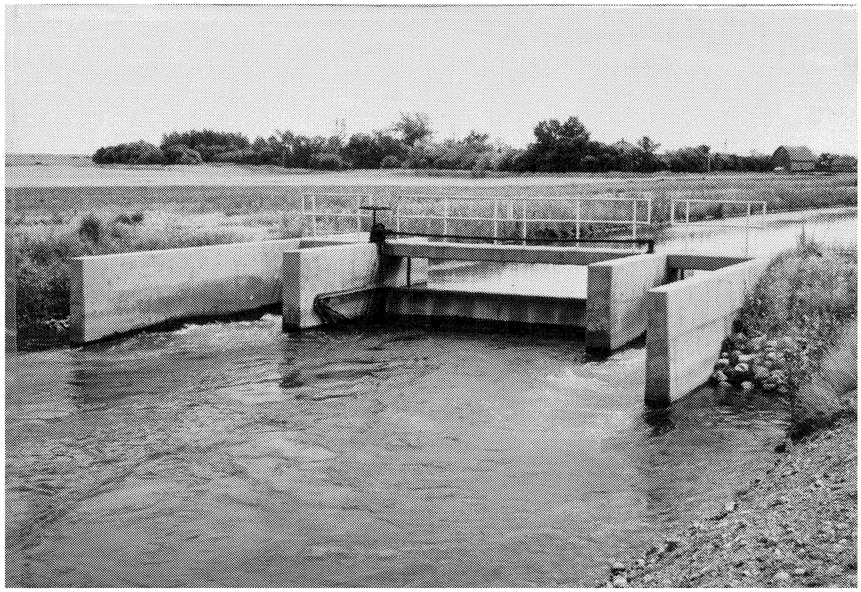

Figure 19. Radial Gate Check

18. The Overflow Check

In the overflow check the water flows over the top of an adjustable barrier, producing weir type flow. The barrier may be stoplogs or a bottom-hinged leaf gate. The upstream water level may be adjusted simply by raising or lowering the crest of the weir.

In the stoplog check the stoplogs are placed in vertical slots or recesses in the piers or abutments, thus forming a vertical weir. Elevation adjustment of the crest is limited to increments of the stoplog height, ususally about 100 mm. The stoplog check is simple and inexpensive, but since the logs must be placed manually by one operator, often while water is flowing over the check, the maximum span is limited to about 2 m. This would require many piers for a wide structure.

The hydraulic action of the overflow check is illustrated in Figure 20. During low flows when the greatest number of logs are in position, the structure operates exactly like a vertical drop structure, as discussed in Chapter 6. Hydraulic performance is good. If the discharge is increased a few logs must be removed to maintain the upstream depth and allow for the increase in head. However, the tailwater depth also increases as the discharge increases, and a point will be reached where the nappe will not plunge to the floor, but instead it will become supported by the tailwater. This condition will normally result for discharges between 60% and 90% of design. The supported jet is usually supercritical, and undular standing waves of considerable amplitude may develop. These waves will extend downstream well beyond the length of a normal stilling basin. The combined effect of wave action and eddy action adjacent to the surface jet will result in some attack on the canal banks. It may be found that extensive riprapping will be necessary to give the required protection. The addition of basin blocks to the floor of the check will not improve this condition because the surface jet will not pass through them.

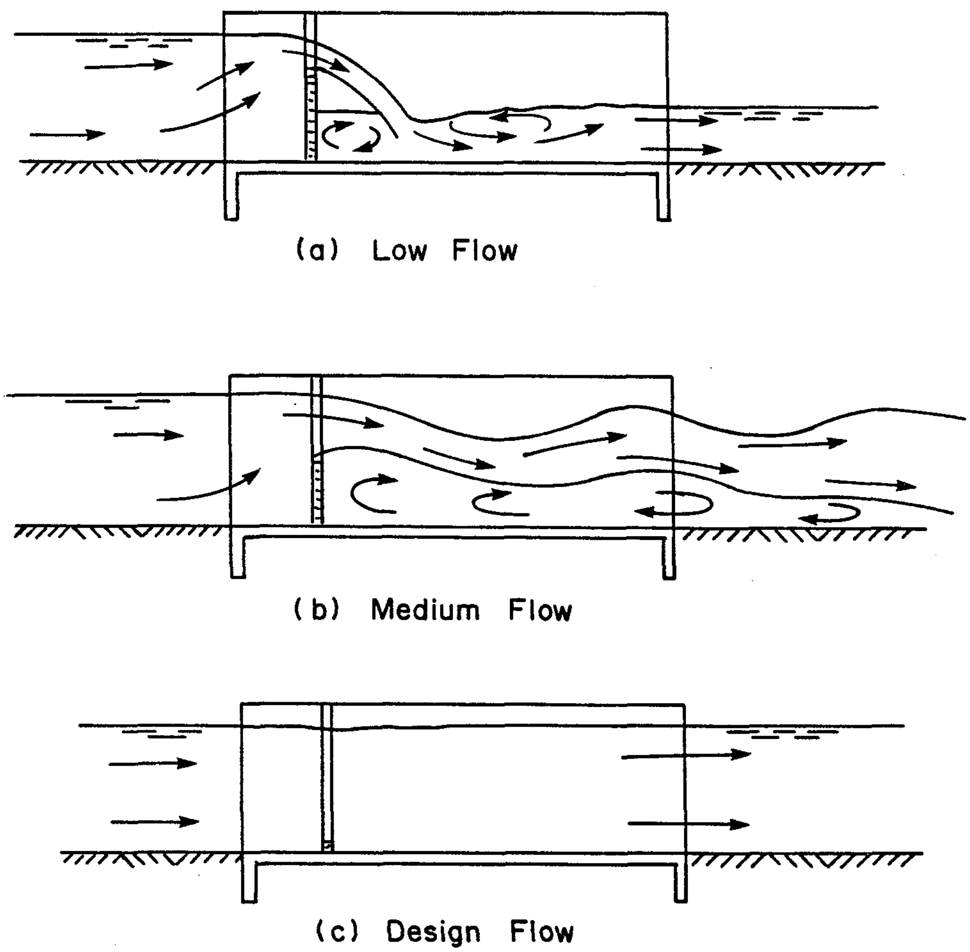

Figure 20. Hydraulic Action of the Overflow Stoplog Check

At design flow where no checking action is required, all the logs may be removed. In this case the velocities through the structure are low and hydraulic performance is good. The head loss across a wide open check may be only one or two centimetres.

The bottom-hinged leaf gate consists of a steel skin plate, reinforced as necessary to carry the hydrostatic load, which is hinged to the floor of the check structures. The gate leaf is sloping with the top downstream, forming an inclined weir. The crest elevation is adjusted by changing the angle of inclination of the weir by a manually operated hoist. A drum type hoist with flexible steel lift cable may be used because the gate will open under the influence of the hydraulic load on the gate leaf if the cable is unwound. The cable is always in tension, for either opening or closing.

Upstream water level adjustment with the bottom-hinged leaf gate is not limited to increments of height, as in the case of stoplogs, because the crest can be located at any desired position. Adjustment is usually somewhat easier and less time consuming than with stoplogs. However, the addition of hardware increases the cost of the structure. Wider spans may be used, although these will be limited by hoist capacity requirements. There will also be a small continuous leakage through the clearance space at the sides of the gate, and at the hinge. If the canal discharge contains any weeds or grass these often tend to clog at points of leakage, and may have to be removed periodically.

The hydraulic action of the inclined bottom-hinged leaf gate is almost identical to the vertical stoplog type. At small discharges true weir flow will occur. At very large discharges the gate leaf may be almost flat on the floor of the check and will be completely submerged. At intermediate discharges the hydraulics will be that of a submerged weir, again producing an undulating supported jet which produces wave and eddy action in the discharge channel.

In summary, the advantages of the overflow check are simplicity, good control of the upstream water level and ability to pass floating debris. The disadvantages are limited span for manual operation, poor hydraulics at intermediate discharges and the tendency to trap bed load sediment in front of the weir.

19. The Underflow Check

In the underflow check the water flows underneath the barrier, producing sluice gate type flow. The gate is usually a radial gate. The upstream level is rasied or lowered by closing or opening the gate. Much wider spans may be used than in the case of the stoplog check. A 6 m gate can be operated manually with ease, and a 15 m gate equipped with an electric motor has been used for a large check in California. The hydraulic performance of the undershot check is good at all discharges. This is due to the fact that the discharge occurs as a floor jet for all operating conditions, and this can be treated effectively with floor baffles. The underflow design will also allow bed load to be passed through the structure. A definition sketch for the underflow check appears in Figure 21.

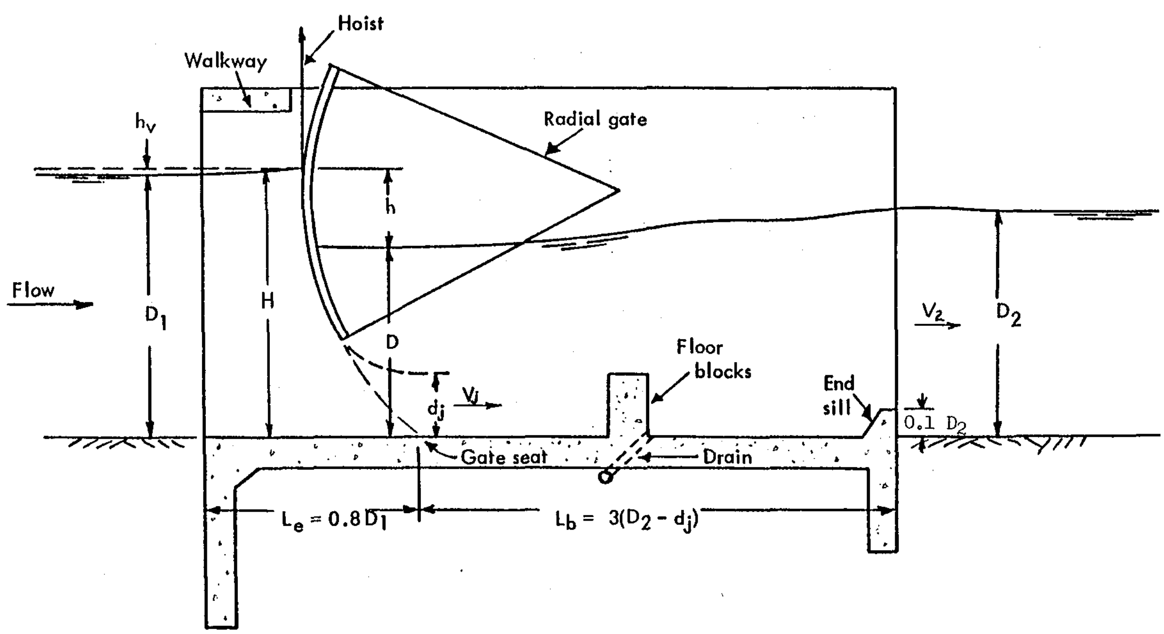

Figure 21. Section of an Undershot Radial Gate Check

The principal disadvantages of the radial gate check are the expense, the tendency to trap floating debris, and the difficulty of controlling the upstream water level. Of course, the problem of control could be solved with an automatic gate, but this would add further to the cost. The reason for the difficulty in controlling the upstream water level is due to the fact that with sluice type or submerged orifice type flow the head varies as the discharge squared. Any fluctuation in canal discharge, which may occur due to a rainstorm or an unannounced change in upstream demands by users, is accompanied by a significant response in water level upstream from the check. Also there is a non-linear inverse relationship between the gate opening and the upstream water level. The result is that considerable operator experience is needed in order to avoid overcorrecting when making gate adjustments.

Hoist capacity for the radial gate check is smaller than for the bottom-hinged leaf gate check. Also side seals may be used to eliminate leakage at the sides of the gate. The advantages of ease of operation, large span, and good hydraulic performance often dictates the selection of the underflow check for the larger structures. The advantages of good water level control may govern for smaller farmer operated structures, and the overflow check is used.

When the check is operated partially open, underflow checks tend to trap floating debris and overflow checks tend to trap moving bed load. Either condition could be considered an asset if it was desired to exclude this debris or sediment from the downstream distribution system, but it would be necessary to periodically clean out the area in front of the check. The debris or sediment could also be flushed downstream during periods when the structure is wide open.

A short approach length preceding the radial gate is desirable to protect the upstream canal banks and bed, and to establish two-dimensional flow at the control. Two-dimensional flow at this point permits more accurate discharge determination, gives better stilling basin action, and simplifies the hydraulic design. It may be shown by a flow net analysis that the flow pattern is practically normal at a length of D_1 upstream from the control. It is not necessary, however, that the inlet length be as great as this. In the first place the upstream depth is frequently checked up to the point where approach velocities are below permissible values; and secondly, it is safe to allow some acceleration of the flow outside the structure. A length $L_e = 0.8\ D_1$ is recommended. This is measured from the upstream cutoff to the gate seat.

The need for a stilling basin on a spillway or drop has long been recognized where erodible materials are concerned. The need for a stilling basin on a check is less obvious. Checks have been built which are little more than a framework to support the check control. The energy due to the drop across the check is largely spent in the generation of eddies in the channel downstream. Such designs may be tolerable in some cases, as for small structures, but normally it is desirable to achieve some measure of velocity reduction before the flow is released to the channel. When it is considered that a drop of only 0.3 m will result in a velocity of 2.43 m/s, and that the canal may sustain only 0.6 to 0.8 m/s without eroding, the desirability of velocity reduction becomes evident. The stilling basin is the part of the structure where this is brought about.

The design of the basin for an underflow radial gate check presents an interesting problem. At design discharge when the gate is wide open the flow is tranquil throughout and a stilling basin would not be required at all. At almost zero flow a small submerged hydraulic jump would form downstream from the gate, but this could be contained in a very small basin. It is evident from this fact that the worst condition for design must occur somewhere between zero flow and maximum flow. A rational approach to the design is possible if the worst condition is taken as the one for which the change in momentum of the flow through the structure is a maximum. Model studies have shown that a design based on this condition will be satisfactory for any other condition as well.

The change in momentum flux through the structure is $q\rho V_j - q\rho V_2$, in which the first term is the momentum of the jet passing under the gate and the second term is the momentum at the end of the structure. This difference also represents the force needed to decelerate the flow to velocity V_2. When $q = 0$ the momentum change is also zero. When q is a maximum the gate is wide open and the velocity through the check is constant. In this case $d_j = D_1 = D_o$ and $V_j = V_2$, and again the momentum change is zero. It may be determined by calculation that when q is approximately 2/3 of the canal design discharge, the difference $q\rho V_j - q\rho V_2$ will be a maximum. This is the flow condition for which the stilling basin must be designed.

The hydraulic jump equation does not apply to the case of basin design for a check. At low flow the downstream depth D_2 will exceed the value d_2 calculated from the jump equation. At larger flows when the drop in water level across the structure is small the jet velocity will become subcritical. In either case the tailwater will submerge the jet at the gate, as indicated in Figure 21. The basin length L_b is intended to include the length over which the jet expands from a depth of d_j to the depth D_2. On a plain level floor this length is about 6 $(D_2 - d_j)$, however, it is sufficient to use a length of 5 $(D_2 - d_j)$ for design. This length can be further reduced if floor baffles are used. Accordingly, for a plain basin

[14] $$L_b = 5\,(D_2 - d_j)$$

or for a basin equipped with floor blocks

[15] $$L_b = 3(D_2 - d_j)$$

In [14] and [15] D_2 and d_j refer to the downstream depth and the jet depth under the gate corresponding to the discharge for which the momentum change is a maximum.

The desirability of using basin blocks in chute and vertical drop structures is well established. Their use has not generally been associated with the design of checks. In principle, however, the value of blocks in the check structure is at least as great as for other structures. Floor blocks are one of the most effective means of decelerating the submerged floor jet. The height of the basin blocks should be equal to d_j. If $d_j < 0.2\ D_2$, the block width should equal the block spacing. If d_j exceeds 0.2 D_2, then the space between the blocks should be increased accordingly, otherwise the blocks will obstruct too much of the flow area. The upstream face area of the blocks should never exceed one-tenth of the downstream flow area.

Evidently the basin design depends upon an evaluation of both D_2 and d_j. The discharge corresponding to maximum momentum change may be taken as 2/3 of the canal design discharge. The value of D_2 at this discharge can be readily determined by applying Manning's equation to the downstream canal elements. It will still be necessary to find d_j, and this requires the simultaneous solution of the energy, continuity and momentum equations, written as:

[16] $$H = D_1 + V_1^2/2g$$

[17] $$h = H - D$$

[18] $$V_j = 0.98\sqrt{2gh}$$

[19] $$d_j = q/V_j$$

[20] $$\gamma D^2/2 + q\rho V_j = \gamma D_2{}^2/2 + \rho q V_2$$

Equation [20] is the momentum equation written from the position of the contracted jet d_j downstream from the gate, to the end of the structure. The equation applies only to the case of a basin without floor baffles. If floor baffles are used, the added drag force must be added to the right hand side of the equation. The reference velocity for the drag equation will be about $0.9V_j$ for this situation, and in combination with a drag coefficient greater than unity and a block height, width, and spacing of d_j, the drag may be represented by $d_j V_j^2/4$, and [20] would become

[21] $$\gamma D^2/2 + q\rho V_j = \gamma D_2{}^2/2 + \rho q V_2 + \rho d_j V_j^2/4$$

Given q and D_1, the corresponding values of V_1, D_2 and V_2 may be calculated and [16] to [20] may be reduced to one equation with D as the unknown. Once D is determined d_j can be evaluated for use in [14].

<u>Example 2:</u>

The gates of a radial gate check are set to maintain an upstream depth of 1.81 m when the unit discharge is 0.88 m^3/s/m. The depth D immediately downstream from the gate is 0.9 m. If the canal discharge increases 20% during the operators absence, by what percentage will the upstream depth increase? What would be the corresponding change if a stoplog check was used?

When the discharge increases the canal depth downstream and the depth in the structure downstream from the gate will also increase. In the absence of a more precise analysis it may be assumed that these depths increase in proportion to the square root of the discharge. Hence, the new depth downstream from the gate will become $0.9\sqrt{1.2}$ = 0.986 m. Since the gate opening remains unchanged, the head h across the gate will also increase from 1.81 - 0.9 = 0.91 m to a value of 0.91 x 1.2^2 = 1.310 m, and therefore D_1 = 0.986 + 1.310 = 2.296 m. This is an increase of 27%.

In the case of the stoplog check, the head would be $(0.88/1.837)^{2/3}$ = 0.613 m, for which approximately 1.81 - 0.61 = 1.20 m of stoplogs would be in place. This exceeds the downstream depth so the structure would operate as a free flow vertical weir without submergence. When the discharge increases 20% the head would increase to 0.613 $(1.2)^{2/3}$ = 0.692 m, and the upstream depth would become 0.69 + 1.20 = 1.89 m. This is an increase of 4.4%.

20. <u>Breastwall Check</u>

If a small drop in the canal is needed in the vicinity of a check it is possible to combine the check and drop in a single structure and thereby save duplication of wingwalls and floor slab. If the drop is not too large a structure similar to the one in Figure 21 but with a higher gate could be used. In this case the upstream level must be held above the downstream level even at full capacity flow, so the gate will never be fully opened.

An alternative design is to substitute a fixed wall, called a breastwall, for a portion of the gate height. In this case advantage is taken of the velocity produced by the drop and a smaller gate opening can be used. This type of structure, shown in Figure 22, is called a breastwall check. Relatively small top seal radial gates can be used for the opening. In fact a small drop may be incorporated intentionally into the check design in order to realize the saving made possible by the smaller gates.

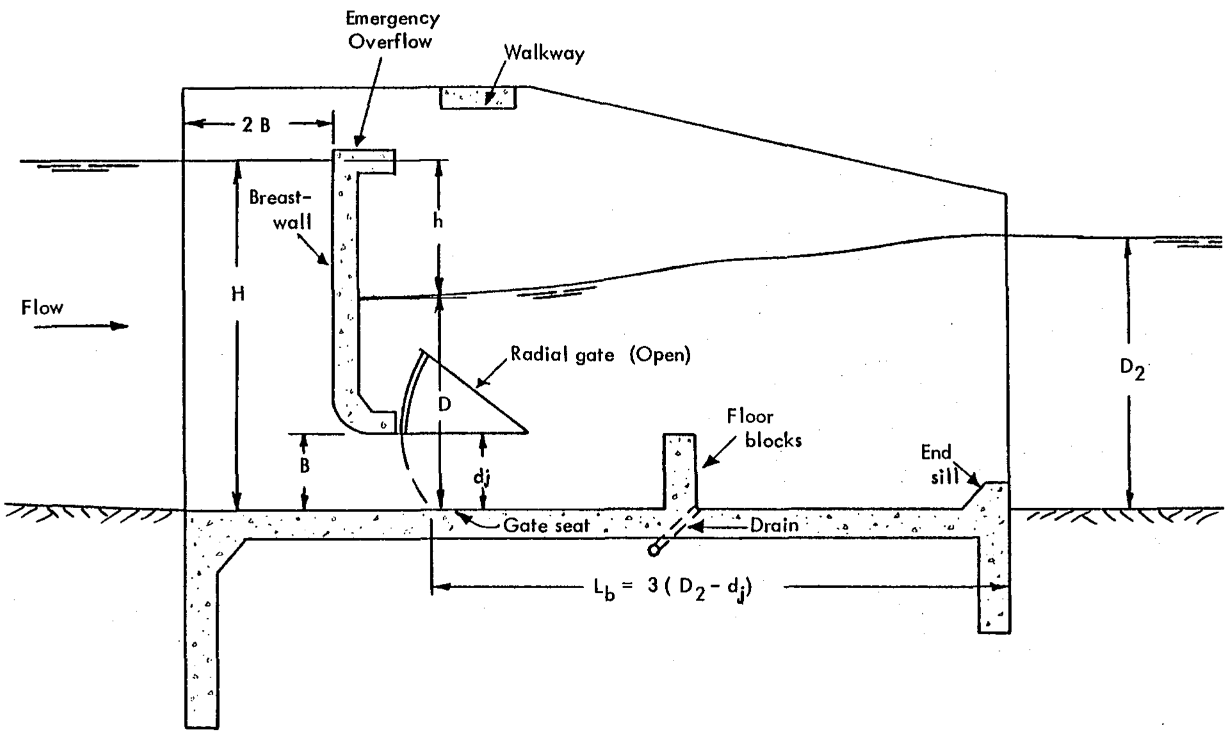

Figure 22. Section View of a Breastwall Check with Rounded Lip and Floor Baffles

The substantial reduction in gate size possible with the breastwall check is shown in the following example: Consider the case of $q = 2.8$ m^3/s/m and $D_2 = 2.7$ m. For a simple check, with $D_1 = D_2$, a 2.7 m high radial gate would be required. If a drop of 0.4 m is included in the design, so that $D_1 = 3.1$ m, then the same design would require a 3.1 m gate. However, if a square lip breastwall is used, a 1.5 m opening between the breastwall and the floor would be large enough to pass the design discharge. Further if the lip of the breastwall is properly rounded, as indicated in Figure 22, a 0.9 m height for B would be sufficient. A suitable curved lip will result if the radius is taken as half the height of the opening B, and if the curve is followed by a straight tangent of length 0.3B.

In designs using the breastwall it is advisable to terminate the wall below the top of the canal banks. In this way an automatic overflow is provided in the event of absence of the operator during a cloudburst, failure of an automatic gate, or debris in the gate opening. It may be desirable to extend the top of the breastwall horizontally to insure that the nappe clears the gate.

The worst case for basin design for the breastwall check will occur closer to the canal design discharge than for the simple check. As more drop is included the maximum momentum change occurs closer to full capacity flow. If the drop $H - D_2$ is more than 20% of D_2 at design flow, then the basin design should be based on full capacity discharge.

The design of the inlet and stilling basin for the breastwall check is similar to that for plain radial gate check. In addition the gate size must be calculated. The unit discharge is given by the orifice equation

[22] $$q = C_d B \sqrt{2gh}$$

in which C_d is the coefficient of discharge, B is the height of the gate opening, and h is the differential head across the opening. The problem is to find the required value of B. The standard handbook method is based on the assumption that the condition is similar to a submerged orifice where $h = H - D_2$ and $C_d = 0.6$, hence

[23] $$B = q/(0.6 \sqrt{2g\,(H - D_2)}$$

This approach greatly simplifies the problem, but gate sizes computed by [23] will be much larger than actually required. In the first place, the depth of water immediately downstream from the opening is not D_2 but D, and as seen for the plain check, D is substantially less than D_2 due to momentum considerations. Further, with a curved lip breastwall the coefficient of discharge can be increased to 0.97. Hence, the true gate size required for the square lip is given by

[24] $$B = q/(0.60 \sqrt{2g\,(H - D)})$$

and for the rounded lip

[25] $$B = q/(0.97 \sqrt{2g\,(H - D)})$$

In [24] the coefficient 0.60 accounts for both velocity and contraction effects, since $dj = 0.61B$. However, in [25] the coefficient is strictly a coefficient of velocity, since with the rounded lip contraction it is eliminated and $d_j = B$. Generally the design discharge q and the required checked up head H are known initially. The value of D must be calculated in order to solve for B. As before, this is found by satisfying momentum considerations as per [20], or [21] if floor baffles are used. The gate size will be much smaller than given by [23], so the extra time spent on the design is easily justified.

Example 3:

A rounded lip breastwall check without floor blocks has a gate opening height of 0.915 m. The downstream depth D_2 is 2.07 m when the unit discharge is 2.788 $m^3/s/m$. Prove that the required upstream head must be 2.329 m under these conditions.

Substituting the given values in [25].

$$0.915 = 2.788/(0.97 \sqrt{2g\,(2.329 - D)}\,)$$

it is found that the depth D on the downstream side of the gate is 1.826 m (i.e., h = 0.503 m). The jet velocity is q/B = 2.788/0.915 = 3.047 m/s. The velocity at the end of the structure is 2.788/2.07 = 1.347 m/s. These values should satisfy the momentum equation if the assumed H = 2.329 m is correct.

Check this, with hydrostatic forces and momentum flux in N/m

$$\gamma 1.8262/2 + 2.788\,\rho\,3.047 = \gamma 2.07/2 + 2.788\,\rho\,1.347$$

$$16{,}355 + 8{,}495 = 21{,}017 + 3{,}755$$

$$24{,}850 \approx 24{,}772$$

Q.E.D.

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PROBLEMS

1. What alternatives are available for crossing a topographical depression with a canal, and what factors have a bearing on the choice of alternative?

2. Why is it necessary to use inlet and outlet transitions on syphons and flumes?

3. Derive an expression for the minimum permissible depth of flow in a rectangular flume, in terms of the flume discharge Q, assuming a most efficient hydraulic section and flow at the limiting maximum Froude number.

4. If, in Example 1, it had been decided to use a flume velocity of 2.59 m/s instead of 2.28 m/s, what is the maximum value that should be used for the width b?

5. A flume designed for 11.5 m^3/s has a width of 3 m and a flow depth of 1.5 m. The average width of the downstream channel is 9 m. The outlet transition has straight sidewalls diverging at 1 in 10, and the floor is level throughout. Calculate the head recovery in the outlet.

6. Calculate the drop in water surface between two reaches of a canal connected across a ravine by a syphon with well designed inlet and outlet transitions, if $Q = 14\ m^3/s$, $D_o = 2.44$ m diameter, $L = 300$ m, Manning's $n = 0.012$, and the average canal width is 9 m.

7. Where are wasteways usually located on a canal system, and why?

8. Given $R_i = 0.9$ m, $B = 0.6$ m, and $C = 0.7$ for the syphon shown in Figure 17, calculate the lowest pressure in the system when the canal water level is 0.3 m above the crest and $h = 3$ m.

9. Explain, with reference to typical discharge equations, why it is easier to control the upstream water level with an overflow check than an underflow check.

10. An underflow type check structures has a breastwall with a square lip ($C_d = 0.6$) and a gate height $B = 0.75$ m. There are no floor blocks. The total head $H = 2.50$ m on the upstream face of the breastwall when the downstream canal depth is 1.80 m. Prove that the actual discharge capacity per unit width of structure is 2.20 $m^3/s/m$.

11. If floor blocks are added to the structure of problem 10 to improve stilling basin action, what other change in the structure would be required to obtain the same discharge with the same upstream and downstream depths? Why?

12. Calculate the total gate width required for a square lip underflow breastwall check without floor blocks, to discharge 28 m^3/s, if $H = 3$ m, $D_2 = 2$ m, and the gate height is 0.8 m. What length of stilling basin should be used?

13. If a rounded lip breastwall check with floor blocks is designed for $H = 3$ m, $D_2 = 2$ m, and $B = 0.6$ m, what basin length should be used?

CHAPTER X

CULVERT HYDRAULICS

A. INTRODUCTION

1. General

A culvert is an artificial water passage under a road, railroad, or canal. It is one of the most frequently used of all types of hydraulic structure and in terms of money invested annually in North America on culverts, it must be considered as one of the most important. The culvert has wide application as a drainage structure, and finds particular application to drainage problems in highway engineering. In some cases culverts have even been used to replace older timber bridges on a major stream.

Outwardly the culvert is a simple structure, yet the hydraulics of a culvert may be surprisingly complex. This is due to the large number and wide range of variables which have a bearing on culvert flow. These variables include headwater and tailwater depths, inlet design, and size, shape, length, slope, and roughness of the culvert. Shapes include round, square, rectangular, egg shaped and arch shaped. Construction materials include wood, concrete, plastic and corrugated steel. Common sizes range from 300 mm to 3600 mm. Standard pipe sizes for corrugated steel pipe are in 100 mm increments up to 1000 mm, and 200 mm increments thereafter. Flow resistance may vary from a Manning's n of 0.011 for a cast and vibrated concrete on a steel form, up to an extreme of 0.035 for a wood box culvert with inside ribs. Culvert inlet conditions include square ends or tapered ends, on thin or thick pipe, with or without wingwalls. The inlet may have a re-entrant, flush, beveled, or rounded lip. The culvert may flow full or partly full, the inlet may be free or submerged, and the outlet may be free or submerged.

A complete analysis of culvert hydraulics, including the effects of each of the foregoing variables, would fill a volume in itself. Nevertheless, if a large number of culverts are to be installed on a project, the culvert designer is well advised to study the problem in considerable detail. There are often large economies to be made by intelligent application of data currently available from a number of sources (see Bibliography).

The culvert market in North America is dominated by precast reinforced concrete pipe (RCP) and corrugated steel pipe (CSP). Concrete pipe is smooth, has good inlet characteristics, and is relatively free from problems of abrasion, corrosion and damage due to impact of road machinery. Special equipment may be required to handle concrete pipe due to the weight of the pipe sections.

Corrugated steel pipe has been used for drainage structures since the beginning of this century. One of the principal advantages of this type of pipe is the large strength to dead weight ratio, thus allowing use of lightweight pipe. This results in ease of handling for installation and low cost per unit length of pipe. This advantage is gained entirely from the fact that the pipe wall is corrugated. The corrugation has the appearance of a sine wave, and for standard CSP has an amplitude of 12.7 mm and a wave length of 67.7 mm. The corrugation increases the stiffness for 1.6 mm (16 gauge) plate by a factor of 100 while increasing the weight of pipe per unit length by only 8%. Corrugations may be annular or helical.

The principal disadvantage of annular CSP is the large hydraulic resistance, up to four times the rate of head loss compared to smooth walled pipe. A separation zone forms around the perimeter of the pipe behind each corrugation. The eddy in this zone absorbs energy from the flow and reduces the hydraulic efficiency of the pipe. In design this must be accounted for by using a larger pipe. Recently this disadvantage has been partially offset by the use of helical CSP. This pipe has lower frictional resistance than annular corrugated pipe. This is discussed in Section 9.

2. Design Discharge

Aside from the complexities of culvert hydraulics, the problem of design is further compounded by the fact that frequently the discharge for which the culvert should be designed is not known. Most culverts are installed on small drainage areas where a detailed hydrological analysis for each culvert cannot be justified, and even if it could, the data required for such an analysis would probably not be available. Many culverts are installed according to certain arbitrary codes laid down by the department or agency concerned with the project, and the result is that many culverts are underdesigned and many more are overdesigned. Neither result is desirable, but, unfortunately, the practice will continue as long as the engineer is faced with the problem of selecting culvert sizes without being allowed time and resources to collect and study the basic data.

Where records are available they should be used. In populated areas with a long history of development, records have usually been kept from which good correlations between precipitation and runoff can be made. Usually some government agency has processed the data so that peak flows corresponding to a particular return period can be quickly determined.

The design discharge for a culvert is frequently based on a return period of 5, 10, or perhaps 50 years, depending upon the importance of the installation. A 50 year return period might be used for a major multi-lane traffic arterial, whereas a 5 year return period may be used for a minor single lane farm road. There is no question of designing a culvert for a "one in a hundred" or "one in a thousand" flood, because the consequences of exceeding the design discharge are not usually serious enough to justify the extra cost of a larger culvert. These consequences do not involve loss of life, but rather minor property damage or temporary loss of service if the roadway is overtopped. In the latter case, some repair work on the slopes may be required after the flood has subsided. However, damage due to overtopping can be minimized by purposely incorporating certain design features in the fill. These features include grassed slopes, asphalt top, and a substantial length of flat profile. The use of a flat profile will insure that overtopping will occur over a broad front, and therefore with low discharge intensity and reduced potential for causing damage.

The allowable upstream water elevation at the design discharge will normally be lower than the crest of the roadway, and may be much lower for high fills where excessive velocities would be produced by a head corresponding to overtopping. The other important factor limiting the allowable headwater is the extent and consequence of temporary upstream flooding of adjacent properties.

In the absence of specific correlations between precipitation and runoff, the rational formula is recommended. The peak discharge is given by

$$Q = C\,i\,A/360 \tag{1}$$

in which Q is the discharge (m^3/s), i the rainfall intensity (mm/h), and A the drainage area (ha). The coefficient C is sometimes referred to as the runoff coefficient, and is dimensionless.

The rational formula applies exactly if the duration of the rainfall of specified intensity exceeds the concentration time for the drainage basin. This is often true for urban or small basins of a few square kilometers. Of course, the accuracy of the prediction still depends upon the accuracy of the estimate for C. Typical values of C are given in Table 1. If the duration of the rainfall is less than the concentration time, the rational formula will give too large a discharge unless C is reduced to account for the effect of overland flow and channel storage. This problem will arise on larger drainage areas and requires more sophisticated analysis.

TABLE 1

VALUES OF RUNOFF COEFFICIENT C

Built up area, 70% impervious	0.80
Steep residential area, 50% impervious	0.65
Flat residential area, 30% impervious	0.40
Rolling vultinvated land, clay loam	0.50
Forested hilly land, clay loam	0.50
Flat cultivated land, sandy	0.20

3. Culvert Hydraulics

Culvert discharge capacity is a function of many variables, as shown by

[2] $$Q = f(H, d_t, D, L, S, n, G)$$

In this equation H is the total head (headwater depth) above the invert, at the inlet, d_t is the tailwater depth above the invert at the outlet, D is the diameter of the pipe, L is the culvert length, S is the culvert slope, n is Manning's resistance factor and G is the geometry of the inlet.

There are two major classifications of hydraulic behavior, referred to as inlet control and outlet control. For inlet control the discharge is governed by the size of the pipe, the geometry of the inlet and the headpool level. It is independent of the pipe length, slope, roughness and tailpool level. In effect the flow enters the pipe with the inlet behaving as an orifice and passes through the pipe flowing partly full. Part full flow will occur regardless of how deeply the inlet is submerged, provided the pipe is sufficiently short or steep or smooth, or some combination thereof, so that the discharge can pass through the pipe without filling the barrel. In addition the tailpool level should be below the crown of the pipe so the air demand for part full flow can be supplied from atmosphere at the downstream end of the pipe.

For outlet control the pipe flows full for all or part of its length, and in this case the pipe length, slope, roughness and tailpool level also have an effect on the discharge. If conditions favorable to both inlet and outlet control are present simultaneously, the governing discharge is the lesser of these two. The rationale here is that the pipe cannot pass more discharge than the inlet will allow, nor can the inlet pass more discharge than the remainder of the pipe can handle. However, exceptions are possible. If both the inlet and outlet of a sloping pipe are submerged this will normally result in full pipe flow because the air supply from atmosphere, necessary to maintain part full flow, is cut off. Outlet control will govern even if the discharge for inlet control is smaller. This is possible because the effective head for inlet control is increased by the suction effect at the inlet produced by the pipe slope. However, if the negative pressure at the inlet drops too low, air may be drawn into the inlet or admitted via the vortex, causing the flow to revert to inlet control. The discharge may become unsteady, alternating between inlet and outlet control. This condition is referred to as slug flow. Outlet control may also occur even if the outlet is unsubmerged, provided the inlet design is such that it tends to promote full flow at the inlet (such as a bellmouth inlet). If the pipe slope is steep, the discharge may be increased by inlet suction, but again there is a possibility that slug flow will develop.

As a general philosophy for culvert design, the culvert should be sized so that the inlet will be submerged at the design discharge. In this way, the designer will be able to get better value for the investment (i.e., more discharge for the same cost). A minimum submergence of 50% of the diameter above the crown is recommended. Greater submergence may be possible for smaller pipes. On the other hand, inlet submergence may be precluded by other considerations, such as excessive flooded area upstream, inadequate grade height to prevent overtopping of the road or limitations on maximum velocity (for fish passage or outlet scour).

B. INLET CONTROL

4. Inlet Types

If no end treatment is used at the pipe inlet, the inlet is usually referred to as a plain inlet. The plain inlet is basically the natural end of the pipe section as delivered to the site, which may be either RCP or CSP. For tongue and groove RCP, the groove end is always placed upstream, as shown in Figure 1. This placement procedure gives better inlet conditions by virtue of the enlarged intake area at the inlet. For CSP, the plain inlet basically simulates a thin-walled re-entrant tube. This type of inlet will be referred to as the CSP projecting inlet. An example is shown in Figure 2.

The hydraulic performance of the RCP groove end inlet is sufficiently good that additional end treatment to further improve performance cannot be justified and is not required. However, there is considerable room for improvement in the CSP projecting inlet. It is known from basic hydraulics that the thin-walled re-entrant tube has the lowest coefficient of contraction when discharging as an orifice ($C_c = 0.5$) and the highest head loss coefficient ($K_e = 1.0$) when flowing full. Accordingly, it is often advantageous to use some end treatment at the inlet which will improve performance. Figure 3 shows a schematic of a number of end treatments often used on CSP inlets. Figure 3(a) is the plain projecting inlet. Figure 3(b) illustrates the flush headwall, representing the case where a concrete headwall is placed around the culvert perimeter. This procedure will reinforce the inlet, eliminate the possibility of an uplift failure and improve the hydraulics. End reinforcement will prevent the occurrence of dented ends or inlet collapse. Normally, the headwall would have to be supported on a footing with stub wingwalls to resist backfill loading. However, concrete headwalls are not generally used for CSP inlets.

Figure 1. RCP Tongue and Groove Inlet

Figure 2. CSP Projecting Inlet

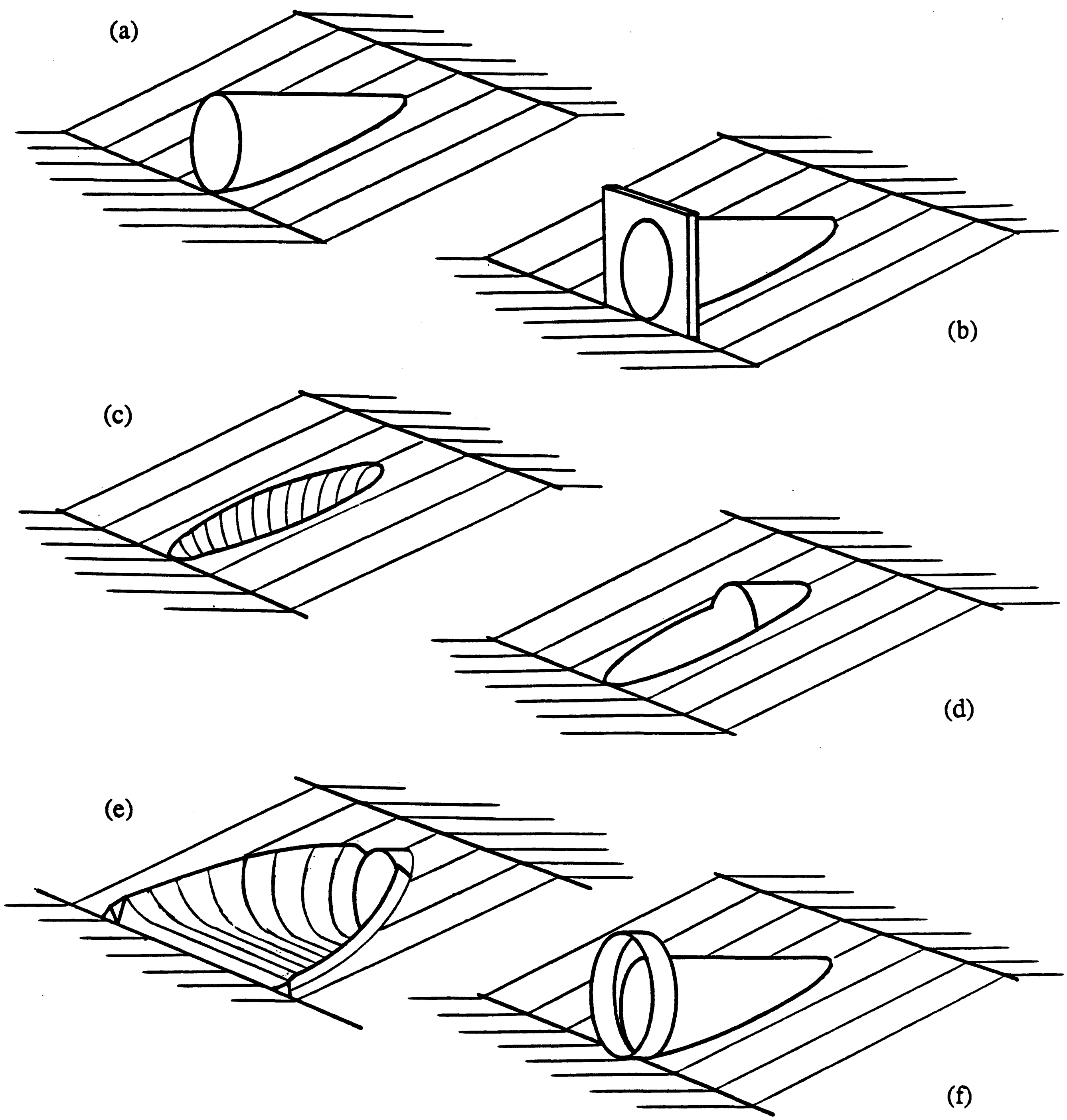

Figure 3. Inlet Types for CSP
(a) Projecting (b) Flush headwall (c) Tapered
(d) Stepped Taper (e) Armtec (f) Cylinder

Many CSP culverts are laid with the opening flush with the shoulder of the road, as in Figure 3(c). In effect, the projecting part of the culvert is eliminated. Initially, it was believed that this would increase the discharge capacity because of the larger entrance area. In fact, the tapered inlet conditions are even worse than the plain projecting inlet. In addition, the tapered inlet is structurally weaker than the projecting inlet due to loss of ring strength. In some cases, the loss of strength has resulted in inward collapse of the sides of the inlet. The stepped taper, shown in Figure 3(d) represents a compromise between the projecting and tapered inlet. The tapered portion of the pipe ends abruptly at a position 1.2 to 1.5 diameters downstream from the inlet edge at the invert. The stepped taper is somewhat stronger than the full taper, but the hydraulics is basically unchanged.

Figure 3(e) shows the Armetc inlet. This inlet is a commercially available culvert end section which can be used at either end of the culvert. For pipe sizes up to 1000 mm in diameter, the Armtec end section has a length of 1.75 D and a width at the mouth, or inlet, of 2.14 D. Larger sizes are somewhat shorter and narrower. This inlet is advertised as improving the hydraulics at the inlet and reducing scour at the outlet. For part full flow with the inlet unsubmerged, the streamlining effect of the inlet gives some increase in hydraulic efficiency, but much of this benefit is lost when the pipe becomes submerged because the end section does not extend around the crown of the pipe. Use of the end section at the outlet does not produce a reduction in channel scour.

Figure 3(f) shows the cylinder inlet, developed by Smith for Saskatchewan Highways and Transportation. The cylinder inlet consists of a short length of larger diameter pipe which projects upstream from the normal projecting inlet. A common invert is used for the culvert and the cylinder. The cylinder diameter is approximately 1.25 D in diameter, and extends upstream 0.2 D in front of the normal inlet. A crescent shaped steel plate can be welded in place to seal the space between the cylinder and the pipe, or the cylinder can be telescoped a short distance over the pipe and the annulus sealed with concrete, asphalt or other mastic sealant. Unlike the Armtec inlet, the cylinder gives minimal benefit for shallow flows, but produces a marked increase in discharge when it becomes submerged.

Figures 4 and 5 show examples of the Armtec inlet and cylinder inlet.

5. Basic Equation

When the upstream water level submerges the pipe at the inlet orifice type flow will develop. The contraction of flow at the inlet will become fully developed when $H/D \geq 1.5$, in which H is the head above the invert at the culvert inlet and D is the pipe diameter. In this case, the classical orifice equation applies

[3] $$Q = CA\sqrt{2gh}$$

in which Q is the discharge (m^3/s), C is the coefficient of discharge, A is the pipe area (m^2), h is the effective head on the opening at the inlet (m) and g is the acceleration due to gravity ($9.81\ m/s^2$). For a true orifice which discharges into atmosphere, the effective head is the vertical height from the upstream energy level (usually the water surface) to the center of the opening, from which

[4] $$h = H - D/2$$

Figure 4. Armtec Inlet

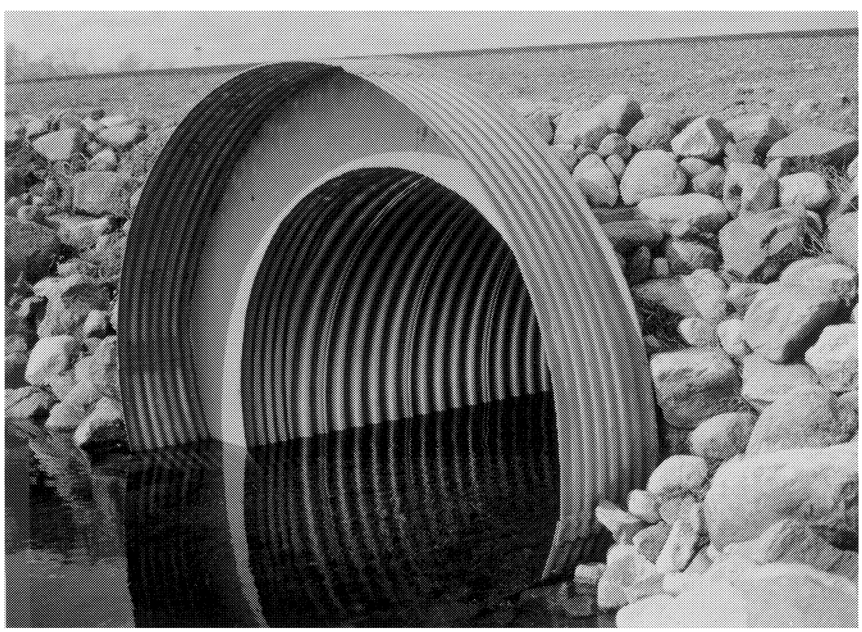

Figure 5. Cylinder Inlet

However, a culvert inlet is not a true orifice. In a true orifice, there is no upstream channel bed flush with the invert and there is no pipe downstream. The presence of the channel bed is to eliminate flow contraction at the invert, slightly increasing the coefficient of discharge in [3], and the inflow is not allowed to fall in a free trajectory in atmosphere because it is supported by the pipe. This support produces some internal pressure in the jet flow immediately downstream from the pipe inlet, so the effective head becomes less than given by [4]. As a result, the orifice equation for inlet control for a culvert becomes

[5] $$Q = CA\sqrt{2g\,(H - \alpha D)}$$

in which α is a coefficient, ranging between 0.5 and 1.0, depending on the inlet design. Since $A = \pi D^2/4$, [5] may be written as

[6] $$Q/D^{5/2} = 0.785\sqrt{2g}\;C\sqrt{H/D - \alpha}$$

which shows that $Q/D^{5/2}$ is a function of H/D alone when C and α are constant.

Design values for C and α are given in Table 2. It is significant that the inlets which have higher C values also have higher α values. The reason for this is that a high value for C means less jet contraction and therefore greater depth of flow inside the inlet. Greater flow depths, in turn, are accompanied by increased pressure on the pipe invert and a corresponding decrease in effective orifice head (i.e., an increase in α). In addition, the downward direction of the flow produced by the tapered inlets tend to further increase the pressure on the invert at the inlet.

TABLE 2

VALUES FOR C AND α FOR INLET CONTROL WITH $H/D \geq 1.5$

Inlet Type	C	α
Projecting (CSP)	0.546	0.6
Tapered and Stepped Taper (CSP)	0.560	0.7
Flush Headwall	0.637	0.7
Armetc (CSP)	0.640	0.7
Cylinder (CSP)	0.756	0.8
Tongue and Groove (RCP)	0.815	0.8
Bellmouth	0.970	0.9

6. Design Curves for Inlet Control

When the culvert discharges under inlet control but the head above the invert at the inlet is less than the pipe diameter, a free surface will occur. The inlet is basically a weir with a circular cross-section in the vertical plane. Weir type flow is governed by the equation

[7] $$Q = KH^n$$

with n being an exponent greater than unity. However, unlike the case for a simple rectangular weir, the coefficient K and exponent n are not constant, but are functions of the H/D ratio. This occurs because the shape of the water prism and effect of the inlet vary with the depth of flow in the inlet. The values of K and n cannot be determined analytically and [7] is not used in practice. Instead, the relationship between H/D and $Q/D^{5/2}$ has been determined experimentally from model tests on culvert inlets. The test data extend up to H/D = 1.5 to include the transition region between weir type flow (H/D < 1) and orifice type flow (H/D > 1.5).

Design curves for inlet control for different inlets up to H/D = 2.4 are shown on Figure 6. For H/D ≥ 1.5 the curves agree with [6] with C and α values from Table 2. If preferred the equation can be used instead of Figure 6 when H/D ≥ 1.5, and of course must be used for larger H/D values which are beyond the range covered by the figure.

Although a bellmouth inlet would rarely, if ever, be used on a culvert inlet, the curve has been shown on Figure 6 as an indication of the maximum attainable discharge capacity. Up to H/D = 1 the bellmouth inlet curve agrees with the theory of critical flow in a circular section, as though the inlet behaves as broad crested weir, circular in cross-section, and with a streamlined entrance. Since the bellmouth eliminates jet contraction, it tends to produce full flow once H/D exceeds about 1.3. Thus, the inlet control curve for larger H/D values is somewhat academic. It could be achieved in practice only by using a steep culvert slope and a large capacity air vent at the crown of the pipe downstream from the inlet. In the absence of an air vent, outlet control would govern, unless the slope is too steep, in which case slug flow could develop as air is intermittently drawn into the inlet via the vortex funnel. For a culvert which is horizontal or has a mild slope, outlet control would govern because the discharge will be smaller than for inlet control.

Figure 6 shows that up to H/D = 0.8, the Armetc inlet is almost as good as the RCP tongue and groove inlet or the bellmouth inlet. Obviously, the streamlining effect of the Armtec inlet comes into play for the shallower flows. Once the inlet is submerged, it becomes less efficient and the curve shifts upwards, between H/D values of 1.0 and 1.2, and becomes equivalent to the flush headwall. This is due to the fact that the sides of the inlet curve into the sides of the pipe, leaving the crown of the pipe fully exposed and projecting, and as soon as the pipe becomes submerged, a strong vertical contraction develops under the crown of the pipe inside the inlet.

Figure 6 also shows that the benefit of the cylinder inlet increases as H/D increases. For example, at H/D = 0.6 the $Q/D^{5/2}$ value for the cylinder inlet is only 4% greater than for the projecting inlet (0.49 vs. 0.47), but at H/D = 2.0 the $Q/D^{5/2}$ value is increased by 28% (2.89 vs. 2.25). Thus, the effect of the cylinder inlet in increasing the discharge does not really come into play until the cylinder is submerged.

Example 1:

A design discharge of 2.3 m^3/s must be passed through a culvert without exceeding a head above the invert of 1.5 m. It has been determined that inlet control will govern. What size of pipe must be used if the culvert is: (a) RCP; (b) CSP with a projecting inlet.

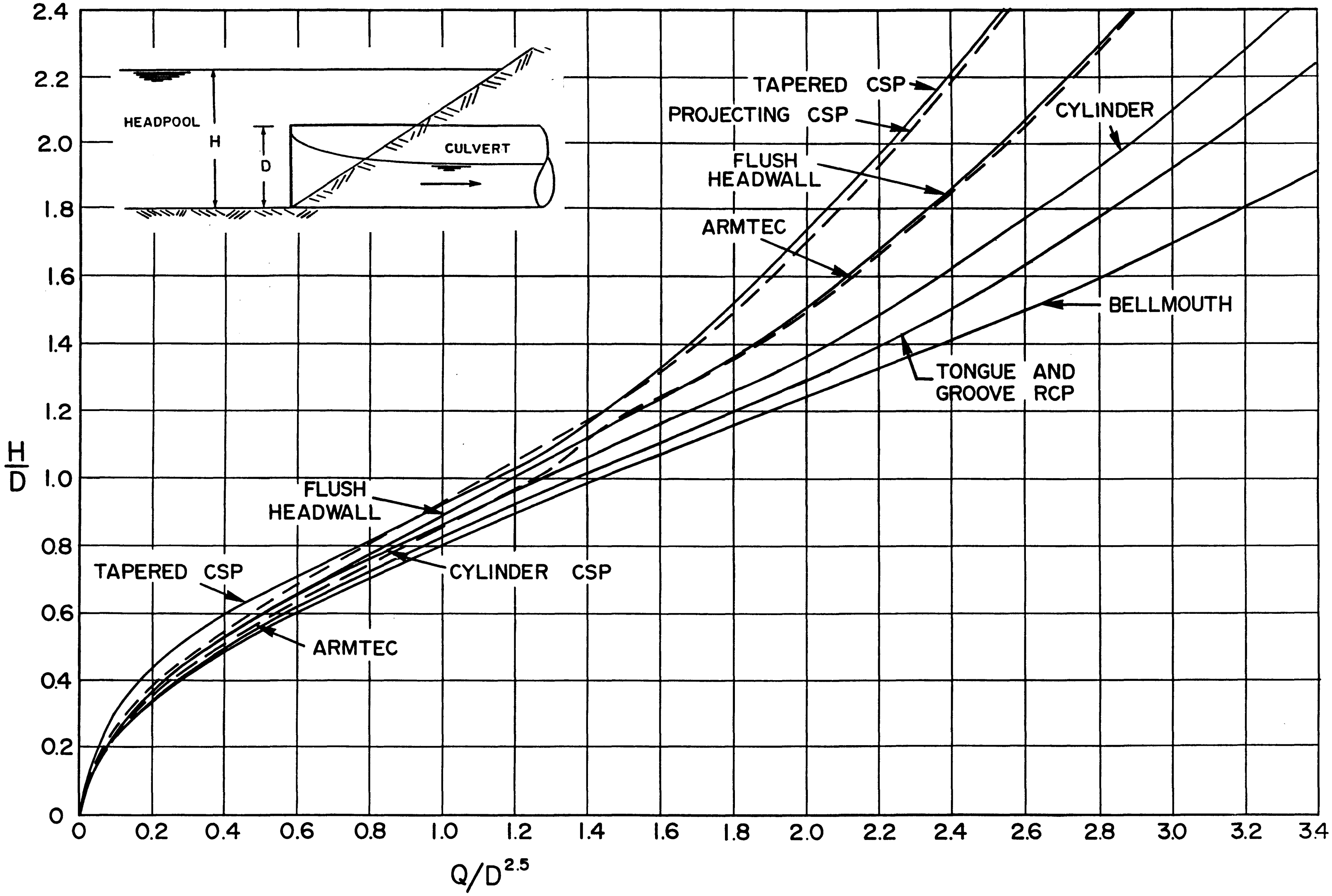

Figure 6. Inlet Control for Various Inlets

Figure 6 will apply, but trial and error is required. Assuming 1000 mm pipe, then for $Q/D^{5/2} = 2.3$ the corresponding H/D is 2.10 for CSP and 1.45 for RCP. Hence 1000 mm is too small for CSP but is satisfactory for RCP, with a head of 1.45 m.

Assuming a 1200 mm pipe for CSP, for which $Q/D^{5/2} = 2.3/1.2^{5/2} = 1.46$, then H/D = 1.21 and H = 1.21 x 1.2 = 1.45 m. The 1200 mm pipe is adequate for CSP.

Example 2:

The design discharge for an 800 mm CSP is 1.45 m^3/s. By how much will the headwater be reduced by attaching a 1000 mm cylinder inlet? Assume inlet control.

Since $Q/D^{5/2} = 1.45/0.8^{5/2} = 2.53$, then from Figure 6, H/D = 2.35 and hence H = 1.88 m. If a cylinder inlet is used, then H/D = 1.72, from which H = 1.38 m. Therefore the head will be reduced from 1.88 m to 1.38 m by attaching a cylinder inlet.

C. OUTLET CONTROL

7. Basic Equation

For outlet control the discharge capacity is determined by the headwater depth, inlet geometry, the slope, roughness, and length of the pipe, and the outlet condition. If the pipe flows partly full throughout its length, the control is either critical depth at the outlet or the depth of the downstream tailwater above the invert, whichever is greater. Although it is a simple matter to determine which of these conditions acts as control, this is by far the most difficult and laborious type of culvert flow to calculate because a water surface profile must be evaluated for the entire pipe length. Starting with the known condition at the outlet, the step method is used to evaluate the profile and determine the flow depth just inside the inlet. The velocity head and inlet loss are added to this depth to determine the pool elevation. This procedure is covered in more detail in Part D of this Chapter.

If the pipe flows full throughout, the effective head h across the system is the difference in elevation between the headpool and tailpool, or between the headpool and the effective position of the hydraulic grade line, whichever value is smaller. This case is easier to evaluate than part full flow because the flow area is constant. Full flow may be anticipated if the inlet is efficient, or if the pipe has a flat slope and is long or rough, or if the outlet is submerged.

The governing equation for outlet control with full pipe flow is

$$h = h_e + h_f + h_o \tag{8}$$

in which h is the differential head acting on the pipe system, and h_e, h_f and h_o are the losses at the inlet, in the barrel and at the outlet, respectively. Since each loss is a function of the velocity head, [8] may be written as

$$h = (K_e + K_f + K_o)\, V^2/2g \tag{9}$$

in which $K_e + K_f + K_o$ are the loss coefficients for the inlet, barrel and outlet, and V is the mean flow velocity in the pipe. From the geometry of the system the value of h becomes

[10] $\quad h = H + SL - y$

in which H is the head above the invert at the inlet, S is the pipe slope, L is the pipe length and y is the datum level at the outlet. The product SL represents the drop in the elevation of the pipe due to the slope. In [9] K_e depends on the inlet geometry, K_o is taken as 1.0, and K_f is given by

[11] $\quad K_f = 2gn^2L/R^{4/3}$

in which n is Manning's resistance factor and R is the hydraulic radius (R = D/4 for full flow). The mean flow velocity in the pipe may be solved from [9], [10] and [11], giving

[12] $\quad V = \sqrt{2g\ (H + SL - y)/(K_e + 2gn^2L/R^{4/3} + 1)}$

8. Inlet Loss Coefficients

The primary cause of inlet losses is the loss associated with diffusion of the contracted jet produced by the inlet as it expands to fill the pipe downstream. An eddy forms in the space surrounding the contracted jet inside the pipe. This eddy absorbs energy from the flow. The greater the jet contraction, the larger the eddy. Also, greater jet contraction produces a higher jet velocity for a given discharge, leading to more violent agitation of the eddy. It is apparent that jet contraction has an important effect on pipe discharge for outlet control as well as inlet control. Thus, those inlets with lower coefficients of discharge for inlet control would be expected to have higher coefficients of entrance head loss for outlet control. The actual coefficient must also include any losses caused by excess boundary friction at the inlet.

When H/D ≥ 1.5, for which the contraction of the inlet is fully developed (i.e., the inlet flow pattern becomes independent of H/D), then it is to be expected that the inlet loss coefficient would become constant. Loss coefficients have been determined from experiments on models and prototypes. Recommended values for design as shown in Table 3. Measured values show some scatter, and in general the average of the measured values for each inlet tends to be about 0.05 units below the tabled value. Tabled values have been rounded up to one-decimal place and are marginally conservative. Refinement to the second or third decimal would suggest an accuracy that does not in fact exist, and is not required for analysis.

The flow pattern in the inlet varies continuously as the H/D value increases from 0 to 1.5. Thus the inlet loss coefficient may also be expected to vary. The effect is shown in Figure 7. With H/D < 1, the culvert will flow partly full throughout its length, even with outlet control. Analysis for this case is dealt with separately in Part D of this chapter. However, for 1.0 < H/D < 1.5 the culvert may flow full. In most cases, the inlet loss coefficient tends to be somewhat smaller than the value at H/D = 1.5, and much smaller for the Armec inlet. For example, for the Armtec inlet at H/D = 0.5, K_e is 0.2, compared to 0.6 when H/D = 1.5. The cylinder inlet, on the other hand, shows an increase in K_e, from 0.3 to 0.5, as the H/D value drops from 1.5 to 0.5. These variations are consistent with observations made for inlet control, in that what is efficient for an inlet under inlet control is also efficient under outlet control, and vice versa.

TABLE 3

RECOMMENDED INLET LOSS COEFFICIENTS
FOR CIRCULAR CULVERTS WITH H/D ≥ 1.5

INLET TYPE	K_e
Projecting (CSP)	1.0
Tapered (CSP)	1.1
Armtec (CSP)	0.6
Flush Headwall	0.5
Cylinder (CSP)	0.3
Tongue & Groove (RCP)	0.2
Bellmouth	0.1

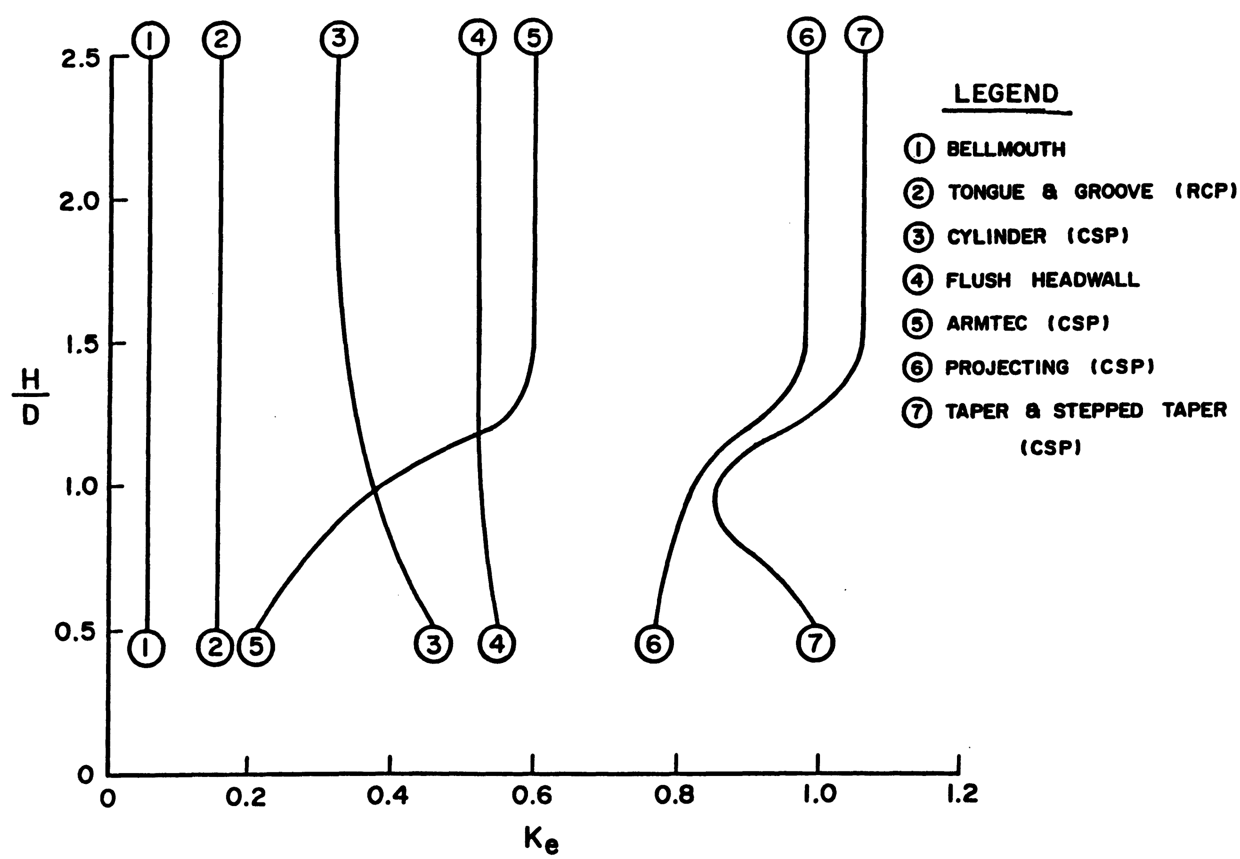

Figure 7. Inlet Loss Coefficient for Various Inlets

9. Friction Loss

a) Concrete or Wood Stave Pipe

Most concrete pipe used for culverts is precast, using smooth steel forms. Values of Manning's n for new pipe, as determined from full scale tests on RCP at St. Anthony Falls Hydraulic Laboratory, fall between 0.010 and 0.012. For field application, a value of 0.012 is recommended for use in [11] to determine the barrel friction loss coefficient.

Wood stave pipe using creosoted fir usually has some minor longitudinal steps or ridges due to differences in the thickness of the staves. These do not interfere with the flow, and in fact appear to have a stabilizing influence. Also there is an absence of transverse joints such as occurs with jointed concrete pipe. Therefore, in spite of a boundary texture that is less smooth than concrete, an n value of 0.012 is again appropriate.

b) Annular Corrugated Pipe

In general, prototype pipe resistance cannot be determined from model studies because resistance is often dependent on the Reynolds number, and the Reynolds number is usually one or two orders of magnitude smaller on the model. The result is that the most useful data on pipe friction is that which is observed for prototype size pipes. In the case of corrugated pipe, the first comprehensive tests on full size pipe were made at the University of Iowa in the early 1920's. Pipe sizes of 12, 18, 24 and 30 inches were tested. At that time the Kutter equation for open channel flow was still popular and the resistance coefficients were calculated from the data in terms of Kutter's n. The average n value was about 0.021.

King recommended that substitution of the much simpler Manning formula, dating from 1890, would be a step forward in the solution of pipe flow problems. The practice of using Kutter's n in Manning's equation developed in the 1920's in the corrugated pipe industry and persisted until the 1950's. For example, this procedure was recommended in the 1947 Armco Handbook. Although the values of Kutter's n and Manning's n are close, they are not exactly the same. The best agreement is for smooth pipe with low n values. This was clearly shown in the studies by Straub and Morris in 1950. They conducted comprehensive tests under exacting conditions on full scale pipes, and found that Manning's n varied with pipe size and Reynolds number, ranging from 0.022 to 0.026 for annular corrugated pipe. They recalculated the original Iowa test data to solve Manning's n and found the average value was 0.024 in contrast to the Kutter n value of 0.021. Lest this difference seem insignificant, it should be pointed out that friction losses vary as the square of the n value. Thus, for 3 decades, the frictional resistance of corrugated pipe was underestimated by a factor of 23%!

The Straub and Morris data is shown plotted in Figure 8, and indicates that n is not a constant, as commonly assumed. The assumption of a constant n is made in practice primarily as a matter of convenience, and in recognition of the fact that the variation of n is not large. The Straub and Morris data show that the resistance increases with Reynolds number, and since the design condition would correspond to the larger Reynolds number for a given pipe, a Manning's n of 0.025 was recommended for design purposes.

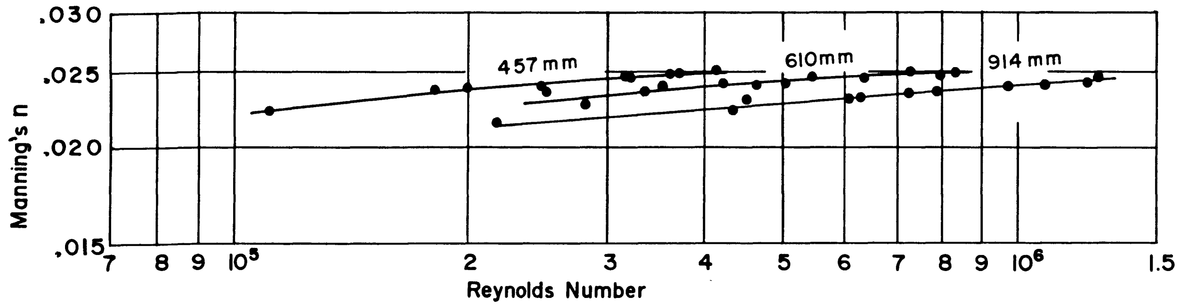

Figure 8. Mannings n vs. Reynolds Number for Annular Corrugated Pipe.

The trend of rising pipe resistance with increasing Reynolds number shown in Figure 8 was not expected. For commercial concrete and steel pipes with random non-uniform roughness the transition from hydraulically smooth to hydraulically rough follows a falling trend, in accordance with the Colebrook-White transition function. However, the classic work by Nikaradse, using artificially sand roughened pipes with a uniform roughness, indicated the rising trend in the transition region between smooth and rough. Evidently the same type of transition applies to uniform strip type roughness characteristic of corrugated pipe.

The resistance characteristics of corrugated strip type roughness were subsequently studied in more detail and a friction factor versus Reynold's number diagram was prepared by Morris in 1959. The diagram is shown in Figure 9. It has all the same elements as the Moody diagram which is used to determine the friction factor for commercial pipes. The differences are that the Morris diagram shows the rising trend in the transition between smooth and rough, and the significant resistance parameter is the relative roughness

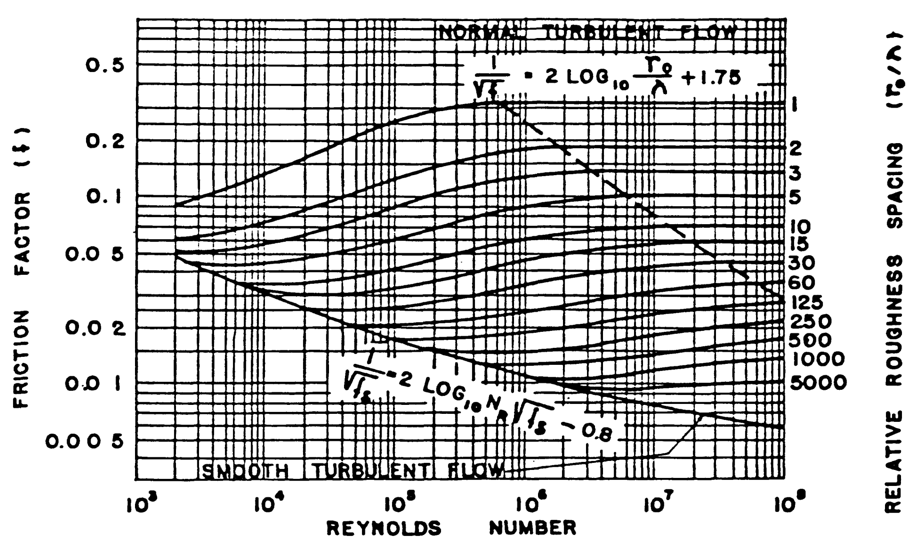

Figure 9. Resistance Diagram for Annular Corrugated Pipe.

spacing rather than the relative roughness height. The relative roughness spacing is r_o/λ, in which r_o is the pipe radius and l is the wave length of the corrugations. The Morris diagram applies provided $2 < \lambda /e < 6$, in which e is the amplitude of the corrugation (e.g. height of the roughness strip). This condition is satisfied for most corrugated pipe.

Figure 9 can be used for all annular corrugated pipe. The friction factor f so determined is used in Darcy's equation for head loss in pipes

[13] $$h_f = fLV^2/(2gD)$$

in which h_f is the head loss, L is the pipe length, V is the mean velocity, g is acceleration due to gravity and D is the pipe diameter. The equation can be used with the same f value in any consistent set of units.

The relationship between Darcy's f and Manning's n is given by

[14] $$n = (fR^{1/3}/8g)^{1/2}$$

with R taken as the hydraulic radius. If this conversion is applied to the f values from Figure 9, it will be found that the n values so determined agree with those on Figure 8 at the corresponding pipe size and Reynolds number. In practice a value of $n = 0.024$ is usually assumed for standard corrugated steel pipe. Figure 9 and [14] may be used to determine n values for other corrugated or ribbed pipe as well, such as multi-plate or liner plate (which have higher n values).

c) Helical Corrugated Pipe

Until recently the majority of corrugated steel pipe for drainage structures was of riveted lap joint construction with annular corrugations. Helical corrugated steel pipe with lock seam joints was first put on the market in 1934, but was confined to small pipe size for subdrainage applications. Since 1975 however, helical lock seam corrugated pipe has been manufactured in larger sizes for drainage culverts, and now accounts for most of the new installations. Adaptation of the helical construction to large pipe has eliminated the need for riveted connections of short jointed sections and has further reduced the cost of the pipe. An unexpected benefit of the use of helical pipe is the rather spectacular increase in hydraulic efficiency.

A significant variable affecting hydraulic resistance in helical pipe is the helix angle θ, representing the angle between the axis of the pipe and the direction of the spiral or helix. This angle is shown on Figure 10, and is given by the equation

[15] $$\theta = \cos^{-1} (w/\pi D)$$

in which w is the width of the corrugated strip used for the spiral winding and D is the pipe diameter.

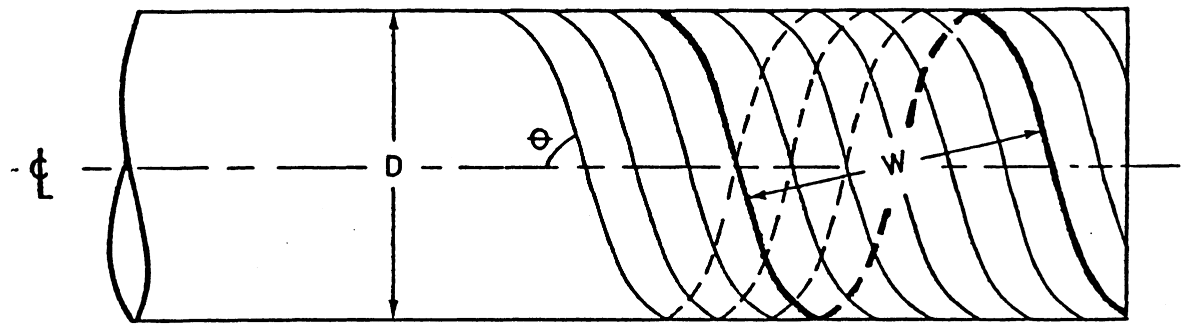

Figure 10. Helix Angle for Helical Pipe

The helical corrugations cause the flow to rotate and due to centrifugal force a radial pressure gradient becomes superimposed on the hydraulic pressure. By a complex process that is not fully understood the axial velocity near the wall of the pipe is decreased, reducing wall shear and head losses.

Measurements at St. Anthony Falls Hydraulic Laboratory by Silberman have shown that in a corrugated pipe with a helix angle of 62°, for example, the velocity vector 0.05 diameters from the pipe wall has a spiral direction which is at an angle of 26 degrees relative to the pipe axis. This means that the flow at the wall is crossing the corrugations at an angle of 36° (62 minus 26), rather than at 90° as for annular corrugations. This has the effect of increasing the flow distance between the crests of the corrugations from 67.7 mm to 115.2 mm, greatly decreasing the effective "waviness" of the pipe wall and providing a better opportunity for the flow to follow the surface without separation. The measured hydraulic resistance in this case was only one-quarter of the value for annular corrugated pipe. For larger diameter pipe with steeper helix angles, the gain in performance is correspondingly reduced.

Ideally, a diagram similar to Figure 9 should be prepared for helical pipe. This would show the effect of pipe size and Reynolds number on the pipe resistance. As yet there has not been enough research to construct the counterpart of this figure, although advance indications are that the relative roughness spacing r_o/λ has less influence on a helical pipe than an annular corrugated pipe. This suggests that helix angle is a more important variable than pipe diameter.

Silberman tested full scale helical pipes in the 305 mm to 1220 mm size, and found that for full flow at large Reynolds numbers and helix angles between 60° and 90°, the best fit line is given by

[16] $$n = 0.683 \times 10^{-5}\, \theta^{1.82}/D^{0.038}$$

with θ in degrees and D in m. The exponent of D is so small that $D^{0.038}$ is always near unity regardless of the pipe size. This of course is one of the principal features of the use of Manning's equation - the relatively constant value of n independent of pipe size.

If the plate width used to fabricate helical pipe is constant, there will be a unique relationship between helix angle and pipe diameter. The common plate width is 610 mm for which [15] would become

[17] $$\cos\theta = 0.194/D$$

Simultaneous solution of [16] and [17] allows expression of n in terms of θ in the form

[18] $$n = 0.683 \times 10^{-5}\, \theta^{1.82}/(0.194/\cos\theta)^{0.038}$$

Equation 18 has been used to prepare Table 4, which shows recommended n values for helical corrugated pipe. These values agree with those recommended by the Corrugated Steel Pipe Institute.

TABLE 4

RECOMMENDED MANNING'S N VALUES FOR HELICAL CORRUGATED PIPE FABRICATED FROM 610 MM PLATE WIDTH

Pipe Diameter D mm	Helix Angle θ degrees	Manning's n
400	60.96	0.0125
500	67.15	0.0148
600	71.12	0.0163
700	73.89	0.0174
800	75.95	0.0182
900	77.54	0.0188
1000	78.80	0.0193
1200	80.69	0.0200
1400	82.03	0.0205
1600	83.04	0.0208
1800	83.81	0.0211
2000	84.43	0.0213
2200	84.94	0.0215
2400	85.36	0.0216

d) Flow Establishment in Helical Pipe

The recommended n values in Table 4 apply to fully established spiral flow in a helical pipe. In the experiments previously referred to the researchers were careful to use only observations on a test length of pipe well away from any influence of end effects. For example, in Silberman's tests the beginning of the test reach was 20 pipe diameters downstream from the pipe inlet. In a real situation, however, the designer must be in a position to assess all the losses in the system, including the losses in the initial part of the pipe where the flow pattern is not developed.

When flow first enters a drainage culvert from a ditch, or enters a sewer from a manhole, the direction of flow is axial. The spiral rotation will subsequently develop due to interaction with the helical corrugations, but in the development length the rate of head loss will be greater than the rate of head loss for fully established flow. This means that there will be an extra loss term which must be considered, and which may be referred to as the development head loss. Hence, the head loss equation for a drainage culvert or for a storm sewer between 2 manholes would take the form

[19] $$h = h_e + h_d + h_f + h_o$$

in which h is the differential head across the system, h_e is the entrance loss, h_d is the development head loss, h_f is the friction loss and h_o is the outlet loss. Each of these losses can be expressed as a coefficient of the velocity head such that

[20] $$h = (K_e + K_d + K_f + K_o)\, V^2/2g$$

in which K_d is the development head loss coefficient.

Experimental values of the development head loss coefficient have been determined by Smith for helix angles of 90, 71.4 and 61 degrees. These are shown plotted on Figure 11. It is noted that the range of variation of K_d over the practical range of pipe size from 400 mm to 2400 mm is small. It is recommended that a constant value be used for all helical pipe, and that the value be taken as 0.2.

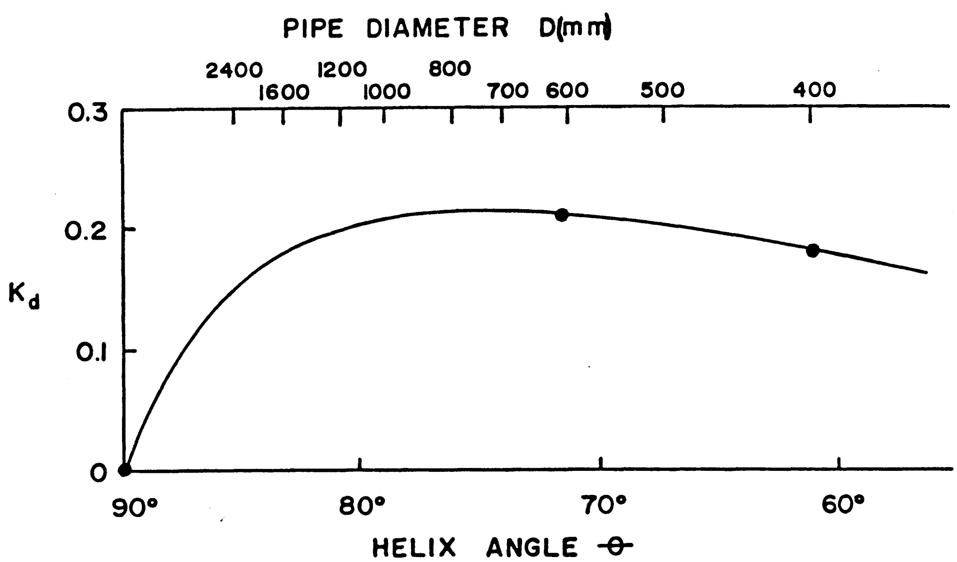

Figure 11. Development Head Loss Coefficient

e) Effect of Annular Ends on Helical Pipe

Corrugated steel pipe sections are commonly rolled in 4 m, 6 m or 8 m lengths. Theoretically, a 24 m culvert could be placed using 6-4 m lengths, 4-6 m lengths, or 3-8 m lengths. The longer sections are generally preferred in order to reduce the number of connections. Annular band couplers are used to connect annular pipe sections. These exterior bands overlap the ends of the pipe and give a strong tight connection. Where complete water tightness is required, they may be fitted with rubber ring sealers.

Annular band couplers do not fit helical corrugations. Dimple connectors have been used but these do not make a strong connection and are difficult to make water-tight. As a result, the practice has arisen of rolling annular ends on each section of helical pipe. This rolling is done after the helical pipe section has been rolled and cut to length. Each annular end is 0.305 m in length. Thus, 15.3% of the 4 m sections is annular, 10.2% of the 6 m sections is annular and 7.6% of the 8 m sections is annular.

Since the n values in Table 4 are based on the assumption that the entire pipe has helical corrugations, the question arises as to the effect of short lengths of annular corrugations located at intervals along an otherwise helically corrugated pipe. Studies to determine this effect were made by Smith for Saskatchewan Highways and Transportation.

It is to be expected that the pipe resistance for a pipe with both annular and helical corrugations must fall between the limiting resistance values for 100% annular/100% helical. While this was confirmed by experiment, it was also found that the annular corrugations were somewhat more dominant. For example, for 8 m sections, which are 6.6% annular, the pipe resistance was greater than for all helical pipe by 9% of the difference between all helical and all annular. For 6 m sections, which are 10.2% annular, the factor was 17% and for 4 m sections, which are 15.3% annular, the factor was 26%.

Rather than adjusting n values to account for the presence of annular corrugations, since there would have to be different n values for each section length and pipe size, it was considered more convenient to calculate the pipe capacity in the normal manner, assuming no annular corrugations, and make a correction to the result. The required correction is shown on Figure 12, which gives the percent reduction in discharge capacity as a function of pipe length and diameter. The curves have been calculated using the previously stated factors and assuming a plain projecting inlet on the pipe.

It is seen from Figure 12 that the correction becomes smaller as the diameter increases, the pipe section length increases, or the culvert length decreases. The diameter effect is due to the fact that the difference in n value between helical and annular pipe becomes smaller as the diameter increases, so that the presence or absence of rolled annular ends becomes less and less significant. Any correction less than 2% could be ignored, so only culverts smaller than 1000 mm diameter would require a correction to the discharge capacity.

The culvert length effect is due to the fact that a larger proportion of the total head loss will be caused by friction loss, rather than inlet and outlet losses, when the culvert becomes long. For example, the upper curve on Figure 12, for the 400 mm pipe constructed with 4 m lengths, would be asymtotic to a percent reduction of 26% for an extremely long pipe where inlet and outlet losses become in insignificant part of the total loss. A large reduction in discharge capacity for small pipe is not surprising because the reduction for a very long 400 m pipe would be 50% if it was changed from all helical to all annular (since the n value for 400 mm annular pipe is double the value for helical pipe).

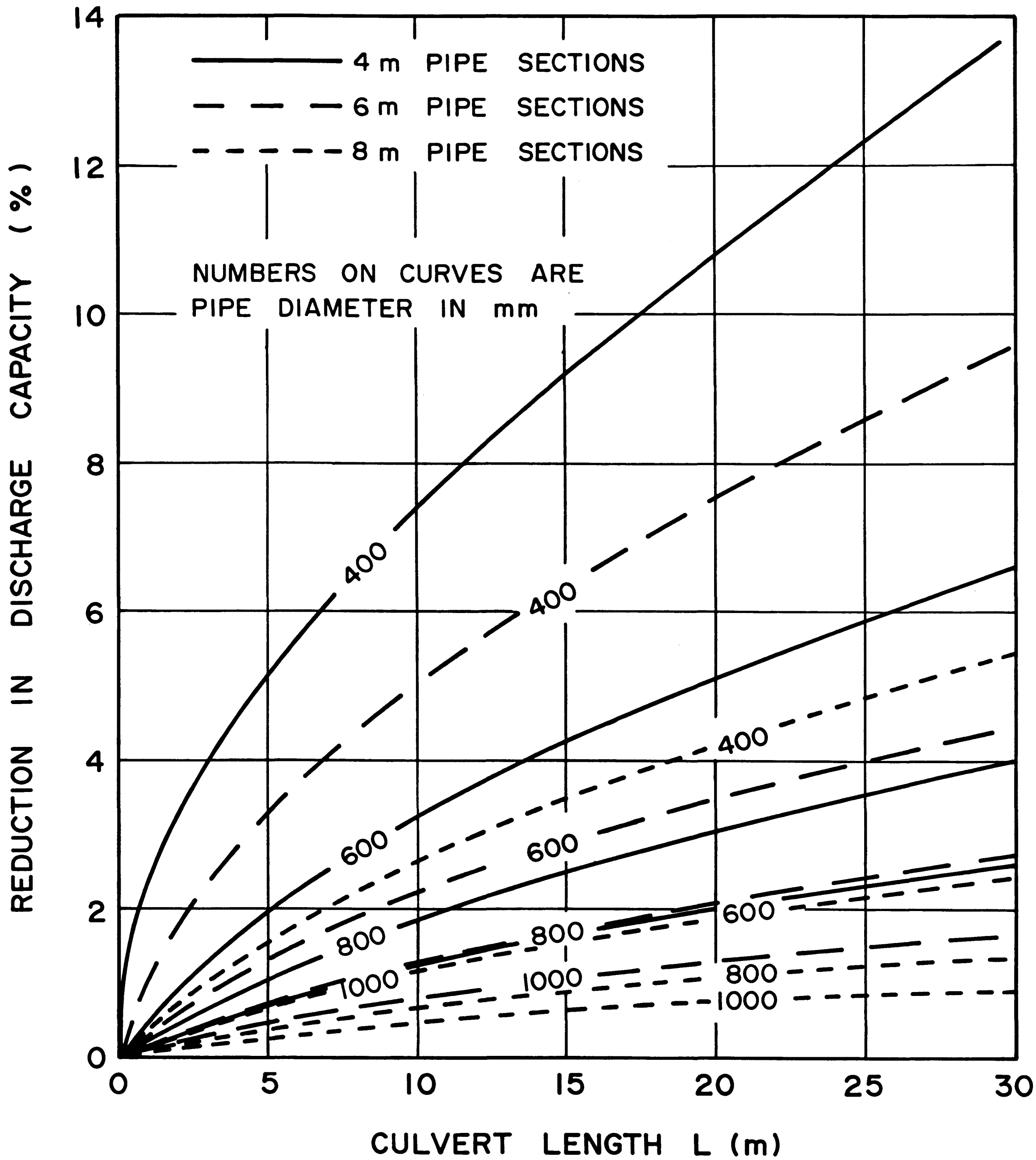

Figure 12. Discharge Capacity Reduction for Helical CSP with Rolled Annular Ends - Outlet Control

10. Outlet Datum

The outlet datum is the reference level at the outlet end of the pipe to which all the losses are added to determine the headwater level. This reference level is often called the effective position of the hydraulic grade line. It represents the elevation at the outlet where the hydraulic grade line, if extended downstream, would intersect the plane of the outlet. This intersection occurs at a distance y above the invert. The value of y depends on the discharge in the pipe and the degree of submergence by tailwater at the end. The submergence is expressed as d_t/D, in which d_t is the tailwater depth above the invert at the outlet. The effect for circular pipe is shown in Figure 13.

Up to a value of approximately $Q/D^{5/2} = 2$, a culvert will not flow full at the outlet end unless submerged by tailwater. The level of the free liquid surface near the end of the pipe is in fact the elevation of the hydraulic grade line. For a culvert with a mild slope, this level cannot drop below the critical depth regardless of how low the tailwater level may be. Hence, in the small discharge range $0 < Q/D^{5/2} < 2$ the value of y is either critical depth or the tailwater depth, whichever is greater. Since the critical depth curve initially rises quite steeply, and since the tailwater is normally shallow for small flows, the critical depth will almost invariably exceed the tailwater depth in this range. Theoretical values for the critical depth ratio d_c/D as a function of $Q/D^{5/2}$ are given in Table 5, and were used to plot the critical depth line in Figure 13.

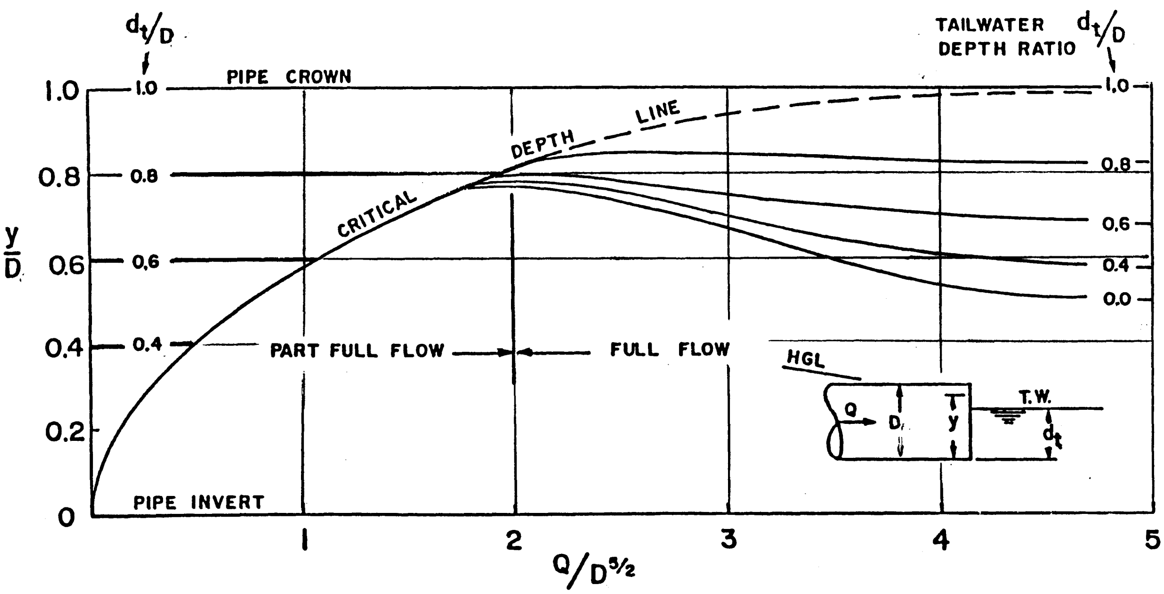

Figure 13. Effective Position of the Hydraulic Grade Line

According to the theory of critical flow a circular section could never flow full because the discharge required for full flow is infinitely great. In fact, the pipe will begin to flow full almost to the end when $Q/D^{5/2}$ exceeds a value of approximately 2. Although critical depth can still be calculated mathematically for these larger flows, it has no particular significance in terms of determining y because the character of the flow is completely changed. For example, at a high discharge ($Q/D^{5/2} = 4.5$) and no tailwater ($d_t/D \leq 0$), the jet will discharge with parallel streamlines (see Figure 18). The pressure in the jet will be atmospheric and the reference point will be at the center of gravity of the jet (i.e., y/D = 0.5). Accordingly, there is a transition for y/D from the critical depth line at approximately $Q/D^{5/2} = 2$ to a value of y/D = 0.5 at $Q/D^{5/2} \geq 4.5$. This is shown by the lower curve on Figure 18.

TABLE 5

CRITICAL DEPTH FOR A CIRCULAR SECTION

$\frac{d_c}{D}$	$\frac{Q}{D^{5/2}}$	$\frac{d_c}{D}$	$\frac{Q}{D^{5/2}}$	$\frac{d_c}{D}$	$\frac{Q}{D^{5/2}}$
0.00	.....	0.35	0.3887	0.70	1.4718
0.01	0.0003	0.36	0.4104	0.71	1.5138
0.02	0.0014	0.37	0.4327	0.72	1.5564
0.03	0.0030	0.38	0.4555	0.73	1.6000
0.04	0.0054	0.39	0.4788	0.74	1.6445
0.05	0.0085	0.40	0.5027	0.75	1.6899
0.06	0.0122	0.41	0.5271	0.76	1.7365
0.07	0.0165	0.42	0.5520	0.77	1.7842
0.08	0.0215	0.43	0.5775	0.78	1.8332
0.09	0.0271	0.44	0.6035	0.79	1.8835
0.10	0.0334	0.45	0.6300	0.80	1.9354
0.11	0.0404	0.46	0.6571	0.81	1.9889
0.12	0.0479	0.47	0.6847	0.82	2.0441
0.13	0.0561	0.48	0.7127	0.83	2.1016
0.14	0.0649	0.49	0.7414	0.84	2.1614
0.15	0.0744	0.50	0.7706	0.85	2.2239
0.16	0.0845	0.51	0.8003	0.86	2.2896
0.17	0.0952	0.52	0.8305	0.87	2.3589
0.18	0.1065	0.53	0.8613	0.88	2.4326
0.18	0.1184	0.54	0.8926	0.89	2.5115
0.20	0.1309	0.55	0.9244	0.90	2.5970
0.21	0.1441	0.56	0.9568	0.91	2.6903
0.22	0.1578	0.57	0.9897	0.92	2.7940
0.23	0.1721	0.58	1.0232	0.93	2.9114
0.24	0.1870	0.59	1.0572	0.94	3.0469
0.25	0.2025	0.60	1.0918	0.95	3.2091
0.26	0.2185	0.61	1.1270	0.96	3.4115
0.27	0.2352	0.62	1.1627	0.97	3.6826
0.28	0.2524	0.63	1.1991	0.98	4.0894
0.29	0.2702	0.64	1.2361	0.99	4.8734
0.30	0.2886	0.65	1.2737	1.00	∞
0.31	0.3075	0.66	1.3119		
0.32	0.3269	0.67	1.3509		
0.33	0.3470	0.68	1.3904		
0.34	0.3676	0.69	1.4308		

Any tailwater submergence, however small, will exert a hydrostatic pressure at the discharge end of the pipe. This will increase the pressure in the discharging jet to a value above atmospheric, and will elevate the effective position of the hydraulic grade line. This effect is shown by the family of transition curves for various tailwater depth ratios. Of course if the tailwater depth equals or exceeds the pipe diameter, then $y/D = d_t/D$.

A problem in culvert design, particularly for small culverts on minor drainage systems, is that the tailwater depth is usually not known. Further, the cost of a channel survey to determine tailwater depth cannot be justified, so the designer is faced with selecting a culvert without having this information. However, it is unlikely that the outlet will be submerged for an installation with one culvert. At the same time, it is equally unlikely that there will be zero tailwater. In fact there must be some tailwater if the culvert invert is set at bed grade elevation. Typically the tailwater depth at design discharge will be somewhere between the centre and the crown of the pipe. A common procedure in practice is to arbitrarily take y equal to the calculated critical depth, shown by the dashed portion of the critical depth line on Figure 13. This may actually be high for most cases, but in the absence of exact knowledge a conservative assumption is necessary. On major installations, for example where a culvert is replacing a bridge, the actual tailwater depth may be known and Figure 13 can be used to get a better estimate of y/D.

11. Culvert Slope

Natural drainage channels in alluvial material tend to have flat slopes. For example, it may be shown that for a discharge of 2 m^3/s and a non-scouring velocity of 0.6 m/s, the slope of a 5 m wide channel with $n = 0.020$ would be about 0.0003. If such a slope was maintained along the length of a 16 m long culvert, the drop in elevation would be only 5 mm, a rather insignificant amount. However, it is not required that culvert slopes correspond exactly to prevailing ground slopes over short lengths. Usually, it is desirable to have some slope on the culvert so that it will be free draining. Slopes of 0.5% to 1% are common.

Equation 10 indicates that the culvert slope can increase the differential head available across a culvert by an amount of SL, and would suggest that the culvert capacity could be substantially increased by using much steeper slopes. Such is not the case, however, because if the outlet is not submerged by tailwater, use of a steep slope simply causes the control to revert to the inlet. On the other hand, if the outlet is submerged, culvert slope becomes redundant because the effective head is headwater elevation minus tailwater elevation, independent of pipe slope. Thus, there is an upper limiting value of pipe slope that will cause an increase in discharge for outlet control. If there is no tailwater submergence, this value is equal to the slope of the energy gradient (i.e., friction slope) for the culvert flowing full at the design discharge, and is given by

[21] $$S_f = V^2 n^2 / R^{4/3}$$

Slopes given by [21] are also quite flat. For example, for $Q/D^{5/2} = 2.5$ and a 1000 mm diameter culvert, S_f would be 0.93% for RCP and 3.71% for annular CSP. If the pipe slope is steeper than the energy gradient, the hydraulic grade line will be located below the crown of the pipe at the inlet and either inlet control or slug flow can be expected to develop.

For part full flow, the limiting maximum slope for outlet control is equal to the critical slope. This condition is covered in more detail in Part D.

Example 3:

A helical CSP 24 m long must be installed at a highway site where the 1 in 20 discharge, selected as the design discharge, is expected to be 2.2 m^3/s. The maximum permissible headwater depth will be 1.8 m, above which the grade will be overtopped. It is estimated that the tailwater level will be 0.5 m above the invert at the outlet at the design discharge. The culvert will be placed on a 0.005 slope and will be equipped with a cylinder inlet with flush invert. What size of pipe is required?

Various terms needed for the solution, such as flow velocity, Manning's n and $Q/D^{5/2}$, all depend upon the unknown diameter. Hence, a trial and error solution is required. However, trial and error for inlet control is quite simple, and even if inlet control does not govern, this would at least give an indication of the probable pipe size. Assuming a 1000 mm diameter pipe, then H/D = 1.8 and from Figure 6, $Q/D^{5/2} = 2.65$, and hence Q = 2.65 m^3/s. This discharge exceeds 2.2 m^3/s by a substantial margin, so try a 900 mm diameter pipe, for which H/D = 2.0, $Q/D^{5/2} = 2.88$, and $Q = 2.88 \times 0.9^{5/2} = 2.21$ m^3/s. Thus, a 900 mm pipe would be satisfactory if inlet control governs.

For outlet control, with a 1000 mm diameter pipe, $Q/D^{5/2} = 2.2/1^{5/2} = 2.2$, so the pipe will flow full in spite of the fact that the tailwater depth is less than the diameter. From Figure 13, for $Q/D^{5/2} = 2.2$ and $d_t/D = 0.5$, y/D = 0.77 so y = 0.77 m. From Table 4, n = 0.0193, and from Table 3, $K_e = 0.3$. The required differential head across the culvert will be:

$$\begin{aligned} h &= (K_e + K_d + 2_g\, 0.0193^2 \times 24/0.25^{4/3} + 1)\, V^2/2g \\ &= (0.3 + 0.2 + 1.113 + 1)\, V^2/2g \\ &= 2.613\, V^2/2g \end{aligned}$$

For full flow, V = 2.2/0.785 = 2.803 m/s and $V^2/2g = 0.400$, so h = 1.045 m. The headwater depth will be:

$$\begin{aligned} H &= y + h - SL \\ &= 0.77 + 1.045 - 0.005 \times 24 \\ &= 1.695 \text{ m} \end{aligned}$$

The 1000 mm pipe is satisfactory for outlet control. A 900 mm pipe could not be used because, since head losses vary as $1/D^4$, the h term would increase from 1.045 m to approximately $(1/0.9^4)$ 1.045 = 1.593 m. Finally, from Figure 12, the capacity reduction for a 24 m long 1000 mm pipe with 8 m sections is less than 1%, and need not be considered.

D. PART FULL FLOW

12. <u>Occurrence</u>

A culvert will flow partly full (i.e., free surface flow) any time that both the inlet and outlet are not submerged, regardless of whether there is inlet or outlet control. This condition occurs most of the time when a culvert passes discharge because the design flow is usually a rare event, and even then it may last only for a short period of time. However, analysis of part full flow at less than the design discharge may still be required in some situations. For example, if the highway crosses a stream where fish migration must be considered, it may be necessary to impose limiting maximum flow velocities for discharges expected during the period of fish migration. Depths and velocities would have to be evaluated along the length of the culvert to insure that the limits are satisfied. There are also cases where part full flow may be specified even for the design discharge. This situation could occur where a very large culvert is used on a major water course and it is required to pass floating ice or debris. In any case, it is not common to have inlet submergence on very large diameter culverts because excessive velocities would occur. For example, use of H/D = 1.5 for design on a 3600 mm culvert could result in flow velocities of 6 m/s, or even greater, depending on the conditions. Part full flow will occur even when the inlet is submerged if there is inlet control. In this case, the part full flow condition will have no influence on determination of the required culvert size, but it may still be necessary to calculate the outlet velocities corresponding to the part full flow depth. Finally, where a culvert is located on a continuously flowing stream, it may be desirable to establish the complete rating curve for the culvert, including the part full flow regime. Again, evaluation of the water surface profile through the culvert would be required.

13. Analysis

For part full flow with outlet control, the culvert slope must always be less than critical slope. The outlet datum is either critical depth or tailwater depth, whichever is greater. Critical depth may be determined from Table 5 in Section 10. The water surface profile extending upstream from the outlet datum will be an M1 or M2, depending on whether the datum level is larger or smaller than the uniform flow depth. If the culvert has zero slope, the water surface profile will always be an H2. The uniform flow depth for a sloping culvert can be calculated with reference to Table 6, which gives the flow depth to diameter ratio d/D as a function of $Qn/(D^{8/3} S^{1/2})$. If $Qn/(D^{8/3} S^{1/2})$ exceeds the tabled value, the culvert will simply flow full. If the uniform flow depth is less than the critical depth, the flow will be supercritical, and inlet control will govern.

It was stated previously for full flow that limiting maximum culvert slopes to maintain outlet control are quite flat. The same statement applies to part full flow. For part full flow, the slope must be less than critical slope. Critical slope is not a constant value, but varies with the flow depth, diameter and n value for the culvert. For example, for d/D = 0.5 (half full), Table 5 gives $Q/D^{5/2} = 0.7706$ and Table 6 gives $Qn/(D^{8/3} S^{1/2}) = 0.156$. These two expressions can be equated, eliminating Q, to solve S explicitly as

[22] $$S = 24.4\, n^2/D^{1/3}$$

in which S is the slope to maintain critical flow at half depth. For a 1000 mm RCP (n = 0.012) [22] gives S = 0.0035, and for a 1000 mm annular CSP (n = 0.024), S = 0.0141. If the exercise is repeated for d/D = 0.75, then

[23] $$S = 35.4\, n^2/D^{1/3}$$

for which the previous slopes would be increased by 45%. It is to be noted that a given culvert slope could produce inlet control for one discharge and outlet control for a larger discharge.

TABLE 6

UNIFORM FLOW IN CIRCULAR SECTIONS FLOWING PARTLY FULL

$\frac{d}{D}$	$\frac{Qn}{D^{8/3}\,S^{1/2}}$	$\frac{d}{D}$	$\frac{Qn}{D^{8/3}\,S^{1/2}}$	$\frac{d}{D}$	$\frac{Qn}{D^{8/3}\,S^{1/2}}$
0.00	0.00	0.35	0.0820	0.70	0.261
0.01	0.00005	0.36	0.0864	0.71	0.266
0.02	0.00021	0.37	0.0909	0.72	0.271
0.03	0.00050	0.38	0.0956	0.73	0.275
0.04	0.00093	0.39	0.1027	0.74	0.280
0.05	0.00149	0.40	0.0105	0.75	0.284
0.06	0.00221	0.41	0.1099	0.76	0.289
0.07	0.00306	0.42	0.1147	0.77	0.293
0.08	0.00406	0.43	0.1197	0.78	0.297
0.09	0.00522	0.44	0.1248	0.79	0.301
0.10	0.00651	0.45	0.1298	0.80	0.305
0.11	0.00795	0.46	0.1353	0.81	0.308
0.12	0.00954	0.47	0.1400	0.82	0.312
0.13	0.01127	0.48	0.1454	0.83	0.315
0.14	0.01314	0.49	0.1507	0.84	0.318
0.15	0.0151	0.50	0.156	0.85	0.321
0.16	0.0173	0.51	0.161	0.86	0.324
0.17	0.0196	0.52	0.166	0.87	0.326
0.18	0.0220	0.53	0.172	0.88	0.328
0.19	0.0246	0.54	0.177	0.89	0.330
0.20	0.0273	0.55	0.182	0.90	0.332
0.21	0.0301	0.56	0.188	0.91	0.334
0.22	0.0331	0.57	0.193	0.92	0.335
0.23	0.0361	0.58	0.199	0.93	0.335
0.24	0.0394	0.59	0.204	0.94	0.335
0.25	0.0427	0.60	0.209	0.95	0.335
0.26	0.0462	0.61	0.215	0.96	0.334
0.27	0.0497	0.62	0.220	0.97	0.332
0.28	0.0534	0.63	0.225	0.98	0.329
0.29	0.0571	0.64	0.231	0.99	0.325
0.30	0.0610	0.65	0.236	1.0	0.312
0.31	0.0651	0.66	0.241		
0.32	0.0691	0.67	0.246		
0.33	0.0733	0.68	0.251		
0.34	0.0776	0.69	0.256		

When the conditions for part full flow with outlet control are met, the water surface profile may be calculated using the non-uniform flow equation:

[24] $\Delta x = (E_2 - E_1)/(S_o - S_{fav})$

in which Δx is the length of the profile segment in m, E_2 is the energy content at the end of the segment in m, E_1 is the energy content at the start of the segment in m, and S_{fa} is the average rate of friction loss. The calculation is started at the outlet where d_2 and E_2 are known. The value of d_1 may be arbitrarily set at 1.1 d_2 for an M2 curve, or 0.9 d_2 for an M1 curve, and the corresponding energy at the upstream end of the length segment solved from

[25] $E_1 = d_1 + V_1^2/2g$

The average rate of friction loss is given by

[26] $S_{fa} = (S_{f1} + S_{f2})/2$

in which

[27] $S_{f1} = V_1^2\, n^2/R_1{}^{4/3}$

and

[28] $S_{f2} = V_2^2\, n^2/R_2{}^{4/3}$

The steps are repeated until the summation of the Δx increments equals the length of the culvert. At this point, the entrance head loss is added to the E_1 value just inside the culvert, giving the upstream energy level. When the calculated flow depth inside the culvert at the inlet is less than the pipe diameter, the velocity corresponding to this depth is used to calculate the inlet loss (as in $h_e = k_e\, V^2/2g$). If the calculated d_1 for one of the steps becomes equal to the clear height, D, of the culvert, the culvert will flow full from that point all the way upstream to the inlet. Full flow analysis must be used for that portion of the culvert.

The area and hydraulic radius relationships for part full flow are given in Table 7, showing A/D^2 and R/D as functions of d/D. Values of A are required for solution of [25] and R for [27] and [28].

14. Inlet Loss Coefficients

Since the shape of the inlet area below the headwater level varies with H/D when H/D < 1, it follows that the inlet loss coefficient will not necessarily be constant or equal to the value for a fully submerged inlet. This fact is demonstrated on Figure 7, as presented previously in Section 8 for full flow. Figure 7 shows that the inlet loss coefficients for a bellmouth, tongue-and-groove RCP and flush headwall show minimal variation, and the same vlaue of K_e could be used for both full flow and part full flow. However, the loss coefficients are significantly affected in the case of the other inlets.

TABLE 7

PROPERTIES OF CIRCULAR SECTIONS FLOWING PARTLY FULL
d=flow depth; D = pipe diameter; A = flow area; R = hydraulic radius

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$
0.00	....		0.35	0.2450	0.1935	0.70	0.5872	0.2962
0.01	0.0013	0.0066	0.36	0.2546	0.1978	0.71	0.5964	0.2975
0.02	0.0037	0.0132	0.37	0.2642	0.2020	0.72	0.6054	0.2987
0.03	0.0069	0.0197	0.38	0.2739	0.2062	0.73	0.6143	0.2998
0.04	0.0105	0.0262	0.39	0.2836	0.2102	0.74	0.6231	0.3008
0.05	0.0147	0.0325	0.40	0.2934	0.2142	0.75	0.6319	0.3017
0.06	0.0192	0.0389	0.41	0.3032	0.2182	0.76	0.6405	0.3024
0.07	0.0242	0.0451	0.42	0.3130	0.2220	0.77	0.6489	0.3031
0.08	0.0294	0.0513	0.43	0.3229	0.2258	0.78	0.6573	0.3036
0.09	0.0350	0.0575	0.44	0.3328	0.2295	0.79	0.6655	0.3039
0.10	0.0409	0.0635	0.45	0.3428	0.2331	0.80	0.6736	0.3042
0.11	0.0470	0.0695	0.46	0.3527	0.2366	0.81	0.6815	0.3043
0.12	0.0534	0.0755	0.47	0.3627	0.2401	0.82	0.6893	0.3043
0.13	0.0600	0.0813	0.48	0.3727	0.2435	0.83	0.6969	0.3041
0.14	0.0668	0.0871	0.49	0.3827	0.2468	0.84	0.7043	0.3038
0.15	0.0739	0.0929	0.50	0.3927	0.2500	0.85	0.7115	0.3033
0.16	0.0811	0.0985	0.51	0.4027	0.2531	0.86	0.7186	0.3026
0.17	0.0885	0.1042	0.52	0.4127	0.2562	0.87	0.7254	0.3018
0.18	0.0961	0.1097	0.53	0.4227	0.2592	0.88	0.7320	0.3007
0.19	0.1039	0.1152	0.54	0.4327	0.2621	0.89	0.7384	0.2995
0.20	0.1118	0.1206	0.55	0.4426	0.2649	0.90	0.7445	0.2980
0.21	0.1199	0.1259	0.56	0.4526	0.2676	0.91	0.7504	0.2963
0.22	0.1281	0.1312	0.57	0.4625	0.2703	0.92	0.7560	0.2944
0.23	0.1365	0.1364	0.58	0.4724	0.2728	0.93	0.7612	0.2921
0.24	0.1449	0.1416	0.59	0.4822	0.2753	0.94	0.7662	0.2895
0.25	0.1535	0.1466	0.60	0.4920	0.2776	0.95	0.7707	0.2865
0.26	0.1623	0.1516	0.61	0.5018	0.2799	0.96	0.7749	0.2829
0.27	0.1711	0.1566	0.62	0.5115	0.2821	0.97	0.7785	0.2787
0.28	0.1800	0.1614	0.63	0.5212	0.2842	0.98	0.7817	0.2735
0.29	0.1890	0.1662	0.64	0.5308	0.2862	0.99	0.7841	0.2666
0.30	0.1982	0.1709	0.65	0.5404	0.2882	1.00	0.7854	0.2500
0.31	0.2074	0.1756	0.66	0.5499	0.2900			
0.32	0.2167	0.1802	0.67	0.5594	0.2917			
0.33	0.2260	0.1847	0.68	0.5687	0.2933			
0.34	0.2355	0.1891	0.69	0.5780	0.2948			

The loss coefficient for the Armtec end section is smaller than for the full flow case, becoming as little as 0.2 when H/D = 0.5. On the other hand, the loss coefficient for the cylinder inlet is greater than for the full flow case, becoming 0.5 at H/D = 0.5. At H/D = 1, the loss coefficients are essentially equal, at 0.4, for both inlets. This result suggests that the Armtec end section would be more appropriate for culverts which are not intended to operate submerged at the design discharge whereas the cylinder inlet would be more appropriate where submergence is intended. This argument applies where inlet control governs as well. Any treatment which improves the efficiency for inlet control also improves the efficiency for outlet control.

15. Friction Losses

The n values to be used in [27] and [28] are not necessarily the same as for full flow. It has been argued that the pattern of secondary currents in a circular section flowing partly full is different than for full flow, and therefore the boundary shear will be affected. Measurements on concrete sewer pipe of small size (100 mm to 300 mm), reported by Camp in 1944, suggested n values 24% higher for half full flow than for full flow. Since the frictional resistance varies as n squared, such a large increase in n appears difficult to justify based on the effect of secondary currents alone.

The 1950 SAF tests by Straub and Morris on full scale RCP and annular CSP (up to 916 mm diameter) did not show a marked difference in n value for part full flow. Experimental values from their work is shown in Table 8. On the basis of these results, it is recommended that n values for part full flow analysis be the same as for full flow, namely 0.012 for RCP and 0.024 for annular CSP.

TABLE 8

EXPERIMENTAL n VALUES FOR PART FULL FLOW

RCP (456 mm)		Annular CSP (610 mm)	
$\frac{d}{D}$	n	$\frac{d}{D}$	n
0.393	0.0110	0.300	0.0230
0.463	0.0110	0.397	0.0238
0.530	0.0109	0.507	0.0234
0.627	0.0106	0.600	0.0236
0.640	0.0104	0.703	0.0238
0.717	0.0104	0.800	0.0238
0.763	0.0102	0.873	0.0234
0.840	0.0104		
Average	0.0106		0.0235

(From SAF reports by Straub and Morris, 1950)

Values of n may change for part full flow if the surface texture of the pipe wall varies. For example, if the invert area of RCP becomes roughened by abrasion, pitting, spalling or chemical attack, it may be expected that the effective n value would increase as the flow depth decreases. Conversely, for CSP with the asphalt cement paved invert, the n value will decrease for shallow flow. The paved invert pipe is often recommended when

CSP is to be used for sewers. The valley between the corrugations in the lower half of the pipe is filled, leaving a smooth surface. In this case, a paved invert CSP flowing half full would have an n value of 0.012.

If n values can be assigned to different parts of the wetted perimeter of a regular section, the composite n for the whole section can be determined from

[29] $$n_c = \Sigma(P_1 n_1 + P_2 n_2 + ...)/\Sigma P$$

in which n_c is the composite n, P_1 is the wetted perimeter with an n value of n_1, P_2 is the wetted perimeter with an n value of n_2, etc. and ΣP is the total wetted perimeter.

Part full flow in helical CSP is a special case. The reduced n values for full flow occur because the flow rotates in a smooth circular spiral without flow deformation, minimizng the interaction with the corrugations. This pattern is not possible for part full flow. The helical corrugations will still tend to rotate the flow, but in the absence of a full circle, the flow will simply ride at a higher elevation on one side of the pipe than the other and surface cross flow will occur from the high side to the low side. This type of rotational flow in an open channel is accompanied by considerable deformation in the flow pattern, increased shear stress, and increased head loss, and is responsible for the extra head loss which occurs. The result is that the n value for helical pipe flowing partly full is greater than for full flow.

The resistance values for helical CSP flowing partly full will be different for each size of pipe, as is the case for full flow as well, because the helix angle is different for each diameter. Further, this resistance will be dependent upon the depth ratio d/D.

Unlike the situation for annular CSP, no full scale tests have been run on helical CSP flowing partly full. However, the effect has been deduced from limited model tests, as reported by Smith in 1993. The results are shown in Figure 14, and are recommended for design at this time.

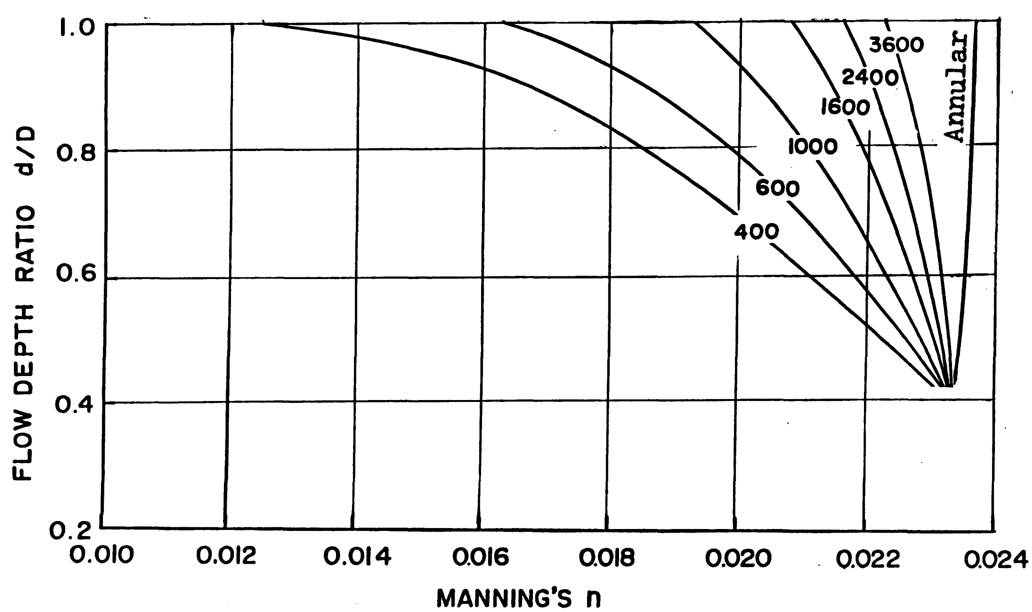

Figure 14. Effect of Flow Depth on Manning's n for Part Full Flow in Helical CSP

16. Wood Box Culverts

Although there are a number of wood box culverts (WBC) in existence, use of the wood box culvert as an alternative to other types has declined in recent years and few are constructed today. Installation tends to be labour intensive, requiring more on-site construction than the pre-fabricated RCP or CSP. However, there are certain situations where a WBC may be appropriate. Wood tends to be non-reactive to soil chemicals which could produce corrosion of steel or sulphate attack on concrete. Also, rectangular wood box culverts may be built with a width to depth ratio up to 4, and may be preferred to circular shapes for passing floating debris or ice (similar to a single span bridge). In the usual case, the WBC is intended to flow partly full, and is being introduced here as an example of analysis for part full flow.

The rectangular wood box culvert consists of treated timber planking covering supported by an inside timber framework. A typical cross-section is shown in Figure 15. The timber frame has horizontal roof beams, called stringers, which carry the load due to the earth cover and traffic on top of the culvert. Vertical posts on each side carry the vertical end reactions for the stringers, and in addition resist lateral earth pressure. The floor of the culvert consists of cross sills located between each pair of posts. The cross sills, which carry the horizontal reaction at the bottom of the posts, are placed directly on grade in the excavation for the culvert. Since the planking covers only the top and sides of the culvert, the space between the cross sills is filled with riprap for erosion protection.

Standard designs allow for construction of post heights (h) of 900 mm, 1200 mm, 1800 mm and 2400 mm, and box widths (w) of 1200 mm, 1800 mm, 2400 mm, 3000 mm and 3600 mm. Thus there are 20 possible combinations of height and width. The clear depth of the box (D) is smaller than the post height by 100 mm, due to the presence of the spreader and cross sill. Culvert lengths can be constructed in multiples of 1.2 m, starting with a minimum length of 6.0 m.

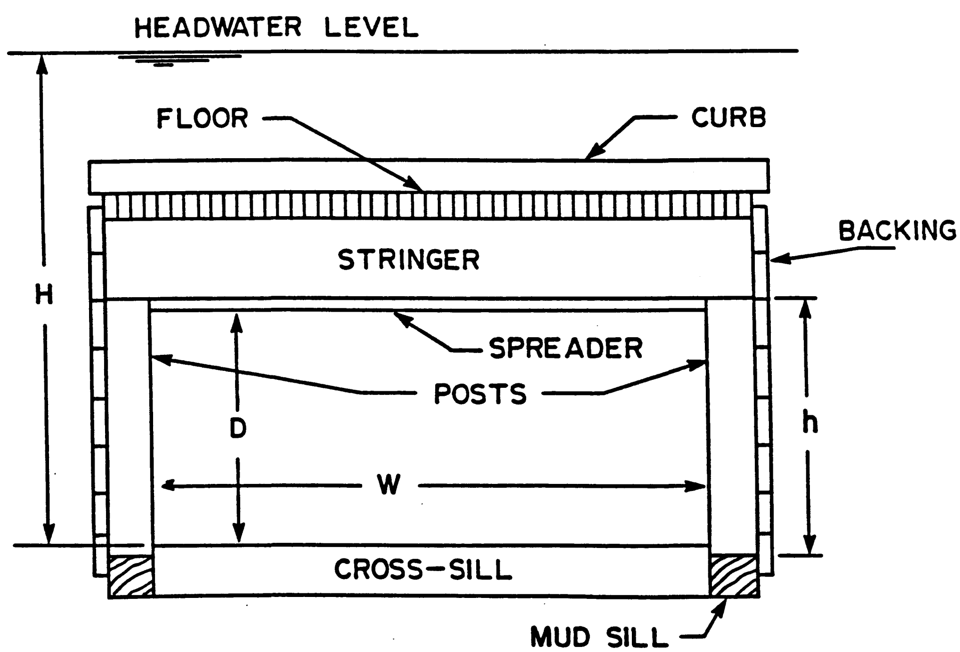

Figure 15. Cross Section Details for Wood Box Culvert

The posts and cross sills are all the same size, 150 mm x 200 mm. The riprap between the cross sills is specified as 100 mm to 150 mm, placed at a layer depth of 200 mm, and therefore fills the space between the sills up to the level of the top of the sill. The depth of the stringers varies with the width of the box, being 300 mm for the narrowest box (1200 mm wide), and 450 mm for the widest box (3600 mm wide). In all cases, the centre-to-centre spacing of each support frame is 800 mm.

The inlet and outlet structure for the culvert are also framed timber construction. They consist basically of earth retaining wingwalls which are tapered down to match the fill slope on each side of the roadway. The outlet wingwalls continue parallel to the culvert axis. The inlet wingwalls are flared out at 30 degrees to the axis in order to give some hydraulic improvement in the entry conditions to the culvert.

Model tests have shown that in spite of the difference in the head loss mechanism on the sides than on the floor, the resistance of the posts is comparable to the resistance for the riprap. As a result, the value for Manning's n remains relatively constant, except at shallow depths of flow where the resistance is greater. The n value on a 1:10 scale model was 0.023, which becomes $0.023 \times 10^{1/6} = 0.034$ for the prototype. This value is much higher than the n value for annular CSP, which is to be expected. The recesses between the posts are large, the posts are square cornered, and for 120 mm riprap, the n value, from $n = 0.049\ d_m{}^{1/6}$, would be 0.034.

The inlet loss coefficient is given by

[30] $$K_e = 0.7\ h/W$$

which shows that the loss is affected by the shape of the culvert. The primary cause of the loss is interaction of flow with the posts on the sides of the wingwalls and contraction at the inlet. These effects are smaller for wider culverts because a smaller proportion of the water prism is affected by the conditions at the sides.

Wood box culverts are constructed with zero slope. Thus, an H2 water surface profile always occurs throughout the length of the culvert. Such a profile is shown on Figure 16. The profile was calculated for a 1800 mm wide by 1200 mm high WBC, starting at critical depth at the outlet, and using $n = 0.034$, $K_e = 0.47$, and shows excellent agreement with the observed profile taken from a 1:10 scale model.

E. EROSION CONTROL

17. General

Erosion control at culverts may be required for a variety of reasons. The most important of these is to prevent failure of the culvert and the embankment through which it is placed. This type of failure can occur if scour at the outlet is allowed to proceed unchecked, undermining the pipe and eroding the toe of the fill. In addition, a large scour hole may be a safety hazard or an inconvenience for roadside operating equipment. Another reason for erosion control is to prevent damage to adjacent roads or farmland located near the culvert outlet. Erosion can persist for a considerable length downstream from the culvert outlet unless it is prevented. Finally, erosion control may be desired for aesthetic reasons.

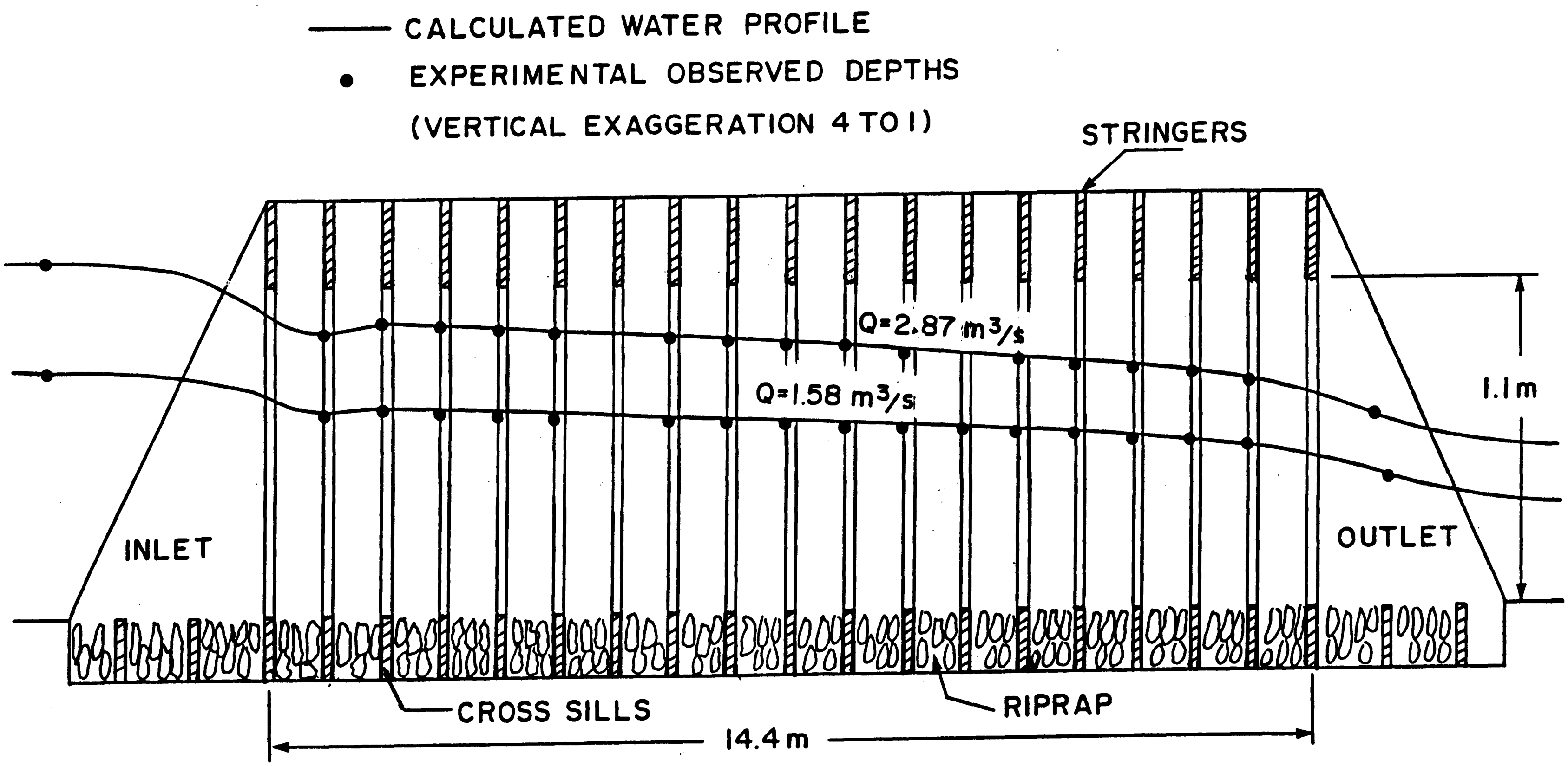

Figure 16. Theoretical and Observed Water Surface Profile Through Culvert; 1800 mm x 1200 mm WBC, n = 0.034; K_e = 0.7 h/W; No tailwater

At a culvert inlet the flow approaches more or less uniformly from all directions as it converges on the inlet. The result is that the velocity a short distance away from the inlet will be much smaller than the velocity inside the culvert. The zone of influence for contracting flow is small, so any protection need not extend more than one pipe diameter upstream from the inlet, or one pipe diameter on each side. In fact, if the crown of the culvert extends upstream from the embankment more than one diameter, slope protection should not be needed except near the bottom of the slope in the vicinity of the culvert invert.

At a culvert outlet jet flow will occur. Velocity reduction occurs along the path of the jet as it decelerates. However, jet flow can persist for many pipe diameters beyond the end of the culvert because the diffusion process which decelerates the flow is not very efficient. In addition, the culvert outlet velocity will normally be larger than the culvert inlet velocity. The result is that scour protection at a culvert outlet must be much more extensive than at the inlet.

There are two basic approaches to erosion control. The culvert may be designed for low non-erosive exit velocities, or scour protection may be provided. In the former case, either an oversize culvert is used or some form of outlet structure is designed to dissipate the excess kinetic energy of flow and reduce the discharge velocity. In the latter case the flow is allowed to discharge into the channel or ditch with unimpeded velocity and the condition is resisted with riprap. If a structure is required it may consist of a concrete apron with baffles, an impact basin, or a hydraulic jump basin. Structures would normally be considered only for major installations with large discharges and velocities.

The method selected for erosion control should always be on the basis of least cost. Normally riprap is given first consideration, and is used almost exclusively for outlet velocities up to 4 m/s. For velocities greater than this some form of outlet structure may have to be considered. As will be seen in Section 19, stone sizes become prohibitively large for high velocities. Whether riprap can be used alone depends upon the availability of riprap (as regards both size and quantity), the size and importance of the culvert, and the natural erosion resistance of the material in the discharge channel.

Tailwater information is often not available for culvert installations, and in this case hydraulic jump outlet structures are not recommended. If velocities are high, the bed material erodible, and adequate sized riprap not available, some benefit can be gained by using a concrete apron with baffles. The flow is allowed to spread out between wingwalls, some energy is dissipated, and smaller riprap can be used.

18. Natural Erosion Resistance

The natural erosion resistance of a ditch or channel depends upon the grain size of the boundary material, the degree of cohesion of the material, and the density of vegetation, if any. If the shear stress on the boundary, often referred to as tractive force, exceeds the shearing resistance of the material, scour will occur. The shear stress depends primarily upon the velocity of flow, but it is also affected by the size and shape of the water prism, the Reynolds number and the roughness of the boundary. Although an oversimplication of the problem, the permissible velocity approach pioneered by Fortier and Scobey is often used. More complicated and sophisticated methods are not usually necessary or justifiable for analysis of culverts.

Table 9 gives the permissible mean velocity for various boundary materials. The values for cohesionless and cohesive material are taken from Fortier and Scobey, and the values for grassed channels from Coyle. As expected, higher velocities may be used for coarser material (in the cohesionless category), or for more strongly cemented material (in the cohesive category). Considering that culverts usually operate at the design discharge with low frequency and duration of flow, these values are probably conservative, particularly for cohesive materials. The erosion response time for cohesive material is much slower than for cohesionless material, and excess velocity of short duration may not produce much scour.

TABLE 9

Permissible Mean Velocities for Various Channel Boundary Materials
(Clear water, straight channel, 1 m depth)

Cohesionless materials	
Fine sand	0.46 m/s
Medium sand	0.56 m/s
Coarse sand	0.66 m/s
Fine gravel	0.76 m/s
Coarse gravel	1.22 m/s
Cobbles	1.52 m/s
Cohesive materials	
Sandy loam	0.53 m/s
Silty loam	0.61 m/s
Alluvial Silt	0.61 m/s
Volcanic ash	0.76 m/s
Firm loam	0.76 m/s
Stiff clay	1.14 m/s
Clay shale	1.83 m/s
Hardpan	1.83 m/s
Glacial till	2.00 m/s
Grassed channel	
(in loam soil)	
Brome grass	1.22 m/s
Grass mixtures	1.22 m/s
Kentucky bluegrass	1.52 m/s
Bermuda grass	1.83 m/s

The velocities are based on the assumption that the channel axis is in line with the culvert axis, so that the maximum velocity, which is greater than the mean velocity, will not be in contact with the boundary. If the flow discharging from the culvert is required to make a sharp turn, for example to flow in the ditch beside the road, the allowable velocities should be reduced to three-quarters of the indicated value if bank erosion is to be avoided. Finally, the values apply to flow depths of approximately 1 m. Shear stresses will be greater for shallower flow at the same mean velocity, so the allowable velocity must be marginally reduced. This reduction amounts to 10% at a flow depth of 0.5 m. On the other hand, the values may be increased by 10% for a flow depth of 1.5 m, or by 20% for a flow depth of 2.5 m.

19. Outlet Velocity

The size of riprap required to prevent scour depends primarily on the outlet velocity. If the outlet is submerged the culvert will flow full and the velocity is simply the discharge divided by the cross-sectional area of the pipe. This is the simplest case to analyze. Submergence may be expected for multi-barrel culvert installations where a large number of smaller diameter pipes is used, or in a controlled tailwater situation. The latter is common on irrigation projects where the crown of the pipe is deliberately placed below the tailwater in order to insure full flow at the outlet. This usually requires that the invert of the pipe be placed below the bed level of the drainage channel.

It is common practice in highway engineering to place the culvert invert at the bed level of the drainage course, and it is unlikely that the tailwater depth will exceed the pipe diameter and submerge the outlet at most sites where a single culvert is installed. In this case the culvert will not flow full at the outlet except for large values of $Q/D^{5/2}$. This condition is illustrated on Figure 17, which shows the sequence of water surface profiles which develop at a free outlet for various values of $Q/D^{5/2}$. In the absence of a channel bed flush with the invert at the outlet, the depth at the end of the culvert will be the same as the brink depth. For small discharges the pipe will flow partly full throughout its entire length and the flow will pass through critical depth a short distance upstream from the end of the pipe. As the discharge increases the flow depth in the pipe will also increase. Eventually the pipe may flow full for part of the length and partly full at the outlet, as shown for $Q/D^{5/2} = 1.4$. The point where the full flow separates from the crown of the pipe will move downstream closer to the outlet as the discharge increases. It may be assumed that the pipe will flow completely full at the outlet for any $Q/D^{5/2}$ greater than 3.0 regardless of the absence of any tailwater. Some actual profiles of discharge at the end of a free circular outlet are shown on Figure 18.

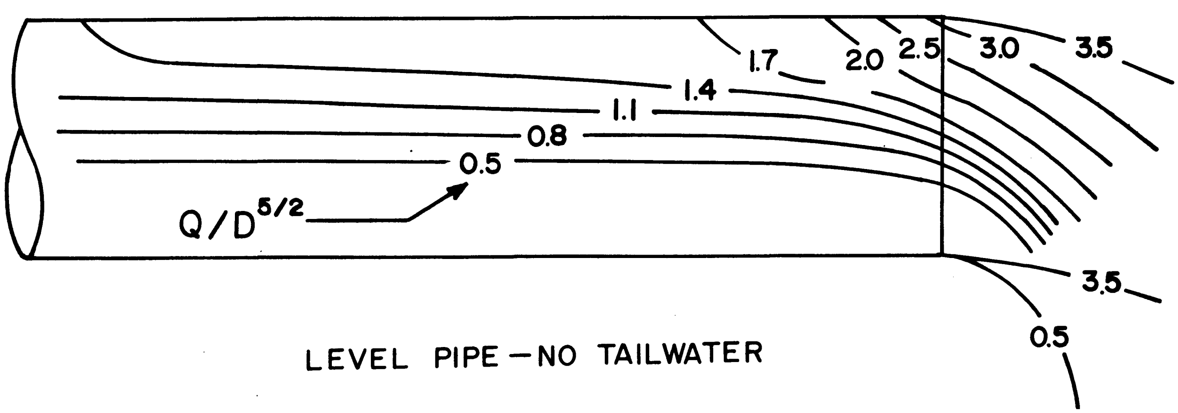

Figure 17. Water Surface Profiles Approaching a Free Outlet

For a given value of $Q/D^{5/2}$ the exact location of the point of separation depends upon the slope and roughness of the pipe. However, for values of $Q/D^{5/2} \leq 2$ the flow in a horizontal pipe or pipe on a mild slope will pass through critical depth d_c a short distance upstream from the outlet. In turn, the brink depth at the end of the pipe, called the end depth d_e, will be somewhat less than critical depth. The horizontal component of velocity at the end of the pipe will be equal to the discharge divided by the end area.

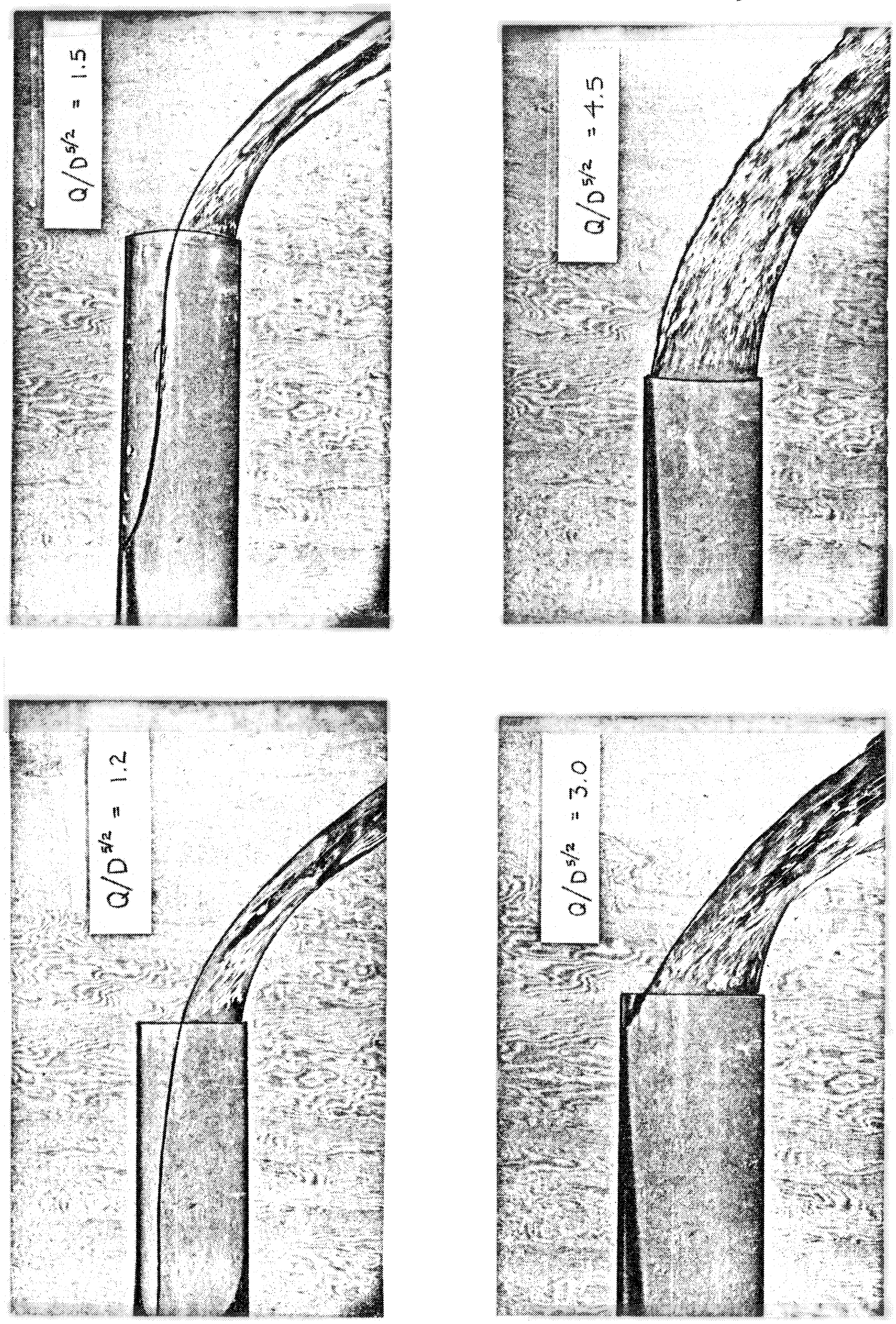

Figure 18. Flow at a Free Pipe Outlet (Zero Slope)

The presence of a channel bed and any tailwater depth d_t above the outlet invert will increase the end depth. The effect is shown on Figure 19, taken from full scale tests by Stevens on culvert outlets at Colorado State University. The end depth ratio d_e/D is seen to be dependent upon both the tailwater depth ratio d_t/D and $Q/D^{5/2}$. For still water ($Q/D^{5/2}$ = 0) the end depth is simply equal to the tailwater depth, as shown by the straight sloping line. The d_e/D intercept at $d_t/D = 0$ is actually 77% of the critical depth for all values of $Q/D^{5/2}$ up to 2, as shown by the solid line curves. The depth value of 77% of critical depth is greater than the brink depth at a free outlet (which is about 73% of critical depth), which simply reflects the influence of the presence of a channel bed at the outlet invert. If the tailwater depth is equal to or greater than critical depth, the end depth will equal the tailwater depth. Hence, where the solid line curves intersect the sloping straight line $d_e/D = d_c/D = d_t/D$. The dashed line curves for $Q/D^{5/2} > 2$ show the relationship for larger values of $Q/D^{5/2}$. In this case the pipe flows full almost to the end, as shown by Figure 17, and there is a rapid transition to full flow as the tailwater depth increases.

The end area A_e is a function of the end depth and culvert diameter. Expressed non-dimensionally, $A_e/D^2 = f\,(d_e/D)$. Table 7 in Section 13 may be used to determine A_e. Once d_e is determined from Figure 19 and A_e from Table 7, the horizontal component of culvert outlet velocity may be calculated by applying the continuity equation.

The foregoing method applies only to the case of a horizontal or mildly sloping culvert. If the culvert is steep, the flow will be supercritical at the outlet and the velocities will be higher. Steep slopes are frequently accompanied by negative pressures at the inlet, slug flow in the barrel, and high velocities at the outlet. Generally, these conditions should be avoided on drainage culverts.

It was pointed out in Section 2 that hydrologic data for small drainage courses is frequently deficient. Data for purposes of tailwater evaluation is often equally deficient. On important installations it is the responsibility of the design engineer to request the necessary field surveys from which the design discharge and corresponding tailwater may be estimated. If it is absolutely necessary to design erosion protection without definite tailwater information, it is suggested that the tailwater depth at design flow be assumed at half a diameter above the invert.

20. Stone Size for Riprap

The forces which tend to dislodge and transport a stone are lift, drag, and shear. Lift occurs due to reduced pressure on top of the stone resulting from pressure drop associated with local flow accelerations over the top of the stone. Drag occurs due to differential pressure across the stone in the direction of flow. Shear occurs due to the fluid friction force which develops at the surface of the stone. These forces act in different combinations for different situations, and any one may be dominant at a particular instant. However, each of these forces is proportional to the cross-sectional area of the stone and the square of the flow velocity, in accordance with the force equations for lift, drag, and shear. If d_m is taken as a characteristic diameter for the stone, then the stone area will vary as d_m^2, and the eroding force will be proportional to $V^2 d_m^2$. The stabilizing force for a stone on the bed is due to its submerged weight, and will be proportional to $\gamma(S_g - 1)d_m^3$. At incipient instability these forces will be equal, and for given values of γ and S_g, d_m will vary as V^2, or

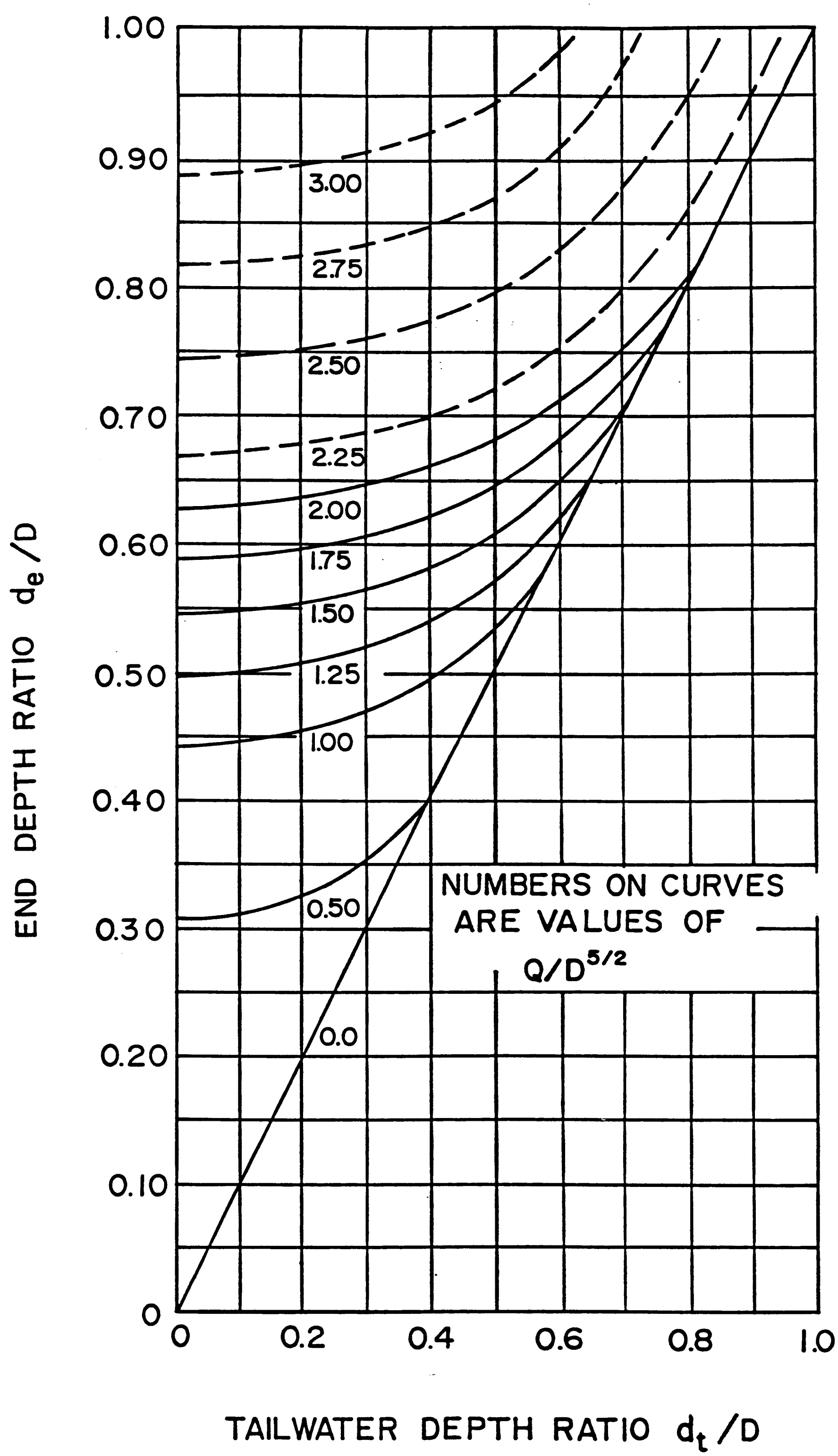

Figure 19. End Depth Ratio for Culvert Oulets (adapted from Stevens, 1969)

[31] $$d_m = k V^2$$

in which k is the coefficient of proportionality. This simple expression has been known since 1829, as embodied in Leslie's "sixth-power law" which states that the required weight of stone to resist erosion varies as the sixth power of the velocity. The difficulty in using [31] for design arises from the fact that k is a function of the size, shape, specific gravity, and gradation of the stone, the velocity distribution and turbulence level of the flow, and the size and shape of the water prism. The problem is further complicated in that the value selected for k will depend upon what stone dimension is designated as d_m, what velocity (i.e., local, mean, maximum) is designated as V, and what factor of safety is to be included.

The limitation of [31] is resolved in practice by ignoring some of the factors. It is assumed that all properties of the stone except specific gravity can be expressed in terms of the equivalent spherical diameter of the median stone, for which 50% of the sample is finer than, by weight. Given the specific gravity as a constant at 2.65 to 2.70, the value of k can then be determined for certain specified hydraulic conditions. For example, for bed riprap below hydraulic jump stilling basins the United States Bureau of Reclamation recommendation gives

[32] $$d_m = 0.041 V^2$$

with d_m in metres and V specified as the bottom velocity in m/s.

The coefficient for [32] would be significantly lower if a smaller factor of safety was used and if the velocity was expressed in terms of the average velocity instead of the bottom velocity. Unfortunately, the same average velocity will produce different bottom velocities, depending on the size and shape of the water prism, and therefore the coefficients would not remain constant in every case. This limitation may be overlooked where culverts are concerned because of the somewhat unique conditions which exist at the outlet. These conditions include a limited range of sizes, limits on culvert shape, a fully developed velocity distribution in the culvert, minor consequences in the event of partial riprap failure, and simple repair procedure, if necessary. It is recommended that the required size of natural field stone be determined from

[33] $$d_m = 0.019 V^2$$

in which d_m is the equivalent spherical diameter of the median stone, and V is the average outlet velocity, as calculated by the method outlined in Section 18. Work by Stevens on full scale pipe outlets at Colorado State University suggests that this equation is conservative.

Equation 33 clearly indicates why protection by riprap alone cannot be contemplated for high velocities. For example, with $V = 6$ m/s, then $d_m = 0.68$ m. Since half the riprap (by weight) consists of larger size stone, and natural stone is not spherical, the longest dimension of the largest stone could be in the order of 1.5 m. Stones of this size may be justified for major hydraulic structures, but are not generally used, or available, for culvert outlets. Accordingly, in culvert hydraulics, considerable attention is given to sizing the culvert to keep outlet velocities below 4 m/s.

21. <u>Extent of Protection</u>

Where riprap is used at culverts it is not uncomon to see a similar treatment used at both ends of the culvert - that is, the riprap is piled around the sides and over the top of the culvert but does not extend very far upstream or downstream from the invert. This "equal" treatment fails to recognize that outlet conditions are always more severe than inlet conditions. As a result, riprap quantity is often exessive at the inlet and inadequate at the outlet.

The zone of contracting flow at an inlet is very localized. At a distance of 1 pipe diameter away from the inlet, in any direction, the flow velocities will be small. In fact, if there is any ponding upstream from the culvert the velocities may be even lower than normal velocities in the ditch. It follows that riprap protection at an inlet can also be localized. In addition, smaller riprap would normally be sufficient at an inlet. The higher velocities corresponding to the contracted jet areas inside the culvert are of no concern as far as scour is concerned. Equation 33 can be applied to the velocity corresponding to the flow area perpendicular to the axis of the pipe at the plane of the inlet. This velocity will invariably be smaller than the velocity at the outlet, unless the pipe flows full.

Model studies have shown that for the projecting inlet or cylinder inlet, riprap extending 1 D upstream, downstream and to each side of the invert is sufficient. This placement defines an area 2D by 2D for the limits of riprap. It is important to note that the 1D extension downstream from the invert, placed on the sloping shoulder of the grade, would not reach as high as the crown of the pipe. When the crown of the pipe projects 1.5 to 2 diameters upstream from the grade, no protection is needed for that area because flow velocities in contact with the grade will be negligible at that position.

In the case of the tapered inlet or Armtec inlet, for which the fill is placed up to the edge of the inlet around the entire perimeter of the opening, additional riprap is required. The flow at the inlet edge of the culvert has contact with the adjacent fill over the full height of the opening. The riprap should extend 1D upslope from the crown of the pipe. For a 2H:1V sideslope for the shoulder, this placement would define an area of approximately 2D by 4D (or double the area for the projecting inlet). Of course the upslope extension of the riprap need not exceed the maximum level of the headwater. This level may be lower than the crown of the pipe for a pipe which is designed to flow partly full at the design discharge.

Scour at the outlet is caused by the jet and the eddies which form adjacent to the jet. These eddies begin where the discharging flow from the outlet first makes contact with the water on each side of the jet, and persist downstream until the jet has expanded to the full channel area. With the projecting outlet, for which the crown of the pipe extends 1.5 to 2D downstream from the shoulder of the grade, no riprap is needed on the slope above a height of D/4, regardless of the depths of tailwater, because the eddy action upstream from the end of the pipe is virtually non-existant. On the other hand, with the tapered outlet or Armtec end section, the adjacent eddies, beginning at the crown of the pipe, are in immediate contact with the grade. In this case, it is necessary to protect the grade up to the level of the maximum tailwater level, or 1.5D, whichever is smaller.

While variations in end treatment often have a significant influence on the performance of an inlet, the jet flow at the outlet is largely unaffected by such treatments. An indication of this effect is given in Table 10. Table 10 gives the length, width and depth of the scour hole in a sand bed, as observed on a model, for 3 outlets. The dimensions are in terms of culvert diameters. The limits of the scour hole are relative to the zero contour, where zero represents the original bed elevation (at the culvert invert). Although the shape of the scour hole is affected to a minor degree, the area and volume is virtually the same for each inlet.

TABLE 10

SCOUR HOLE MEASUREMENTS
Sand bed; $Q/D^{5/2} = 1.5$; tailwater depth = 1D

	Scour Hole Dimensions		
Outlet Type	Length	Width	Depth
Projecting	6.4D	2.5D	0.6D
Tapered	5.5D	2.6D	0.5D
Armtec	5.0D	3.3D	0.6D

The length of riprap protection used downstream from the outlet depends upon the degree of protection desired. If it is intended to prevent all erosion of the discharge channel and adjacent properties, the riprap must continue until the velocity has been reduced to a value which will not scour the channel. This length depends upon the size of the pipe, the outlet velocity, and the magnitude of the permissible velocity for the channel. Naturally, the length is greater for larger pipes or higher outlet velocities, and shorter if the channel has high natural erosion resistance. Riprap specifications which call for a fixed length or fixed number of diameters for the extent of riprap, irrespective of velocities, are obviously lacking in logic.

If the tailwater depth above the invert is equal to the end depth, or if the pipe outlet is submerged, velocity reduction occurs by lateral diffusion of the jet and by boundary friction forces which oppose the motion. The rate at which the jet will expand naturally is very small, about 1 laterally to 10 longitudinally on each side. Hence, to reduce the culvert velocity from 3 m/s to 1 m/s, for example, assuming $d_t = d_e = D$, would require that the flow be expanded to an average width of 2.36 D. The required value for the protected length would be $10(2.36D - D)/2 = 6.8D$.

If the tailwater is less than the end depth, gravity forces become effective and the flow will spread more rapidly. For example, if $d_t = 1/2\ d_e$, the flow will be able to spread at a rate of about 1 laterally to 3 longitudinally. Ironically, this will not necessarily greatly reduce the required extent of protection, because although the flow spreads more rapidly, it must spread to double the width to achieve the same final velocity with only half the tailwater depth. For example, assuming a velocity reduction from 3 m/s to 1 m/s and $d_t = 1/2\ d_e = 1/2$ D, the flow must be expanded to 4.72 D, and the required protected length becomes $3(4.72D - D)/2 = 5.6$ D.

The preceding calculations for protected length are intended to indicate the nature of the problem only. Other factors which may affect the rate of deceleration of jet are riprap size, channel bed configuration, and channel alignment. A catch all empirical rule recommended for design purposes is that a protected length of approximately three pipe diameters should be used for each 1 m/s of velocity reduction required. This is the approximate length that would be required to contain the decelerating jet, and would give a required length of 6D for a velocity reduction from 3 m/s to 1 m/s.

The length of riprap protection for most culvert outlets for highways is much shorter than indicated by the foregoing rule. In these cases it must be expected that some scour may occur at the end of the riprap. Of course, the scour potential of the jet decreases with distance travelled, so the scour is not as severe as it would be if no riprap was used at all. The use of a shorter protected length may be justified where the sole objective is protection of the culvert and not protection of the discharge channel. Models have shown that a protected length of 2D downstream from the outlet is sufficient to prevent underming of the culvert or erosion of the shoulder of the road. The extent of downstream channel scour depends upon the frequency and duration of the larger discharges. If short protected lengths are used these outlets should be inspected after a year of high runoff and additional riprap placed as required. On the other hand, outlets with complete initial protection should be maintenance free.

In theory, the required stone size for riprap could be reduced with increasing distance from the culvert outlet. In practice, the entire riprap job is usually covered by one specification, and therefore the protection may be ultra safe at the downstream end. Large stones placed at the end of the protection are wasted in the sense that the full benefit of their erosion resistance is not utilized. A better overall job results if a higher percentage of the larger stone is placed near the culvert outlet. With proper supervision this can be done quite simply at the job site when the riprap is dumped.

Example 4:

Given a 1600 helical CSP 24 m long with a cylinder inlet, a 1% slope, a tailwater depth at the outlet of 0.9 m and passing a discharge of 6.5 m^3/s:

1) Calculate the upstream head H.
2) Calculate the required riprap size at the outlet.

1. The n value for this pipe, from Table 4, is 0.0208. For full flow, the velocity would be $6.5/(0.785 \times 1.6^2) = 3.23$ m/s, and the slope of the energy line would be $3.23^2 \times 0.0208^2/0.4^{4/3} = 0.0336$. Also, $Q/D^{5/2} = 6.5/1.6/^{5/2} = 2.007$. Since the pipe slope is flatter than the slope of the energy line outlet control may be assumed, and since $Q/D^{5/2} > 2$ full flow may be assumed. Thus

$$h = (K_e + K_d + K_f + K_o)\, V^2/2g$$

applies, with $K_e = 0.3$ (for a cylinder inlet), $K_d = 0.2$, $K_o = 1.0$, and

$$\begin{aligned} K_f &= 2gn^2L/R^{4/3} \\ &= 2g \times 0.0208^2 \times 24/0.4^{4/3} \\ &= 0.69 \end{aligned}$$

so

$$\begin{aligned} h &= (0.3 + 0.2 + 0.69 + 1)\, 3.23^2/2g \\ &= 1.165 \text{ m} \end{aligned}$$

From Figure 13, for $d_t/D = 0.9/1.6 = 0.56$ and $Q/D^{5/2} = 2.007$, $y/D = 0.79$, giving $y = 0.79 \times 1.6 = 1.264$ m. Also

$$\begin{aligned} H &= y + h - SL \\ &= 1.264 + 1.165 - 0.1 \times 24 \\ &= 2.19 \text{ m} \end{aligned}$$

From Figure 6, for inlet control for $Q/D^{5/2} = 2.007$, the required H/D value is 1.35, giving H = 2.16 m. Since the H value for outlet control is greater than this value, outlet control with full flow is confirmed.

2. From Figure 19, for $d_t/D = 0.56$ and $Q/D^{5/2} = 2.007$, $d_e/D = 0.70$, so from Table 7, $A/D^2 = 0.5872$ and $A = 0.5872 \times 1.6^2 = 1.503 m^2$, from which $V = 6.5/1.503 = 4.32$ m/s. The required riprap size given by

$$\begin{aligned} d_m &= 0.019V^2 \\ &= 0.019 \times 4.32^2 \\ &= 0.355 \text{ m} \end{aligned}$$

so 350 mm stone may be specified

F. CULVERT OUTLET EXPANSION

22. Purpose

There are some culvert applications wherein the maximum outlet velocity may be specified, usually in order to simplify problems of possible scour and erosion at the outlet. For example, the velocity may be specified at 2 to 3 m/s, which velocity is considered sufficiently low that scour can be easily resisted at the outlet with riprap, natural till, or perhaps a grassed channel. Under conditions of outlet control with the pipe flowing full, the required pipe size is then determined from the continuity equation as

[34] $$D = \sqrt{4Q/(\pi V)}$$

in which D is the pipe diameter, Q is the maximum (design) discharge, and V is the pipe velocity.

In many cases the pipe size selected on the basis of limiting outlet velocities may be larger than needed to pass the discharge with the available headwater. This raises the natural question as to why a large diameter should be used for the whole pipe if the larger size is only needed at the outlet, and leads to a logical suggestion that an expansion might be used at the outlet end of the pipe. This is a fairly recent concept in culvert design but has economic merit in certain situations.

The method proposed here consists simply of using an abrupt expansion at the outlet end of the culvert. This can be achieved in practice by telescoping one length of larger pipe at the outlet over the preceding pipe. This approach is shown in Figure 20 for a precast concrete pipe. The idea can also be adapted to corrugated pipe culverts. In this way a smaller diameter can be used for most of the pipe, and the outlet velocity specification can still be met. The purpose of using an abrupt expansion as opposed to a gradual expansion is that the former can be accommodated quite simply using existing sizes of pipe, and fabrication of a special tapered transition section is unnecessary.

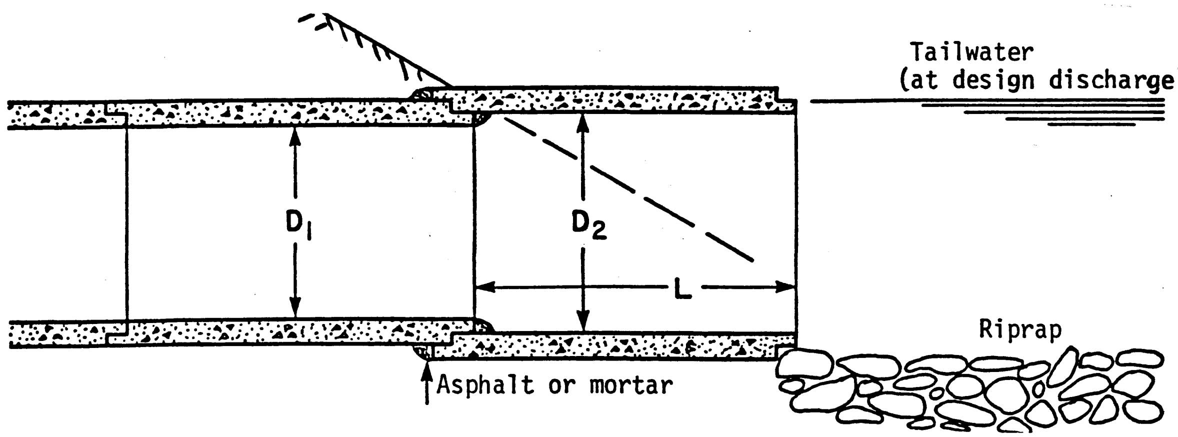

Figure 20. Definition Sketch for Abrupt Expansion

The proposed design applies only to outlet control situations where the tailwater is at or above the crown of the pipe at design flow. Downstream submergence of this type is fairly common in flatland drainage problems, or in multi-pipe installations where smaller diameters are used because of limited headroom. Downstream submergence is particularly prevalent on irrigation projects where, under controlled tailwater situations, the conduit is deliberately set to insure full flow and thereby utilize the full area of the pipe.

An interesting side benefit of the outlet expansion is that the outlet head loss is reduced as well as the outlet velocity. There will be a coresopnding reduction in the headwater level, or, alternatively, it may be possible to take advantage of the reduced loss and use a smaller pipe size.

23. Theory for Abrupt Expansion

In the absence of an expansion the outlet head loss will be

[35] $$h_o = V_1^2/2g$$

With an expansion, in which the velocity at the outlet is reduced to V_2, there will be a loss due to the abrupt expansion in addition to the loss of one velocity head at the outlet. This will be

[36] $$h_o = (V_1 - V_2)^2/2g + V_2^2/2g$$

Equation 36 may be written as

[37] $$h_o = (1 - 2V_2/V_1 + 2V_2^2/V_1^2)V_1^2/2g$$

and by substitution of the continuity equation

[38] $$h_o = (1 - 2A_r + 2A_r^2)V_1^2/2g$$

in which the area ratio $A_r = A_1/A_2$. In spite of the two losses which result from using an abrupt expansion, the head loss according to [38] is smaller than the loss given by [35].

The term in the round brackets in [38] represents the outlet loss coefficient K_o as given by

[39] $$K_o = 1 - 2A_r + 2A_r^2$$

The minimum value for K_o may be found by setting $dK_o/dA_r = 0$, from which $A_r = 0.5$ (corresponding to a diameter ratio $D_r = D_1/D_2 = 0.707$) and $K_o = 0.5$. Hence, the outlet loss may be reduced from one velocity head to half a velocity head with such an expansion, for example a 1000 mm expansion on a 700 mm pipe.

From the point of view of ease of construction and economy it is unlikely that area ratios as small as 0.5 will be used in practice. Fortunately there is still considerable benefit for more practical diameter ratios in the 0.8 to 0.9 range. For example, if an 800 mm expansion is used on a 700 mm pipe (instead of 1000 mm previously cited), then $A_r = (7/8)^2 = 0.766$ and from [39] $K_o = 0.64$. The outlet velocity will be reduced to 0.766 times the pipe velocity, and the riprap size will be reduced to $0.766^2 = 0.587$ of the size required if no expansion is used. Accordingly, the most appropriate expansion in most cases will simply be the next larger size of standard pipe.

24. Performance

It must be evident that to obtain the full benefit of velocity reduction and head loss reduction at the outlet, the length of the added larger diameter pipe must be sufficient to permit complete expansion of the flow. If the pipe is too short, allowing only partial expansion, both the outlet velocity and head loss will increase and [38] will not apply.

Experiments to determine the outlet efficiency have been conducted by Smith, results from which are shown in Figure 21. The description "recovery efficiency", shown on the ordinate of Figure 21, represents the ratio of the head saved (i.e., recovered) by conversion of upstream velocity head to pressure head in the expansion, to the maximum possible recovery, given by $(1 - K_o)\ V_1^2/2g$. Maximum recovery would occur when the expansion is long enough to allow K_o to drop to its minimum value, as given by [39], and the head saved would be $2A_r(1 - A_r)V_1^2/2g$. It is seen from Figure 21 that for a diameter ratio of 0.883 a length not less than 1.5 D_1 must be used to obtain the full benefit of the expansion (i.e. 100% efficiency). A larger expansion naturally requires a greater length, as shown by $L/D_1 = 3$ for $D_r = 0.761$.

It may be preferable in some cases, particularly for corrugated steel pipe, to attach the larger pipe with a common invert rather than concentrically. This would conform to the practice recommended for the cylinder inlet. In this case the entire expansion must take place on one side of the pipe rather than being distributed around the perimeter, and greater lengths will be required to achieve a full expansion. This effect is shown in Figure 22, in which L/D_1 for 100% efficiency is increased from 3 for a concentric expansion to 6 for a common invert expansion.

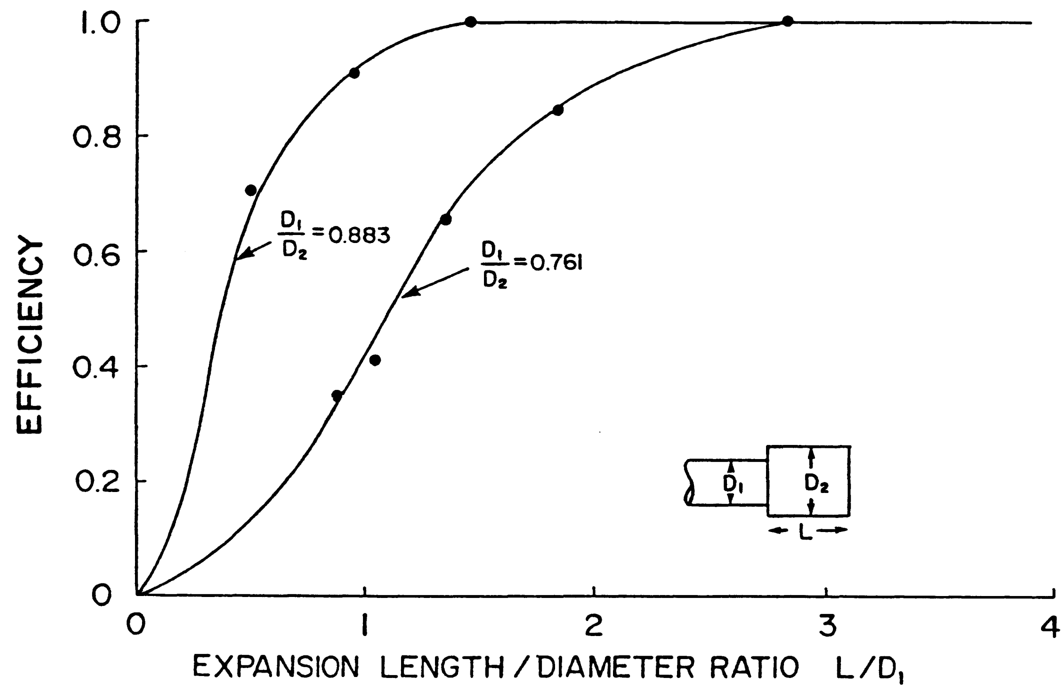

Figure 21. Cylinder Outlet - Effect of L/D_1 on Efficiency

A design chart showing the minimum length required for various diameter ratios is given in Figure 23. Although small diameter ratios would result in low outlet velocities, large L/D_1 values would have to be used and there is little gain in terms of reduced outlet loss. For example, K_o is the same for $D_r = 0.6$ as it is for $D_r = 0.8$. As a compromise between economy and performance it is recommended that diameter ratios smaller than 0.8 not be used in practice.

It is important to note a particular limitation of the outlet expansion, that is, it will not function properly at the design discharge without tailwater submergence. If there is no tailwater the jet from the culvert upstream will simply discharge into the enlarged section without expanding. In order for the jet to expand, the annular space around the jet must be filled with liquid, and this requires a tailwater level at or above the crown of the pipe at the outlet. A reduction in the tailwater level below the crown of the pipe will reduce the efficiency of the expansion. This effect will be even more pronounced for the common invert design than the concentric design.

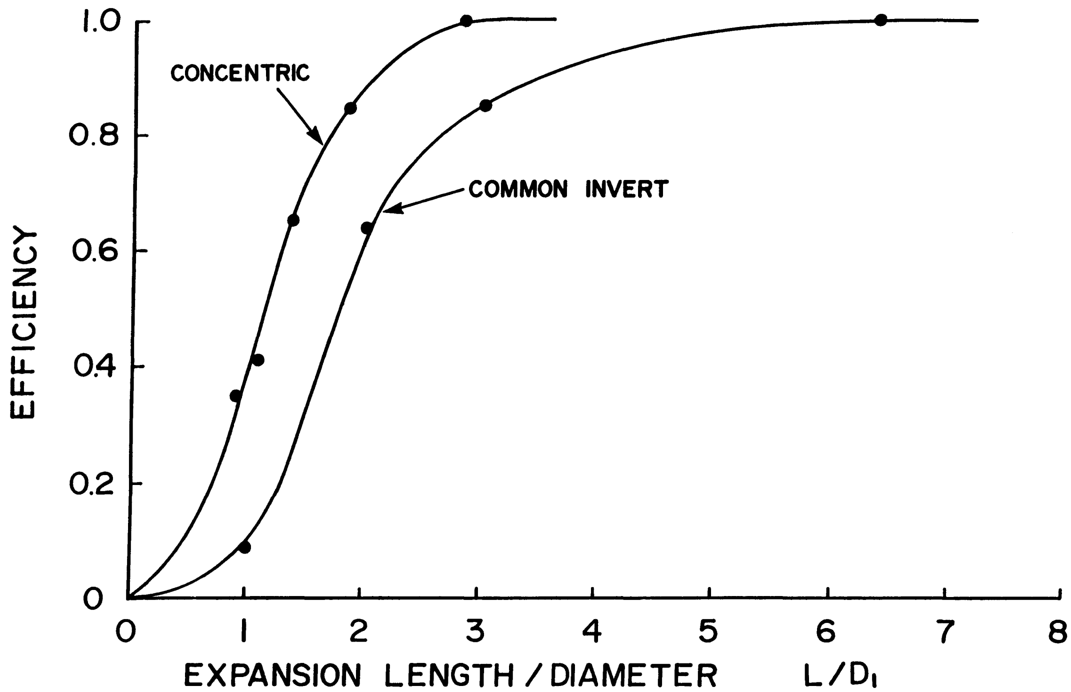

Figure 22. Cylinder Outlet - Effect of Invert Position - D_1/D_2 = 0.761

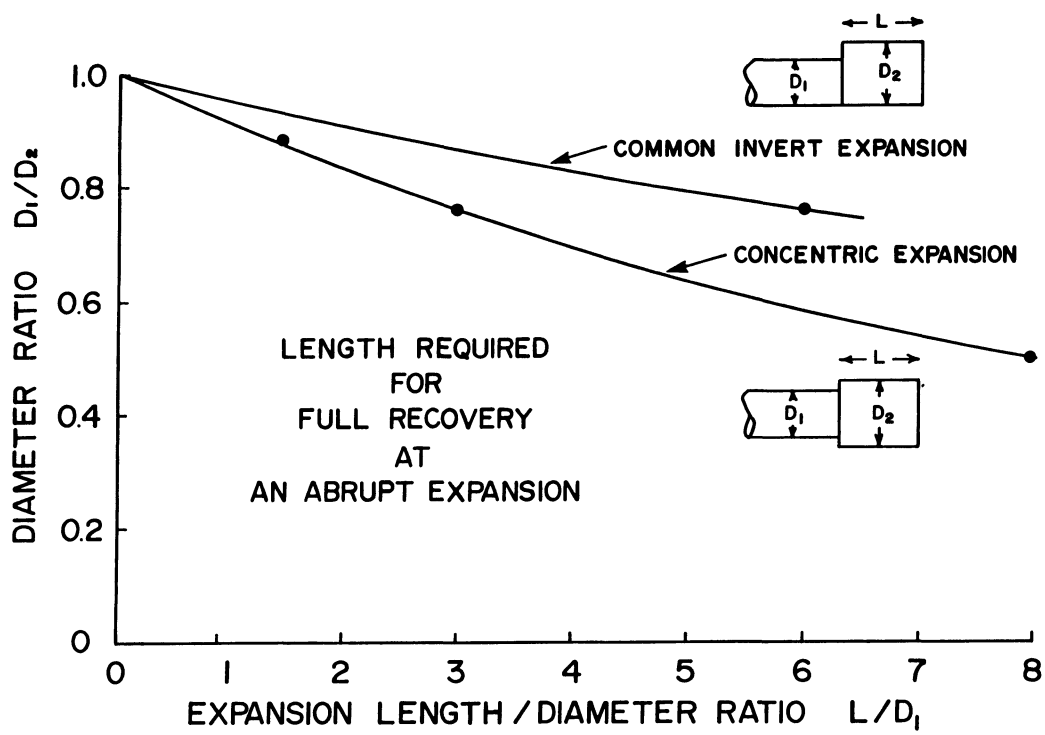

Figure 23. Cylinder Outlet - Design Curves for Length of Expansion

G. FAILURES

25. Washout

A washout may be defined as a gap in the fill caused by the action of flowing water. This type of failure is particularly undesirable when it occurs at the site of the culvert because the damage to the culvert is often greater than the damage to the fill. The latter can be easily replaced, but the culvert may be misaligned, disjointed, bent, or cracked. Although all or part of the culvert may be salvaged, the bedding condition will be unsatisfactory and a complete reinstallation will be required.

A washout at the culvert can occur due to piping or overtopping. Piping failures are the most common failure in culvert installations because frequently the backfill around the culvert is not carefully placed and compacted. There may be void spaces along the barrel which will permit a high rate of seepage flow. This is particularly true of installations made in winter, where use of frozen material for backfill is an open invitation for a piping failure. If installations must be made in the winter, dry granular material should be used around the pipe.

Because a culvert is usually sized for a relatively high frequency discharge, overtopping of the grade may have to be considered as a realistic possibility sometime in the life of the structure if the height of the grade is such that overtopping is imminent at the design head. As discussed in Section 2, this need not result in a washout. Here again, however, good compaction of the fill adjacent to the culvert is desirable. Poor compaction of the backfill means lower density and lower shear strength, and a corresponding increased risk of a washout at the culvert in the event of overtopping. A further countermeasure for an unpaved road is to construct the grade fractionally higher in the immediate vicinity of the culvert. In this way, the flow depth over the grade will be less at the culvert than elsewhere along the road.

26. Inlet Failures

The most serious inlet failure is uplift. This may occur for a projecting lightweight pipe if there is inlet control. Since the pipe only flows partly full for inlet control, the weight of the pipe and the water in it may be less than the buoyant force acting on the submerged pipe. The weight of the fill will prevent uplift of the entire pipe, but if the side slopes of the grade are flat, a considerable length of the pipe may be exposed at each end. It is possible to develop a large bending moment in the pipe which may result in bending the inlet of the pipe upward. This type of failure is most common in CSP with a projecting inlet. It can be avoided either by making sure that the crown of the pipe does not project more than 2D upstream from the grade, or by improving the inlet so that the depth of water flowing in the pipe is increased.

Another type of failure possible with CSP is collapse of the inlet, which may result on larger size pipe if the end of the pipe has been cut on the bevel so that it matches the side slopes of the grade (i.e., tapered inlet). Although tests have shown that this will not improve the discharge capacity of the culvert, this practice has persisted in some jurisdictions. Unfortunately, a sloping cut will reduce the structural strength of the pipe at the inlet because thin walled pipe depends upon a continuous ring for its strength. When operating with inlet control, the pressure on the sides and bottom of the pipe is greater on the outside than on the inside. This difference in pressure may result in inward collapse of the tapered section. A failure such as this will increase the risk of overtopping because of the great reduction in the capacity of the inlet. This type of failure may be avoided quite simply by using a square end, or by reinforcing the tapered end to withstand external hydrostatic pressure. A further disadvantage of the tapered inlet is that more riprap is required around the inlet, and it cannot be adapted for use with the cylinder inlet to reduce inlet losses.

27. Outlet Failure

The principal type of outlet failure is undermining of the pipe by the scour due to inadequate riprap. Unlike the case for the inlet, the zone of influence for outlet velocity is extensive, and, in addition, the velocity at the outlet may be higher than at the inlet. This problem has been dealt with in Part E, Erosion Control.

Unchecked scour at the outlet can result in the pipe being cantilevered out of the fill over the scour hole. Corrugated metal pipe, because of the large strength to deadweight ratio, will often maintain its structural integrity with a cantilever length of 2 or 3 diameters. In the case of a concrete pipe, because of the large deadweight and shorter jointed sections, it is possible for the end section to separate and drop into the scour hole. Evidently, corrugated metal pipes are more vulnerable at the inlet and concrete pipes more vulnerable at the outlet. In either case, the failures can be prevented with proper attention given to design and construction procedures.

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PROBLEMS

1. Froude criteria for similarity are important when gravity forces predominate. This applies at the inlet and outlet of a culvert where free surface flow exists. Show that the pipe parameter $Q/D^{5/2}$ is basically a form of the Froude number.

2. Calculate the discharge for a 1200 mm CSP with a projecting inlet when the headwater is 1.40 m above the invert. Assume inlet control. What size of tongue and groove RCP could be used for the same discharge at approximately the same head?

3. Solve Problem 2 using 4.0 m for the headwater depth instead of 1.40 m.

4. Solve Problem 2 if the tailwater depth is 1.2 m above the invert of the pipe at the outlet, given that the pipe length is 30 m and slope is 1%. Assume annular corrugations for the steel pipe.

5. Determine the outlet velocity and riprap specifications for both the CSP and the RCP in Problem 2, assuming the tailwater depth is 0.60 m above the invert at the outlet and the maximum non-scouring velocity for the ditch is 1.5 m/s.

6. Determine the riprap specifications for both the CSP and RCP in Problem 4, assuming the maximum non-scouring velocity for the ditch is 1 m/s.

7. A 30 m long, 1200 mm diameter CSP (n = 0.024) has a projecting inlet. The headwater depth is 2.0 m above the invert at inlet, the slope is S, and the tailwater depth is d_t above the invert at outlet. Calculate the discharge if:

   (a) $S = 0$ and $d_t = 0.90$ m

   (b) $S = 3\%$ and $d_t = 0.90$ m

   (c) $S = 3\%$ and $d_t = 1.50$ m

8. The 1200 mm annular CSP of problem 4 (projecting inlet, L = 30 m, S = 1%, H = 1.4 m, $d_t = 1.2$ m, $Q = 1.84\ m^3/s$) is replaced by a 1000 mm helical CSP with a 1250 mm cylinder inlet and a 1200 mm outlet expansion. How will the discharge capacity of the new smaller pipe compare with the original?

9. Multi-plate corrugated plate is frequently used for culverts which must carry large external loads. The average corrugation of multi-plate is 50.8 mm deep and 152.4 mm wave length. Calculate Manning's n for an 1800 mm diameter culvert by the Morris method for strip roughness.

10. It is calculated that a 1000 mm concrete pipe with a submerged outlet will discharge 2.5 m^3/s with a differential head of 0.91 m and will have an outlet velocity of 3.18 m/s. However, because of upstream flooding and a shortage of riprap in the area it is necessary to keep the differential head under 0.70 m and the outlet velocity under 2.5 m/s. Could this objective be met by using one 2 m length of 1200 mm diameter pipe at the outlet, or would it be necessary to use a larger size for the whole pipe?

CHAPTER XI

FLOW MEASUREMENT

A. GENERAL

1. Importance

Accurate and convenient flow measurement of water in open channels is important in many fields of engineering. It is particularly important in irrigation, drainage, and water supply, and to a lesser extent in projects for navigation, flood control and power.

Measurement is necessary on irrigation projects in order to permit economic design and efficient operation. Economic design requires that the sizes of canals and structures be kept to a minimum. This is possible only if the system is well regulated and discharges are not allowed to exceed design values. Since it is impossible to estimate rates of flow accurately by eye, measurement is essential. Measurement also provides the only fair and reasonable basis for making charges for water. Finally, water budget studies to determine seepage and evaporation losses depend on accurate measurement of surface water. This type of information is valuable for purposes of extending a system or making future designs.

Similar arguments may be advanced regarding the importance of measurement on water supply and distribution systems. Supply may involve flow in rivers, canals, or other channels. Distribution systems usually are pressure pipe systems, and as such are outside the scope covered by the present chapter. Measurement is still important, however, as proven by the much higher per capita consumption of water in unmetered systems. Associated with the distribution system is the collection system of sanitary and storm sewers. These involve non-pressure flow and therefore may be classified as open channels. Some of the methods treated in this chapter may be applied to these cases.

2. Methods

Direct discharge measurements by gravimetric or volumetric means, although important in laboratory work, are seldom applicable to field situations. Use of a current meter and velocity-area integration is often used, but can be time consuming and costly. Accordingly, indirect methods should be used wherever possible. These methods involve principles of fluid dynamics, in which the continuity equation, energy equation, and occasionally the momentum equation, can be applied and reduced to a single discharge equation. In many cases, one simple reading is all that is required to calculate the discharge by this equation.

It is desirable to take full advantage of the natural weir and orifice characteristics of all structures on a system. At drop structures, checks, turnouts, drop manholes, sewer outfalls and channel contractions principles of fluid dynamics may be applied to solve the problem of flow measurement. In this way flow measurement may be possible without a special measurement structure.

There are an unlimited number of possibilities for structure shapes or devices to which principles of fluid dynamics can be applied for purposes of flow measurement. A few of these are discussed in this chapter. There has been a tendency in the past for designers to restrict themselves to standard stereotyped methods. This is not necessary--more imagination is called for. The intention of this chapter is to indicate a few possibilities to the reader.

The standard rectangular and V-notch weir are discussed in Part B, and are brought in as introductory subjects. These weirs often have severe limitations in field applications. Part C covers weirs of zero height, in which the weir is set with the crest at the bed level of the upstream channel. Part D deals with brink depth methods. These may often be applied to field situations as they exist. Part E deals with critical depth methods. This method usually requires construction of a special structure.

B. STANDARD THIN PLATE WEIR

3. Classical Rectangular Weir

The classical rectangular weir consists of a smooth vertical plate with a rectangular shaped opening at the top. The edges of the opening are sharp to insure a line of contact with the flow. The crest is horizontal and located a height P above the bed of the approach channel. The width of the channel is B and the width of the weir opening is b. The head of water above the weir crest is h, measured at a point at least 3h upstream from the weir. The total head on the weir is H, which includes the velocity head in the approach channel, and hence h = H if the approach velocity is negligible.

Referring to Figure 1, an equation for the ideal discharge for this weir can be derived by neglecting friction, contraction, surface tension and velocity head, as follows:

$$dQ = V\,dA$$

$$dQ = \sqrt{2gy}\,b\,dy$$

$$Q = \sqrt{2g}\,b \int_0^h \sqrt{y}\,dy$$

$$Q = 2/3\,\sqrt{2g}\,b\,h^{3/2} \qquad [1]$$

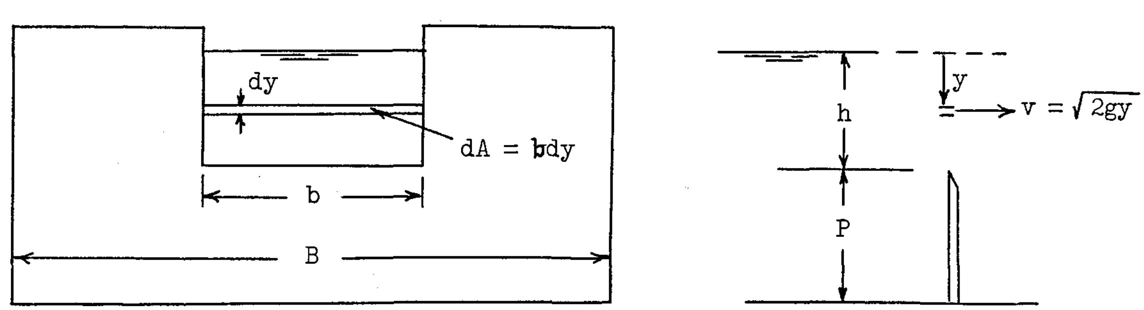

Figure 1. Rectangular Weir

An experimental coefficient of discharge C_d, which is non-dimensional, must be introduced into [1] to calculate the actual discharge, resulting in

$$[2] \qquad Q = C_d \, 2/3 \sqrt{2g} \, b \, h^{3/2}$$

The terms $C_d \, 2/3 \sqrt{2g}$ may be combined into a single coefficient C, called the weir coefficient, from which

$$[3] \qquad Q = C \, b \, h^{3/2}$$

When the equation is used in this form, the coefficient C must include the effect of all factors which affect the discharge. These factors include the surface tension and viscosity of the liquid, and velocity of approach and contraction of the flow. The last two factors are usually determined by the geometry of the weir (P/h, b/h, B/h, etc.).

If the liquid is water with $h \geq 0.1$ m, $P/h \geq 6$, and there are no side contractions ($B/b = 1$), then C is a constant equal to 1.837. This value was first determined by Francis in 1850. The same coefficient may be used for $P/h < 6$ if the head term is taken as the total head, then

$$[4] \qquad Q = 1.837 \, b \, H^{3/2}$$

In this case, of course, it is not possible to measure H directly. It must be calculated by adding the approach velocity head to h, which can be measured. Trial and error may be required.

A less restrictive case, good for any weir height and head provided that $B/b = 1$, is given by Rehbock as

$$[5] \qquad C = 2/3 \sqrt{2g} \, (0.605 + 0.001/h + 0.08 \, h/P)$$

In this equation the first term in brackets accounts for friction and contraction, the second term cohesion and adhesion, which is a low head effect, and the last term accounts for approach velocity. The value of C from [5] may be used in [3] avoiding the need for trial and error.

4. Classical V-notch Weir

The particular advantage of the triangular or V-notch weir is that it is good for accurate measurement of small flows, and at the same time is capable of passing very large flows with a modest increase in head. The ideal discharge equation may be derived using an analysis similar to that used for the rectangular weir. Referring to Figure 2, and neglecting approach velocities, then

$$dQ = \sqrt{2gy} \, (h - y) \, 2 \tan(\theta/2) \, dy$$

$$Q = 2 \tan(\theta/2) \sqrt{2g} \, b \int_0^h (h - y) \sqrt{y} \, dy$$

$$[6] \qquad Q = (8/15) \sqrt{2g} \tan(\theta/2) \, h^{5/2}$$

Again, by inserting a coefficient of discharge to account for friction and contraction,the actual discharge equation becomes

[7] $$Q = C_d\,(8/15)\,\sqrt{2g}\,\tan(\theta/2)\,h^{5/2}$$

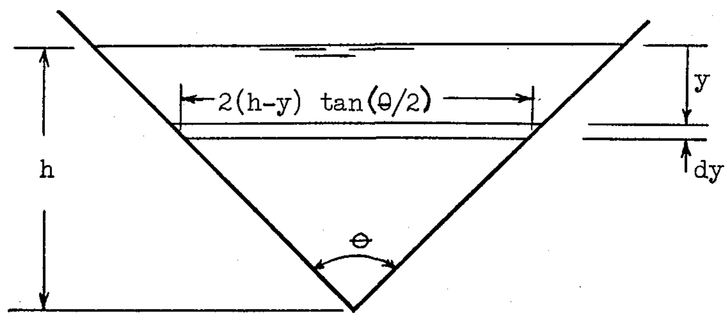

Figure 2. V-notch Weir

The relationship between C_d and h for various notch angles, as determined by Lenz, is shown in Figure 3. For the 90 degree weir and heads greater than 0.2 m, [7] may be reduced to

[8] $$Q = 1.37\,h^{2.5}$$

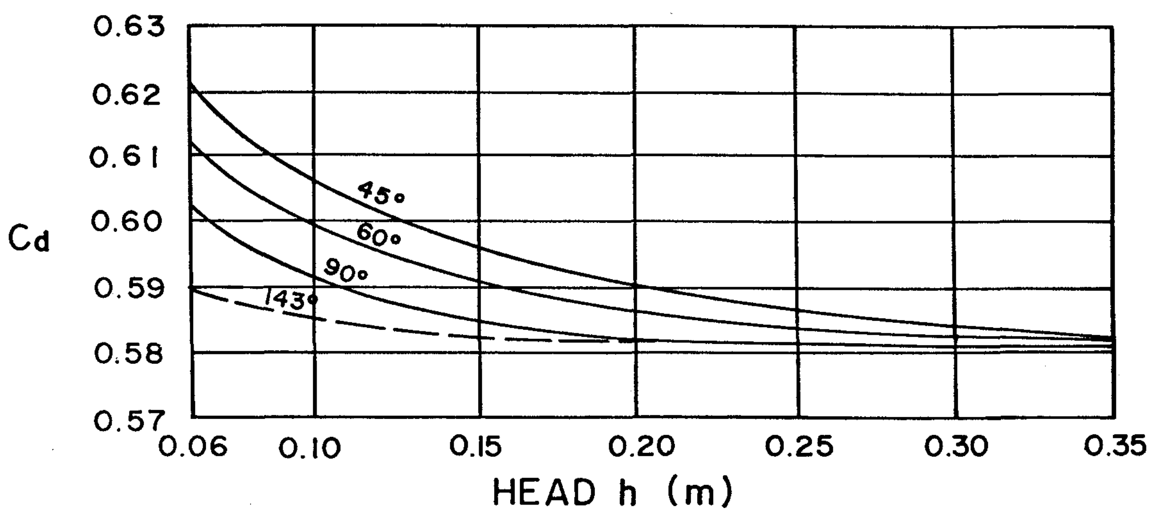

Figure 3. Coefficient of Discharge for Triangular Weirs

From the trend of the curves in Figure 3 it may be speculated that for larger angles the coefficient C_d is much closer to 0.581 for the whole range of heads, as shown for example by the dash line for $\theta = 143°$. This angle corresponds to weir side slopes of 3 horizontal to 1 vertical, which is expected to be more useful on large capacity field structures than the smaller angles. For $\theta = 143^{\circ}$, the discharge equation would become

[9] $$Q = 4.11\ h^{5/2}$$

The C_d coefficients shown in Figure 3 are for negligible velocity of approach. If the velocity of approach is significant, [7], [8] and [9] still can be used if h is replaced by H. However the values for C_d will also be affected and Figure 3 will not apply exactly.

C. WEIRS OF ZERO HEIGHT

5. Purpose

The standard weirs discussed in Part B must be placed with the weir crest above the floor of the approach channel. This may be a particular disadvantage in certain field applications, since the weir acts as a barrier to any material which moves along the bed of the channel. This problem can be avoided if the weir height is eliminated (i.e., P = 0). In this case there is really no weir at all, but rather a lateral contraction.

In this section the results of tests on a rectangular and triangular lateral contraction are reported. These have been variously referred to as constrictions, or notches, but in essence are really rectangular and triangular weirs with zero weir height. In the case of the triangular weir, the apex of the V-notch is placed at the bed elevation of the upstream channel.

6. Rectangular Contraction

Despite the absence of a weir height, [3] can also be applied to the rectangular lateral contraction provided there are no submergence effects from downstream and if an appropriate correction for the weir coefficient is made. The value h represents the upstream water depth measured at a point outside of the range of local drawdown. Two cases are possible. There may be a drop in the channel bed at the position of the contraction, in which case the discharge through the opening will have a free fall; or the channel bed may continue through the contraction at the same elevation, in which case the discharging jet will be supported. Despite this support, a unique relationship will still exist between the upstream head and the discharge, provided the flow is still supercritical downstream from the contraction. This condition will be satisfied as long as the downstream tailwater depth does not exceed half the upstream head.

Figure 4 shows results of experimental work by Hill. The figure shows the effect on C of various contraction ratios b/B. The value B is the width of the channel; b is the width of the contraction. In the tests the channel was rectangular, however a reasonable estimate for the discharge can be made for non-rectangular channels if B is taken as the average width of the approach flow.

The solid curves in Figure 4 are for the case where the discharging jet is supported by the downstream channel bed. The dashed curves correspond to the case where there is a free fall downstream from the contraction. It is reasonable that when h/b is very large, the presence or absence of downstream support should not be particularly significant. This observation is borne out by the fact that the curves for the two cases converge at large h/b values. Support is more significant when h/b is small, as is shown in particular by the marked reduction in C for b/B = 0.75 when the jet is supported downstream.

The lateral contraction of the jet increases as b/B → 0, and this fact is largely responsible for the decrease in C for the smaller b/B ratios. Theory indicates, however, that the contraction is almost fully developed for b/B = 0.25, and the coefficient will be only 2% lower for b/B = 0. This case was not actually tested and therefore a curve for b/B = 0 is not shown on Figure 4, but a 2% reduction will give a satisfactory correction for interpolation purposes.

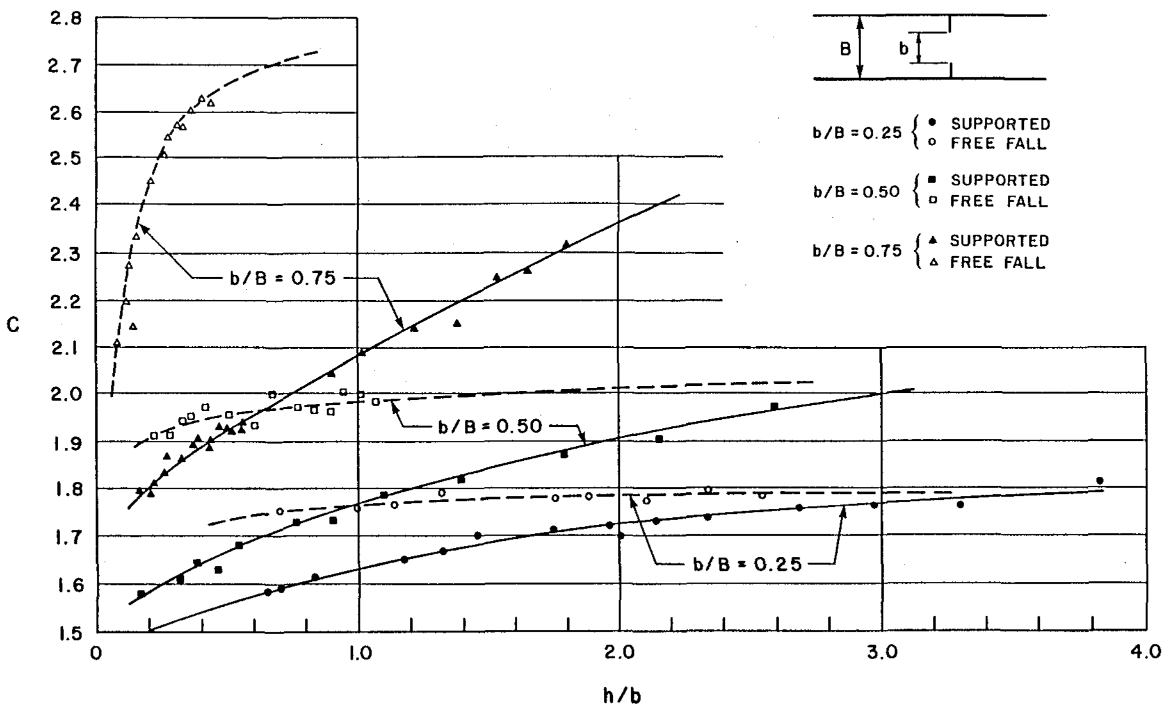

Figure 4. C versus h/b for a Rectangular Contraction

7. <u>Triangular Contraction</u>

Equation [7] for the triangular weir may also be applied to the triangular contraction with the apex of the triangle placed on the bed of the channel. Naturally the coefficient of discharge is affected. Figures 5 and 6 show the values of C_d versus h for contractions with $\theta = 90^o$ and $\theta = 60^o$, as reported by Liang. On each figure a family of curves is drawn corresponding to different approach channel widths B.

Both figures show that when the head is low (small h/B values) the coefficient C_d is the same for all widths, and in fact is the same as for the standard triangular weir. The reason for this is that the velocity head is negligible and the contraction is fully developed. As the head increases the approach velocity head becomes significant, naturally more so for the narrow approach channel than the wide channel, and the curves break away from the parent curve. The subsequent increase in coefficient can be readily explained by the fact that the contraction of the nappe is reduced as h/B increases, and also the effect of velocity head is reflected in the coefficient C_d because[7] is in terms of h rather than H.

It should be pointed out that for the triangular contraction it does not matter whether the channel bed continues downstream at the same elevation or not. In other words downstream support does not significantly affect the discharge from the V-notch, provided of course that there is no submergence by tailwater.

While Figures 5 and 6 were drawn directly from the experimental data for particular sizes of weir, it is possible to use the curves to predict coefficients for larger weirs by applying similarity criteria. For example, if h = 1.20 m and B = 4.8 m for a prototye with $\theta = 90^o$, this weir has the same h/B ratio as the test case for h = 0.23 m and B = 0.915 m, for which $C_d = 0.613$. The prototype discharge would be 2.284 m^3/s. The reader should be cautioned, however, that this scaling up procedure will be accurate only when applied to the experimental data for which $h \geq 0.15$ m. At lower heads scale effects due to cohesion, adhesion, and viscosity preclude exact similarity for a triangular weir.

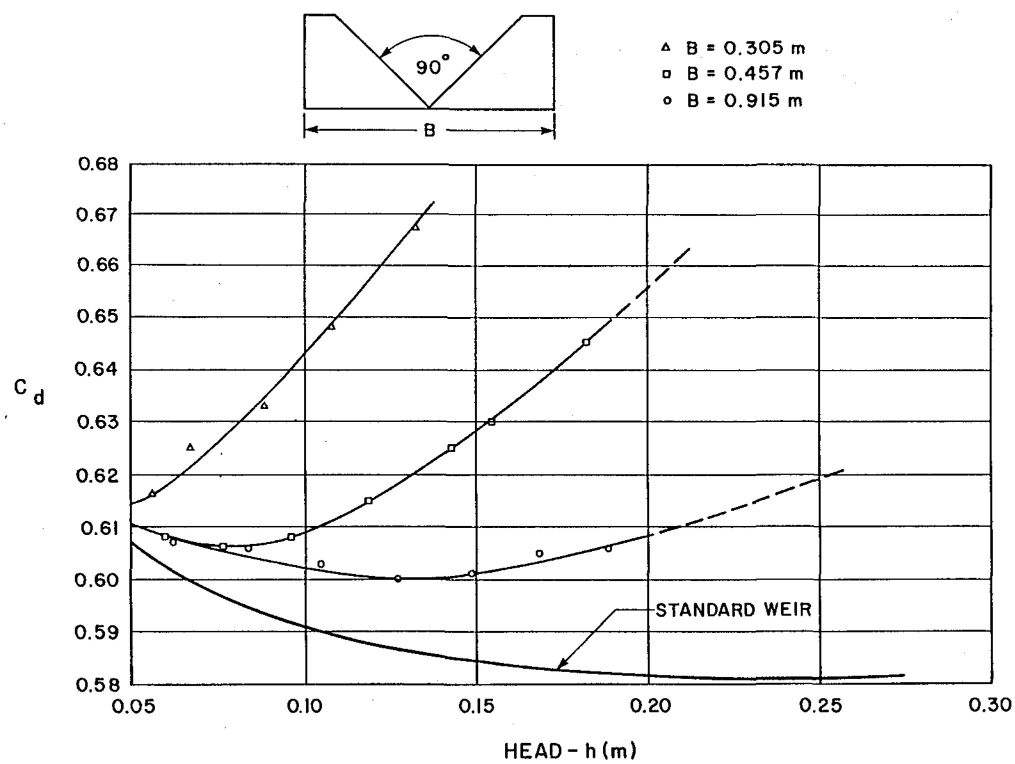

Figure 5. C_d *vs h for a 90° V-notch Contraction*

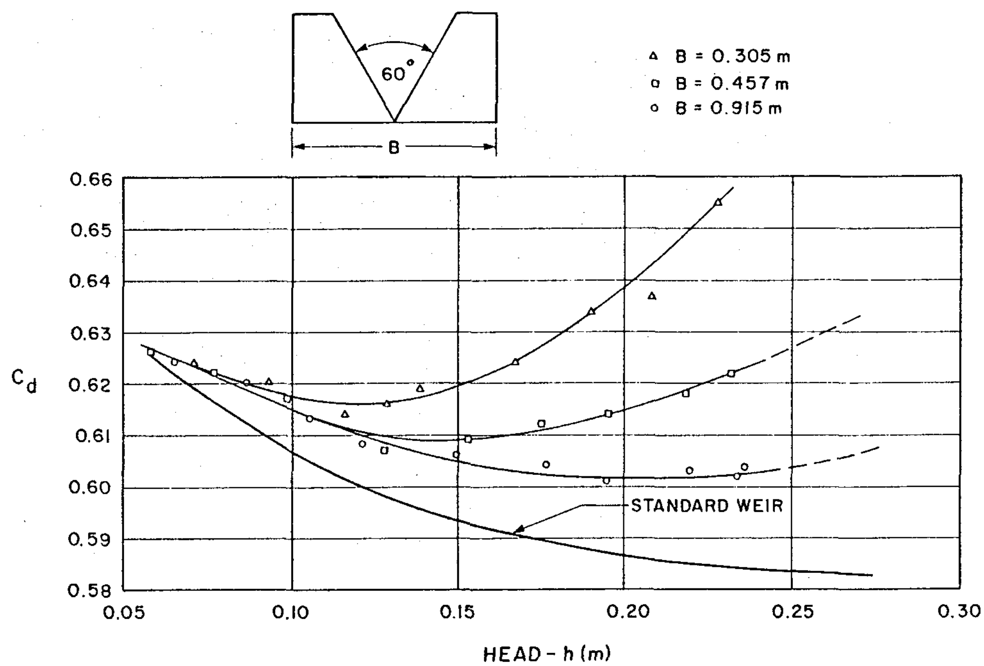

Figure 6. C_d vs h for a 60° V-notch Contraction

D. BRINK DEPTH METHODS

8. General

In Section 6 the case of a rectangular weir with P = 0 was discussed. If in addition B/b = 1.0, then the lateral contraction is also eliminated, and there will be no weir at all. Given a free drop at the end of the channel, the flow will simply pass over as a free jet. However, there will be a unique relationship between the end depth, or brink depth, and the discharge. Such a relationship will exist regardless of the shape of the channel, although of course it will be different for different shapes. In the following sections the relationship between brink depth and discharge for rectangular and circular sections is discussed.

9. Rectangular Section

Consider channel flow approaching a drop off, as shown in Figure 7. If the channel is level, or has a mild slope so that the flow is subcritical upstream, then the flow will pass through critical depth D_c a short distance upstream from the brink. The momentum equation between the critical depth section and the brink may be written as:

[10] $P_c - P_b - F_f = q\rho\,(\beta_b V_b - \beta_c V_c)$

in which P_c is the hydrostatic pressure force at critical depth, P_b denotes the pressure force at the brink, F_f represents the channel boundary friction force, q is the unit discharge, ρ is the density, V_b is the average velocity at the brink, V_c is the critical velocity, and β_b and β_c are the momentum correction factors.

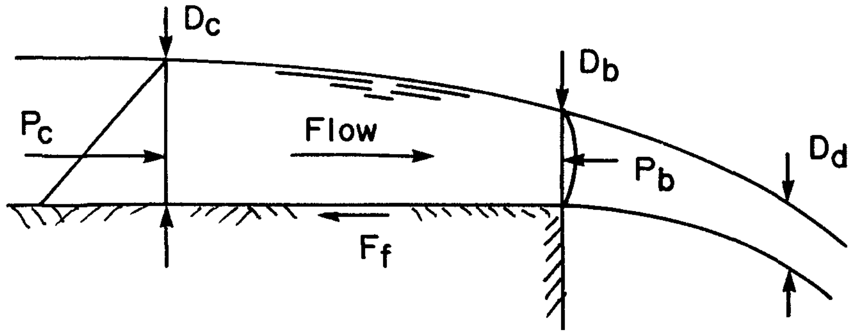

Figure 7. Definition Sketch For Critical Flow and Brink Depth

The factors β_b and β_c are near unity and can be taken as such with very small error. Furthermore, because the problem is one of rapidly varied, non-uniform flow, wherein the channel length being considered is short, the force F_f may be neglected. This force is negligibly small in comparison to P_c for a smooth boundary material. Neglect of these factors is common procedure in applying the momentum equation. This is done, for example, in the analysis of a hydraulic jump, yet the resulting error in computed sequent depth is usually of no consequence.

The pressure at the brink is not hydrostatically distributed, being atmospheric at the surface and at the bottom, and having a small but definite value in the zone between, as illustrated in Figure 7. The development of this pressure is caused by convergence of streamlines accompanying the flow curvature at the brink. The resulting pressure force P_b is significant and cannot be neglected. As P_b is not subject to easy evaluation, [10] does not lend itself to a simple solution for the depth at the brink.

Considering now a vertical section through the nappe a short distance downstream from the brink where the stream lines are essentially parallel, it is evident that the internal pressure must be essentially atmospheric. In this case, the momentum equation, written between the critical depth section and the downstream section reduces to

[11] $P_c = q\rho\,(V_d - V_c)$

or

[12] $\gamma D_c^2/2 = q^2\rho\,(1/D_d - 1/D_c)$

in which γ is the specific weight of water, V_d denotes the horizontal component of nappe velocity, and D_d is the vertical depth through the nappe, at the downstream section.

Given that $q^2 = gD_c^3$ for critical flow, [12] can be solved exactly for the depth D_d, giving $D_d/D_c = 0.667$. The actual depth at the brink, where a finite internal pressure exists in the nappe, must be greater than the depth D_d. Hence, the limits for the brink depth are established as being between the values D_c and D_d, and presumably much closer to D_d than D_c because the force P_b is much closer to zero than it is to P_c.

Actual experiments by Rouse have proved the validity of the preceding theory. By experiment, the ratio $D_d/D_c = 0.667$ as given by theory, and the ratio $D_b/D_c = 0.715$. It may be shown that [12] would yield the ratio $D_b/D_c = 0.715$ if a value for $P_b = 0.2\ P_c$ was used. In any case, the unit discharge may be determined from a single measurement of the depth at the brink by the relationship

[13] $q = \sqrt{g\,(D_b/0.715)^3}$

or

[14] $q = 5.18\ D_b^{3/2}$

10. Circular Section

Brink depth methods may be applied to flow conditions at the free outlet of a circular pipe. The theory for the circular section is somewhat more complex. The gross section of flow must be considered, rather than a unit width, and the geometric properties of the gross section vary with the depth. It might be expected that the D_b/D_c ratio would not be a constant value, as in the case of the rectangular section, but would vary with the discharge.

For lack of exact knowledge of the values for β_c, β_b, and F_f, the assumptions are made that $\beta_c = \beta_b = 1$, and F_f is negligible, as in the case of the rectangular section. The force P_b cannot be evaluated by analytical means; however, it has been demonstrated that the assumption of zero internal pressure in the jet will lead to an exact solution for the flow area A_d at a vertical section beyond the brink, and furthermore, that this area is only slightly less than the area A_b at the brink itself. The solution for the area A_d will give, therefore, a close limit for the area A_b.

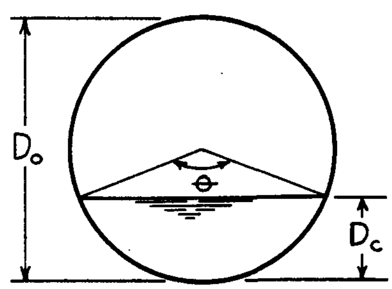

Figure 8. Definition Sketch for Circular Section

As applied to the circular section, the momentum equation would become

[15] $P_c = Q^2 \rho\,(1/A_d - 1/A_c)$

in which Q is the total discharge.

With θ as shown in Figure 8 and expressed in radians it may be shown that

[16] $A_c = (\theta - \sin\theta)\,D_o^2/8$

and

[17] $P_c = \gamma D_o^3 [(4/3) \sin^3 (\theta/2) - \theta \cos (\theta/2) + \sin \theta \cos (\theta/2)]$

[18] $Q = g^{1/2} (\theta - \sin \theta)^{3/2} D_o^{5/2}/[512^{1/2} (\sin \theta/2)^{1/2}]$

Equation [17] simply represents the product of the area of the segment, given by [16], times the pressure at its center of gravity, and [18] satisfies the condition that the specific energy is a minimum for the given discharge, which therefore must be the critical discharge.

Simultaneous solution of [15] to [18] will yield the area A_d in terms of Q and D_o. By assuming $A_b = A_d$, an approximation for Q could be calculated from the measurement of the depth at the brink. This approximation will be on the high side because of neglect of the pressure force at the brink. In this respect [15] to [18] are primarily of academic interest. The actual relationship must be determined by experiment, as has been done by Smith and others.

Figure 9 shows an experimental plot of $Q/D_o^{5/2}$ versus D_b/D_o. The equation of this curve within the range $0 < Q/D_o^{5/2} < 1.6$ is

[19] $Q/D_o^{5/2} = 4.58 (D_b/D_o)^{1.84}$

or

[20] $Q = 4.58 D_b^{1.84} D_o^{0.66}$

The discharge may be solved directly either from [20] or Figure 9. It is significant that the sum of the exponents for the length terms on the right hand side of [20] is 5/2. This is a requirement of the similitude ratio for discharge according to the Froude criteria for similarity. The brink depth method is not reliable beyond the upper limit ($Q/D_o/^{5/2} = 1.6$) because critical depth approaches the pipe diameter and the pipe tends to flow full closer to the outlet. The test data scatter widely, as shown between the dashed lines on Figure 9.

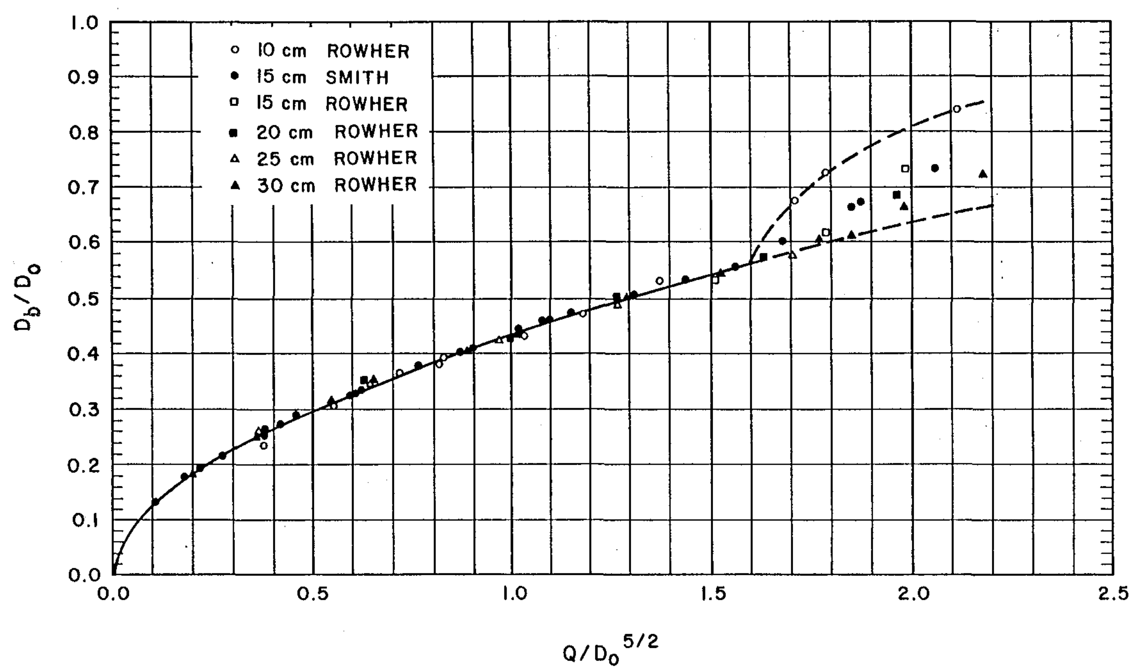

Figure 9. Brink Depth vs Discharge for a Circular Section

E. CRITICAL DEPTH METHODS

11. General

One of the disadvantages of the methods discussed in the previous sections is that the discharge equations could be applied only if there was no tailwater interference. In order to satisfy this condition a free drop is usually required. Should the available drop in water surface elevation across the structure be insufficient, tailwater may partially submerge the discharging jet. It is still possible to obtain a unique relationship between discharge and the upstream head alone by constructing a special type of measuring structure, known as a critical depth meter. These structures can be submerged by tailwater up to critical depth without affecting the upstream head.

One of the earliest types of critical depth meter was the Parshall flume. The standard form for the Parshall flume consists of a short rising floor, an inlet section which has a level floor and sidewalls converging at 1 in 5, a throat section with parallel sidewalls and floor dropping at 1 in 2.67, and an outlet section with rising floor and sidewalls diverging at 1 in 6. It should be noted that the form selected for the Parshall flume was largely arbitrary. A number of other shapes could have been used to the same end. The popularity enjoyed by the Parshall flume is primarily due to the fact that it has been extensively rated, and discharge tables are available for a range of sizes.

An alternative to the Parshall flume, which may also be classified as a critical depth meter, is the streamlined broad crested weir. Again it is possible to have various shapes in cross section for this weir. In the following sections the rectangular and triangular broad crested weirs are discussed.

12. Rectangular Broad Crested Weir

A definition sketch is shown in Figure 10. A rational approach to the discharge equation is made possible by the fact that the control section is a constant section with level floor and parallel sidewalls. Based on the assumption of critical flow on the crest, whereby the depth is 2/3 of the total head H and the velocity head is 1/3 H, the theoretical discharge equation may readily be computed as

$$[21] \quad q = 2/3\, H \sqrt{2g(1/3)\, H}$$

or

$$[22] \quad Q = 1.705\, b\, H^{3/2}$$

Equation [22] is seen to take the same form as the classical equation for a rectangular weir. The coefficient 1.705 is lower than for a sharp crested weir as given in [4].

The condition of critical flow is independent of the height P or width b of weir. It is required only that the flow should arrive at the control section in a two-dimensional state, and that the crest length L be long enough to produce a short reach of parallel flow. This requirement may be met by the use of a semi-streamlined approach to the control section. It is evident that the weir coefficient will be considerably smaller than 1.705 if the upstream corner of the broad crest is sharp or right angled, since separation and eddy loss would occur.

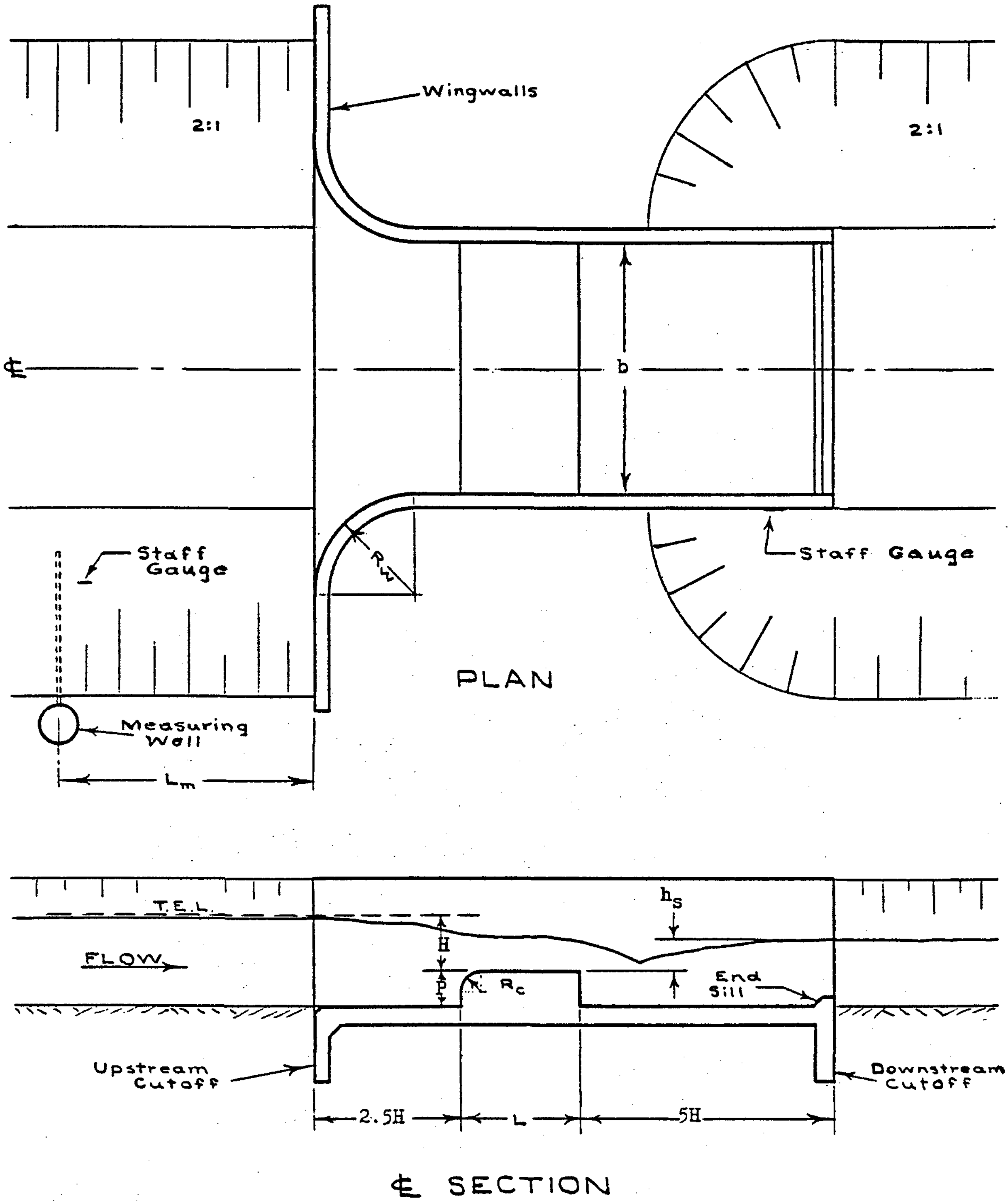

Figure 10. Definition Sketch for a Rectangular Broad Crested Weir

It is reasonable to expect that the actual coefficient will be affected by the length of the weir L. The broad crest is like a short reach of open channel. Allowance for friction on the crest and development of the boundary layer indicates that the coefficient would be reduced for greater lengths of crest. In other words, the coefficient should increase with an increase in H/L.

A plot of the actual variation of the discharge coefficient C with H/L is shown on Figure 11. It is seen that for small values of H/L, C is substantially less than the theoretical value of 1.705. If H/L exceeds 0.6, the coefficient actually becomes greater than the theoretical. The weir is then so short, relative to the head, that parallel flow is not established. Instead there is a concave down curvature of the flow surface, and the pressure in the jet on the weir becomes less than hydrostatic. This reduced pressure is reflected by an increase in the coefficient. For design purposes the maximum value of H/L should be limited to 0.5, for which C = 1.687, otherwise non-parallel flow will occur and the coefficient will be affected by tailwater submergence at a lower value of the tailwater head h_s.

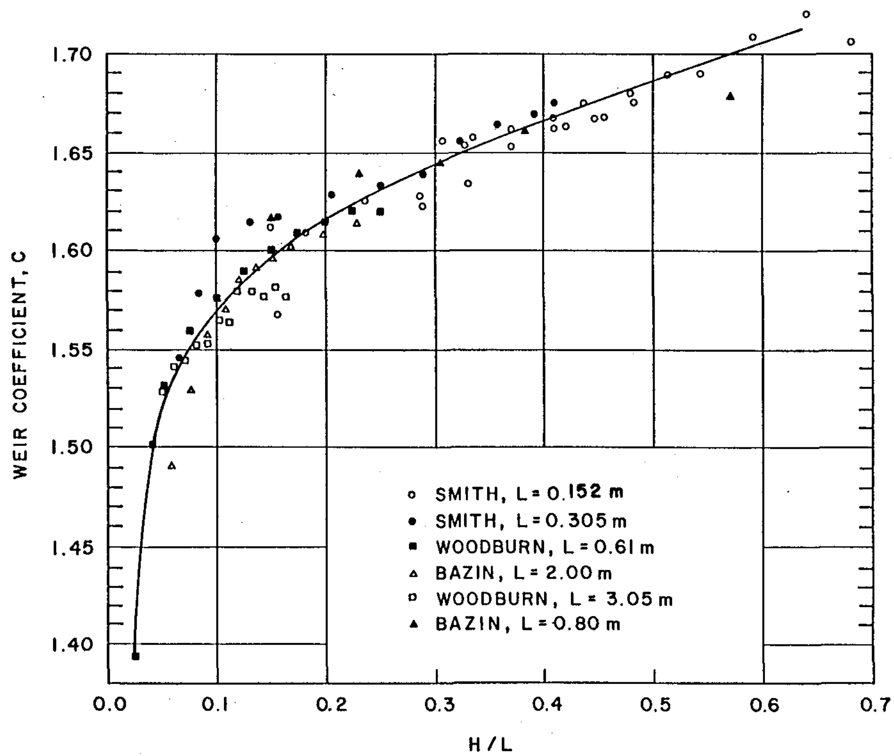

Figure 11. Weir Coefficient for Rectangular Broad Crested Weir

The weir height P is known to affect the discharge coefficient for a sharp crested or ogee weir. This results from the fact that the form of the nappe and discharge coefficient is affected by the velocity of approach, which in turn depends upon the weir height P. In the case of the broad crested weir the form of the flow over the broad crest is independent of the velocity of approach, and therefore the coefficient should not be affected. This has been borne out by tests.

Theory indicates that the radius of the curve R_c should have no effect on the coefficient, provided the radius is large enough to avoid flow separation or distortion of the flow pattern at the inlet. This can be achieved if $R_c = 0.4\ H_{max}$. This has also been verified by test.

The curved upstream wingwalls R_w serve to streamline the approach to the weir in plan just as the curve R_c does in section. An entrance length $L_e = 2.5$ H, comprised of a curve $R_w = 2.0$ H and a parallel section of 0.5 H, is sufficient to insure the same coefficient as for the purely two-dimensional case. Although the coefficient should be slightly smaller for a narrow weir than a wide one, due to the relatively greater effect of the sidewall boundary layer, practically this effect is so small as to be of little consequence on prototypes. Figure 11 can be used for any width.

If the broad crest is long enough so that essentially parallel flow will occur at some section, pressures will be hydrostatically distributed. As such, the discharge cannot be affected for any tailwater level below critical depth. For the rectangular shape this means that $h_s/H \leq 0.667$. The capacity of the broad crested weir to take this submergence without affecting the head-discharge relationship is one of the features which makes it adaptable to low head water measurement. A staff gauge should also be mounted on the structure outside the stilling basin sidewall in order to read the downstream water level. This gauge may be used to check on the submergence, since it is doubtful if the limiting submergence could be determined from inspection of the flow pattern by an inexperienced operator.

Normally the rating curve is made up to read in terms of the upstream water level at the staff gauge or float. This water level is less than the total head H, by the amount of the velocity head in the approach channel at the section of the gauge. Although this velocity head is low, it can still be quite significant and must always be taken into account.

Example 1:

A rectangular broad crested weir is to be designed to measure a maximum discharge of 5.67 m^3/s, at which flow the downstream canal depth D_2 will be 1.07 m. In order to maintain gravity command of the land downstream it will be necessary to limit the head difference across the weir to 0.23 m.

(a) Calculate the width and weir height for the structure.

(b) Calculate the discharge when the head is 0.30 m.

The solution will proceed as follows:

(a) In order to retain the convenience of measurement with a single upstream reading, then h_s must not exceed 2/3 H. Also, it is specified that $H - h_s \leq 0.23$ m, from which H = 0.686 m and $h_s = 0.457$ m.

Since $P = D_2 - h_s$

then $P = 1.07 - 0.457 = 0.613$ m

and $b = 5.67/(1.687 \times 0.686^{3/2}) = 5.915$ m

(b) Since the required minimum crest length $L = 2.0\ H_{max}$, or 1.372 m, then for a 0.30 m head H/L = 0.219, and from Figure 11, C = 1.625, from which

$Q = 1.625 \times 5.915 \times 0.303^{3/2} = 1.579 \text{ m}^3/\text{s}$

13. Triangular Broad Crested Weir

This weir is a weir which is triangular in cross section but broad crested in longitudinal section, as shown in Figure 12. This weir combines the advantages of both weirs in that it may be used for small as well as large discharges, and the head discharge relationship is unaffected by submergence up to critical depth. Furthermore, because critical depth for a triangular section is 80% of the upstream head, this type of weir can take greater submergence than a rectangular section before the capacity is affected.

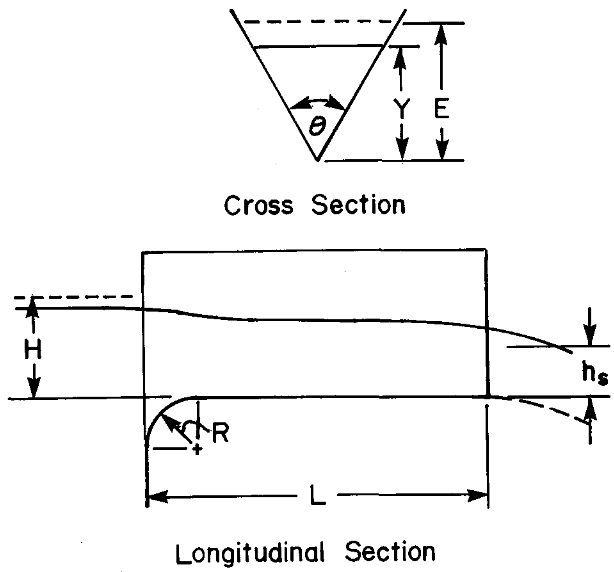

Figure 12. Definition Sketch for Triangular Broad Crested Weir

That critical depth for a triangular channel is 4/5 of the specific energy may be proved as follows:

The specific energy for flow in an open channel is given by

[23] $E = y + V^2/2g$

in which y is the flow depth above the lowest point of the bed, and V is the average velocity. For a triangular channel this may be written as

[24] $E = y + Q^2/\{2g[y^2 \tan(\theta/2)]^2\}$

in which Q is the discharge, and θ is the included angle at the apex of the triangle, in degrees. For critical flow the specific energy must be a minimum. Setting $dE/dy = 0$ results in $V^2/2g = y/4$, from which

[25] $y = 0.80\,E$

If a short length of triangular channel is fabricated and placed in a canal or ditch so that the apex of the triangle is at or above the stream bed, it is possible to have the flow pass through critical depth as it discharges over the triangular channel. In effect, the channel is a weir with a broad crest. The theoretical discharge may be calculated from the continuity equation, noting that for flow at critical depth

[26] $V = \sqrt{2gE/5}$

and

[27] $A = (0.80\ E)^2 \tan(\theta/2)$

in which A is the area of flow. Replacing E by H, in which H represents the total head above the apex of the triangle, the theoretical discharge becomes

[28] $Q = 1.268\ H^{5/2} \tan(\theta/2)$

Equation [28] would be exact if the following conditions could be satisfied: (1) The head loss between the point where H is measured and the critical section is negligible; (2) flow over the crest occurs with parallel stream lines and a hydrostatic pressure distribution; (3) the velocity distribution is uniform; and (4) the submergence head due to tailwater does not affect the relationship.

Condition 1 can be satisfied if the distance to the point of measurement is short, and if the weir has a streamlined entrance and a short smooth broad crest. Of course, "short" is a relative term, and the same weir cannot be short for all discharges and heads. The weir will be short relative to the head only when the H/L value is large, in which L is the length of the broad crest in the direction of flow, including the radius of the curve at the inlet. Streamlining at the entrance is necessary to avoid flow separation and eddy loss on the broad crest. This can be prevented with a simple curve of radius R_c, where $R_c = 0.4\ H_{max}$, in which H_{max} is the maximum head under which the measuring device will operate. The head should be measured about 3 H_{max} upstream from the weir, beyond the range of any local drawdown effect.

Condition 2, (i.e., parallel flow with a hydrostatic pressure distribution), can be achieved if the weir crest, L, is sufficiently long. According to experimentation, this condition requires a value of $L/H \geq 2.0$. Because condition 1 requires a short crest, and condition 2, a long crest, it is not possible to completely satisfy both conditions simultaneously. If the crest is too short and pressures become nonhydrostatic, the discharge would exceed the value given by the theoretical discharge in [28].

Condition 3, (i.e., a uniform velocity distribution), can never be exactly obtained. A truly uniform velocity distribution could occur only for irrotational or frictionless flow. Although the flow may be irrotational over most of the depth, there will also be a developing boundary layer on the broad crest, within which there will be a velocity variation with a zero velocity at the boundary. Because the average velocity in the boundary layer is less than in the flow above it, the weir coefficient C will be reduced accordingly. The boundary layer is known to develop, beginning at the upstream edge of the weir on the curved inlet, and increase in thickness along the broad crest. The effect of the boundary layer may be visualized as an imaginary wedge-shaped thin layer of "dead" water which does not contribute to the discharge over the weir. The thickness of this layer at any point is called the displacement thickness, δ^*, and is less than the actual thickness of the boundary layer itself. Because δ^* increases with crest length, the reduction in weir coefficient below the ideal value of 1.268 will be greater for large L/H values (or conversely, small H/L values).

Condition 4, (i.e., the discharge relationship is unaffected by submergence head), can be satisfied exactly. As long as the downstream water depth adjacent to the discharge from the weir does not exceed critical depth, or 0.80 H in this case, submergence will not be a factor. This is expressed symbolically as $h_s/H < 0.80$, in which hs is the submergence head due to tailwater.

The equation for the actual discharge may be written as

$$[29] \quad Q = C H^{5/2} \tan(\theta/2)$$

in which C is a variable dependent upon H/L. A plot of the weir coefficient C versus H/L, from Smith and Liang, is given in Figure 13. A single curve is drawn through the results from all weirs tested. Theoretically, the value of C should not be exactly the same for each weir at the same H/L value, because the boundary layer will be relatively thicker for the weirs with the shorter L value. Practically, the difference is small, particularly for large H/L values, so a single curve is still representative. The coefficient is less than the theoretical value of 1.268 for H/L values less than 0.6. This agrees exactly with the trend for the rectangular sharp crested weir (Figure 11).

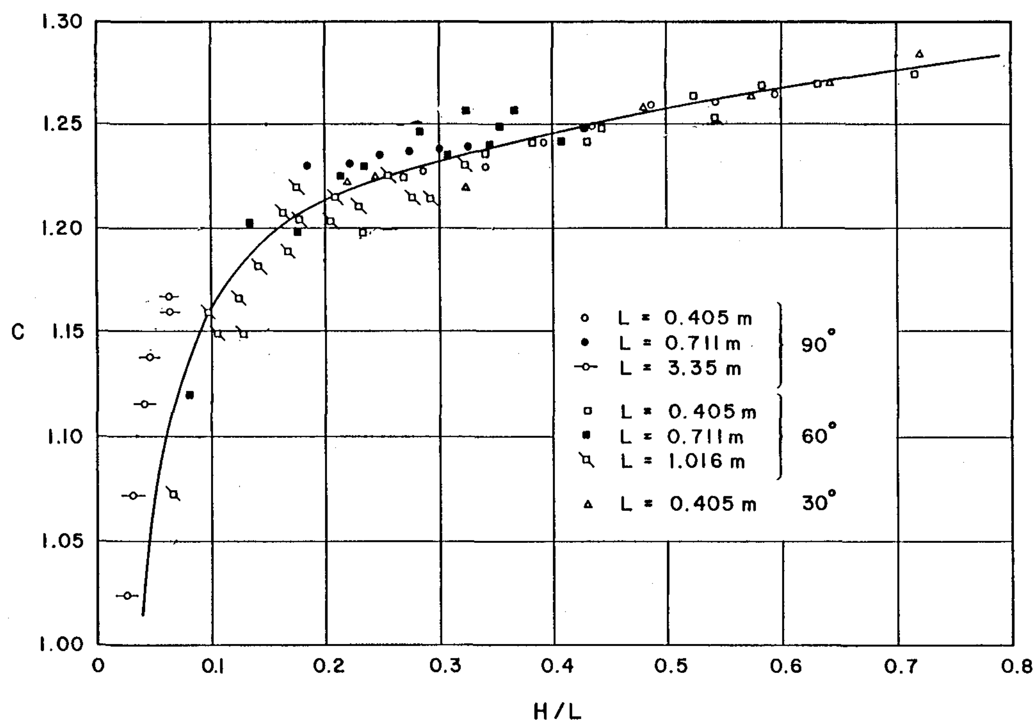

Figure 13. C vs H/L for a Triangular Broad Crested Weir

F. SUBMERGENCE

14. Effects

Any open channel flow measurement device may operate, at one time or another, with the downstream water level above the weir crest. The ventilated free nappe weirs are affected as soon as submergence begins. The broad crested weirs are intended to take a large submergence without having the discharge, head or coefficient affected by this submergence. However, due to temporary blockage from downstream, or some other cause, excessive submergence could occur even on a broad crested weir.

Any of the weirs previously discussed can be used for measurement in spite of submergence provided the submergence effect is taken into account. This means that two readings must be taken, both upstream head H and downstream head h_s, in which h_s is the depth of tailwater above the crest. The advantage of discharge measurement with a single upstream reading is lost. Downstream levels are usually more fluctuating and difficult to read accurately. Also, the effect of submergence can be somewhat different for two identical weirs if the downstream geometry of the structure and the channel is not similar. Thus, there is usually some loss of accuracy in attempting to determine the discharge under submergence effects.

Figure 14 shows the effect of submergence on the discharge coefficient for various types of weirs. This figure may be used to estimate the correction needed, recognizing that the curves were derived from tests of particular installations and will not apply exactly to every situation. The ratio h_s/H is the submergence ratio, that is, the downstream head divided by upstream head. The value C represents the weir coefficient as it would be for that upstream head if there was zero submergence, and C_s represents the reduced coefficient under submergence effect. Hence the ratio C_s/C is really the ratio of the discharge with submergence to the discharge without submergence.

It is noted that the rectangular sharp crested weir is greatly affected by submergence. The V-notch sharp crested weir is not nearly as sensitive. For equal submergence, say 50%, about half the nappe of the rectangular weir would be affected but only 25% of the V-notch nappe would be affected. This difference accounts for the reduced sensitivity of the V-notch to submergence. The ogee weir at design flow will also have a nappe at atmospheric pressure and will be affected almost as soon as submergence begins, but the presence of the weir shape underneath allows the flow to repel the tailwater to a greater degree than for the rectangular sharp crested weir. For example, even though the tailwater head may be 70% of the upstream head, which would reduce the discharge over the sharp crested weir to 70%, the ogee weir will prevent the tailwater level from fully penetrating upstream and the discharge will be reduced to only 88%.

The broad crested weirs are least affected, as previously discussed. In all cases however, $C_s/C = 0$ when $h_s/H = 1$, because the flow will be zero when the water level is equal on both sides of the weir.

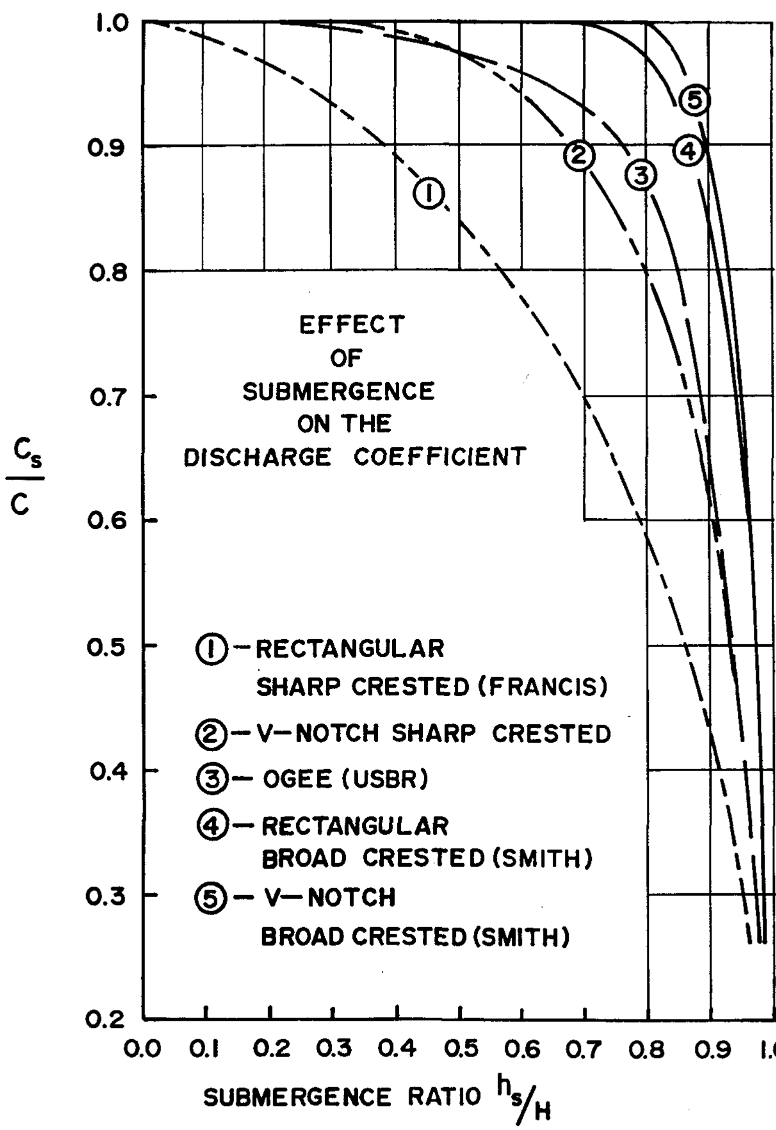

Figure 14. Effect of Submergence on Weir Coefficient

Example 2:

Calculate the discharge over a rectangular broad crested weir 4 m wide and with a crest length L = 1 m, given that the upstream and downstream water levels are 0.6 m and 0.5 m respectively. Assume the velocity of approach is negligible.

From Figure 11, for $H/L = 0.6$, $C = 1.71$, and from Figure 14, for $h_s/H = 0.5/0.6 = 0.833$, $C_s/C = 0.95$. Hence $C_s = 0.95 \times 1.71 = 1.62$, and $Q = 1.62 \times 4 \times 0.6^{3/2} = 3.01$ m^3/s.

This result can be roughly checked by noting that the flow over the weir will be subcritical and at a depth approximately equal to the tailwater depth. The velocity head will be $H - h_s$, hence $Q = AV = 0.5 \times 4 \sqrt{2g\,(0.6 - 0.5)} = 2.80$ m^3/s, which is within 7% of the value determined from Figures 11 and 14.

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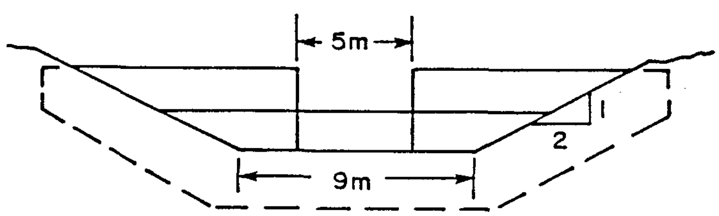
PROBLEMS

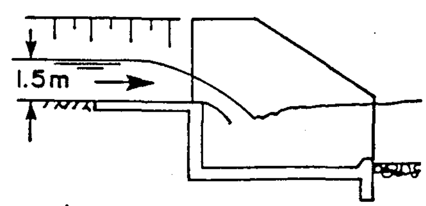
1. Discuss the importance of water measurement on an irrigation distribution system.

2. What is the disadvantage of a standard sharp crested weir for water measurement for irrigation?

3. A vertical drop structure is located in a canal with a bottom width of 9 m and 2:1 side slopes. The drop structure has a rectangular opening 5 m wide, and the bottom of the opening is flush with the canal bed. Estimate the discharge in m^3/s when the upstream water depth is 1.5 m.

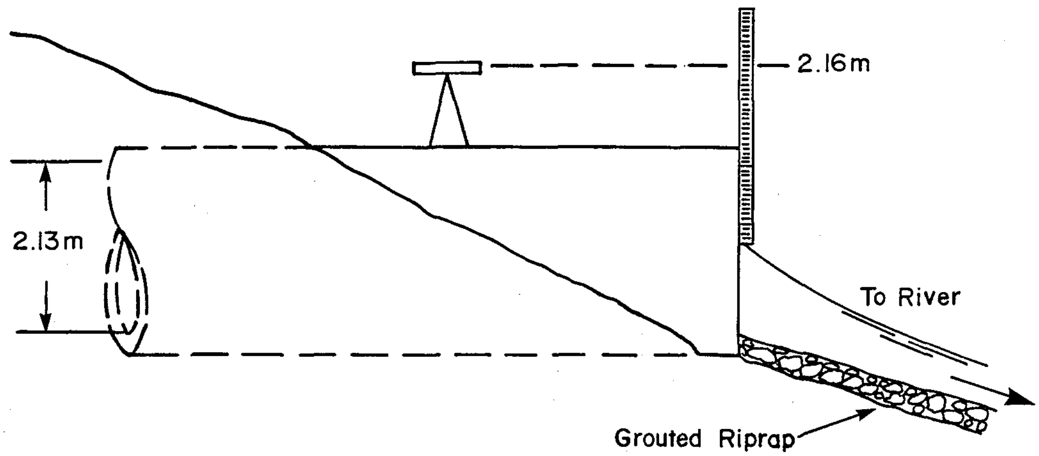


Front Elevation

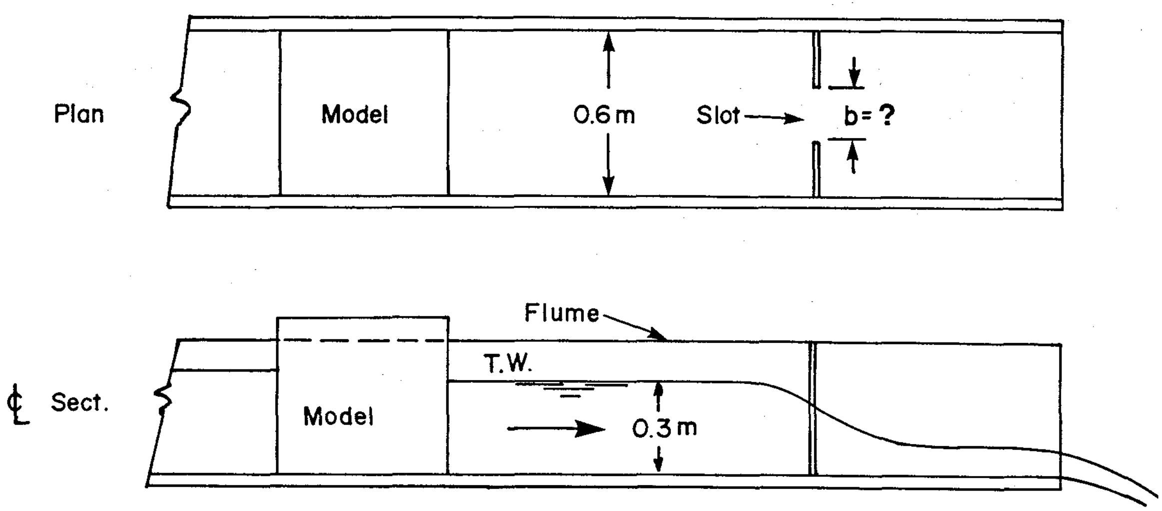


℄ Section

4. A 2.13 m diameter concrete storm sewer outfall has a wall thickness of 0.20 m. During a heavy storm a rod reading of 2.16 m was taken with the rod held at the level of the water surface at the plane of the outlet. With the rod held on top of the pipe the rod reading was 0.95 m. Calculate the discharge. (See the definition sketch on the following page.)



5. A hydraulic model is to be tested in a 0.6 m wide rectangular flume. When the model discharge is 0.037 m3/s the model tailwater depth must be 0.30 m. This depth could be created with a vertical sharp-crested weir with a height of 0.20 m located downstream from the model, but the tailwater would then be 0.20 m deep when Q = 0. Instead it is decided to use a rectangular vertical slot. In this way the tailwater will automatically drop to zero when Q = 0. What slot width must be used?

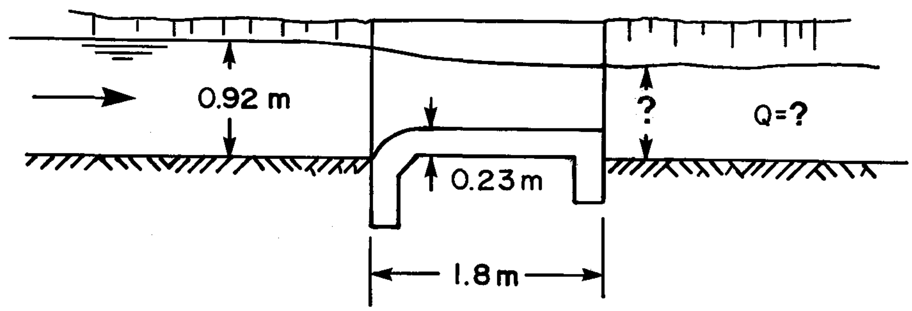


6 (a) It is determined that 0.3 m of differential head will be available for the measurement of 14 m^3/s using a rectangular broad crested weir. What minimum width of structure can be used for this measurement without requiring both upstream and downstream measurement?

(b) Calculate the discharge over this structure during a lower flow when the head H is only 0.3 m, and there is no tailwater submergence head.

7. A V-notch broad crested weir is used to measure runoff on a hydrological test basin. The weir has side slopes of 3 horizontal to 1 vertical, and the apex of the

triangle is 0.23 m above the stream bed. The length of the broad crest is 1.8 m. Calculate the discharge when the upstream flow depth is 0.92 m, given that the upstream channel flow area is 2.32 m2. What maximum flow depth can occur downstream without affecting the head-discharge relationship?



APPENDIX A

SYSTEM INTERNATIONAL D'UNITES

IN

WATER RESOURCES ENGINEERING

Preamble

Measurements that describe quantities used in water resources engineering have been expressed in a variety of traditional ways. These include such terms as the acre-foot, the second-foot-day, cubic feet per second per square mile, inches per square mile, U.S. gallons per minute, Imperial gallons per minute, and the miner's inch. Because of this diversity of terms, conversion of numbers among the U.S. customary measurement system, the English measurement system, and the metric measurement system (which includes some variations among user countries) has long plagued students and teachers alike.

Today, long overdue, Canada and the United States officially are shifting over to a new, internationally recognized and adopted system for measurement; System International d'Unites, or, SI. Over the next several years, students and practitioners in water resources engineering will need to be conversant in two or more measurement systems--the new SI, as it is phased in, and the older U.S. customary, English and metric systems as they are phased out of use. Even further into the future some practioners will need to remain conversant in the older as well as the SI systems in order to make use of data published prior to the introduction of SI.

The System International d'Unites was adopted in 1954 and formally named in 1960 by an International General Conference on Weights and Measures. It is a modernized metric system to permit exact definitions and to eliminate differences in standards among member nations. Improvements in this system have been made at subsequent international conferences. The System International d'Unites is officially abbreviated SI in all languages. It includes units most frequently used in the various fields of science and industry.

Like predecessor systems, SI consists of a limited number of basic units that describe fundamental quantities and a large number of derived units that come from the basic units to describe other quantities. SI has seven base units and two supplementary units from which are developed the large number of derived units used in everyday measurement. The SI base and supplementary quantities, their units and symbols are given in Table 1. Among these, four quantities--length, mass, time and temperature--have broad water resources applications. The remaining five quantities find a much lesser degree application in the more specialized subfields of water resources engineering.

The derived quantities, their units and symbols, and the formulas that express them in terms of base units are given in Table 2.

Because the orders of magnitude of many quantities cover wide ranges of numerical values, fourteen SI prefixes have been established to deal with decimal point placement. These prefixes, their SI symbol, and the multiplication factors that they represent are given in Table 3.

Of the SI base units, only the unit for mass (the kilogram) contains a prefix. But in using decimal multiples or submultiples for mass, prefixes are attached to the word "gram" (e.g. megagram or milligram).

The U.S. customary gravimetric engineering measurements treat force as a base unit and mass as a derived unit. In SI, the reverse is the case. The older metric systems use kilogram-mass and kilogram-force, which causes confusion. The force unit in SI is the newton whereas the kilogram is restricted to the measurement of mass. Pressure (force per unit area) is expressed in units of its defining derived quantities, newtons per square meter, but is also given a distinctive unit name, the pascal. (This situation is also the case for some other derived quantities.) To distinguish between gauge pressure and absolute pressure, the word "pressure" is given the appropriate adjective but no modification is made to the symbol Pa (pascal).

Symbols are used in the singular form, not as plurals (e.g., 5 m but not 5 ms). Symbols are not capitalized in SI unless the unit is derived from a proper name and is abbreviated (e.g., kelvin, K; or pascal, Pa). No period is used after an abbreviated SI unit symbol except at the end of a sentence. The only numerical prefixes which are capitalized are T, G and M.

The units of some terms that contain multiple symbols need special attention. The product of two or more units is denoted by a multiplication dot. Quotients are designated by placing a slope line (slash) between all units of the numerator and all units of the denominator. All symbols to the left of a slash are in the numerator and all symbols to the right are in the denominator.

Comparison of order of magnitude of quantities and conversion of numbers between SI and other measurement systems can be expected to be necessary for many years. A large number of such conversions have been assembled here for convenient reference.

Table 4 presents numerical values to convert to SI units from several commonly used units. This is done for the physical quantities most frequently used in water resources practice.

Table 5 presents in comparative form the equivalent units in SI and other measurement systems for the common physical quantities of length, area, volume, discharge, velocity, mass and force. The equivalent values are given to four significant figures with exponetial notation (where needed) as in Table 4.

Table 6 provides numerical values for some commonly used physical properties. Because the given properties of water do depend upon temperature, their numerical values are chosen for one or two typical temperatures. Numerical values are given to three significant figures except for atmospheric pressure, given to four significant figures.

TABLE 1. BASE AND SUPPLEMENTARY UNITS IN SI MEASUREMENT

Quantity	Unit	SI Symbol
Base Units:		
length	meter	m
mass	kilogram	kg
time	second	s
electric current	ampere	A
thermodynamic temperature	kelvin	K
amount of substance	mole	mol
luminous intensity	candela	cd
Supplementary Units:		
plane angle	radian	rad
solid angle	steradian	sr

TABLE 2. DERIVED UNITS IN SI MEASUREMENT

Quantity	Unit	SI Symbol	Formula
acceleration	meter per second squared	...	m/s^2
activity (of a radioactive source)	disintegration per second	...	(disintegration)/s
angular acceleration	radian per second squared	...	rad/s^2
angular velocity	radian per second	...	rad/s
area	square meter	...	m^2
density	kilogram per cubic meter	...	kg/m^3
electric capacitance	farad	F	$A \cdot s/V$
electrical conductance	siemens	S	A/V
electric field strength	volt per meter	...	V/m
electric inductance	henry	H	$V \cdot s/A$
electric potential difference	volt	V	W/A
electric resistance	ohm	Ω	v/A
electromotive force	volt	V	W/A
energy	joule	J	$N \cdot m$
entropy	joule per kelvin	...	J/K
force	newton	N	$kg \cdot m/s^2$
frequency	hertz	Hz	s^{-1}
illuminance	lux	lx	lm/m^2
luminance	candela per square meter	...	cd/m^2

TABLE 2. (cont'd) DERIVED UNITS IN SI MEASUREMENT

Quantity	Unit	SI Symbol	Formula
luminous flux	lumen	lm	cd·sr
magnetic field strength	ampere per meter	...	A/m
magnetic flux	weber	Wb	V·s
magnetic flux density	tesla	T	Wb/m^2
magnetomotive force	ampere	A	...
power	watt	W	J/s
pressure	pascal	Pa	N/m^2
quantity of electricity	coulomb	C	A·s
quantity of heat	joule	J	N·m
radiant intensity	watt per steradian	...	W/sr
specific heat	joule per kilogram-kelvin	...	J/kg·K
stress	pascal	Pa	N/m^2
thermal conductivity	watt per meter-kelvin	...	W/m·K
velocity	meter per second	...	m/s
viscosity, dynamic	pascal-second	...	Pa·s
viscosity, kinematic	square meter per second	...	m^2/s
voltage	volt	V	W/A
volume	cubic meter	...	m^3
wavenumber	reciprocal meter	...	(wave)/m
work	joule	J	N·m

TABLE 3. SI PREFIXES

Prefix	SI Symbol	Multiplication Factor
tera	T	1 000 000 000 000 = 10^{12} = E+12
giga	G	1 000 000 000 = 10^9 = E+9
mega	M	1 000 000 = 10^6 = E+6
kilo	k	1 000 = 10^3 = E+3
hecto**	h	100 = 10^2 = E+2
deka**	da	10 = 10^1 = E+1
deci**	d	0.1 = 10^{-1} = E−1
centi	c	0.01 = 10^{-2} = E−2
milli	m	0.001 = 10^{-3} = E−3
micro	μ	0.000 001 = 10^{-6} = E−6
nano	n	0.000 000 001 = 10^{-9} = E−9
pico	p	0.000 000 000 001 = 10^{-12} = E−12
femto	f	0.000 000 000 000 001 = 10^{-15} = E−15
atto	a	0.000 000 000 000 000 001 = 10^{-18} = E−18

**Avoid use of this prefix where possible.

TABLE 4. CONVERSION TO SI UNITS FOR PHYSICAL QUANTITIES FREQUENTLY USED IN WATER RESOURCES PRACTICE

To Convert From:		Multiply By:	To Obtain:	Units
Acceleration				
foot/second²	×	0.304 8	= meter/second²	m/s²
Area				
acre	×	4 047	= meter²	m²
foot²	×	0.092 90	= meter²	m²
hectare	×	1.000 E+4	= meter²	m²
inch²	×	6.452 E−4	= meter²	m²
mile²	×	2.590 E+6	= meter²	m²
yard²	×	0.836 1	= meter²	m²
Bending Moment or Torque or Moment of Force				
foot-pound$_{force}$	×	1.356	= newton-meter	N·m
Capacity (see Volume)				
Density (see Mass/Volume)				
Discharge (see Volume/Time)				
Energy or Work				
BTU	×	1 055	= joule	J
calorie	×	4.187	= joule	J
foot-pound$_{force}$	×	1.356	= joule	J
kilowatt-hour	×	3.600 E+6	= joule	J
Energy/Area·Time				
BTU/foot²-second	×	1.135 E+4	= watt/meter²	W/m²
BTU/foot²-hour	×	3.152	= watt/meter²	W/m²
calorie/cm²-minute	×	697.3	= watt/meter²	W/m²
Flow Rate (see Mass/Time or Volume/Time)				
Force				
dyne	×	1.000 E−5	= newton	N
kilogram$_{force}$	×	9.807	= newton	N
kip	×	4 448	= newton	N
pound$_{force}$	×	4.448	= newton	N
Force/Area (see Pressure)				
Heat				
BTU/foot²	×	1.136 E+4	= joule/meter²	J/m²
BTU/hour-foot²−°F	×	5.678	= watt/meter²−kelvin	W/m²·K
BTU/pound$_{mass}$	×	2 326	= joule/kilogram	J/kg
BTU/pound$_{mass}$−°F	×	4 187	= joule/kilogram−kelvin	J/kg·K
BTU/second-foot²−°F	×	2.044	= Watt/meter²−kelvin	W/m²·K
calorie/cm²	×	4.184 E+4	= joule/meter²	J/m²
calorie/gram	×	4 187	= joule/kilogram	J/kg
calorie/gram−°C	×	4 187	= joule/kilogram−kelvin	J/kg·K
Length				
foot	×	0.304 8	= meter	m
inch	×	0.025 40	= meter	m
mile, nautical	×	1 852	= meter	m
mile	×	1 609	= meter	m
yard	×	0.914 4	= meter	m

TABLE 4. (cont'd) CONVERSION TO SI UNITS FOR PHYSICAL QUANTITIES FREQUENTLY USED IN WATER RESOURCES PRACTICE

To Convert From:		Multiply By:		To Obtain:	Units
Mass					
$pound_{mass}$ (avoird.)	×	0.453 6		= kilogram	kg
slug	×	14.59		= kilogram	kg
ton (long, 2240 lb_m)	×	1 016		= kilogram	kg
ton (short, 2000 lb_m)	×	907.2		= kilogram	kg
ton (metric)	×	1 000		= kilogram	kg
Mass/Time					
$pound_{mass}$/second	×	0.453 6		= kilogram/second	kg/s
Mass/Volume or Density					
gram/cm³	×	1 000		= kilogram/meter³	kg/m³
$pound_{mass}$/foot³	×	16.02		= kilogram/meter³	kg/m³
$pound_{mass}$/gallon	×	119.8		= kilogram/meter³	kg/m³
slug/foot³	×	515.4		= kilogram/meter³	kg/m³
Power					
BTU/second	×	1 054		= watt	W
BTU/minute	×	17.57		= watt	W
BTU/hour	×	0.293 1		= watt	W
calorie/second	×	4.184		= watt	W
calorie/minute	×	0.069 73		= watt	W
foot-$pound_{force}$/second	×	1.356		= watt	W
horsepower (550 ft·lb_f/s)	×	745.7		= watt	W
horsepower (electric)	×	746.0		= watt	W
horsepower (water)	×	746.0		= watt	W
Pressure or Stress or Force/Area					
atmosphere	×	1.013 E+5		= pascal	Pa
bar	×	1.000 E+5		= pascal	Pa
dyne/cm²	×	0.100 0		= pascal	Pa
foot of water (39.4°F)	×	2 989		= pascal	Pa
$gram_{force}$/cm²	×	98.07		= pascal	Pa
inches of mercury	×	3 386		= pascal	Pa
inches of water (39.4°F)	×	249.1		= pascal	Pa
$kilogram_{force}$/meter²	×	9.807		= pascal	Pa
kip/inch²(ksi)	×	6.895 E+6		= pascal	Pa
millibar	×	100		= pascal	Pa
millimeters of mercury	×	133.3		= pascal	Pa
$pound_{force}$/foot²	×	47.88		= pascal	Pa
$pound_{force}$/inch² (psi)	×	6 895		= pascal	Pa
Stress (see Pressure)					
Temperature					
degree Celsius	+	273.15		= kelvin	K
degree Fahrenheit	+	459.67	÷ 1.8	= kelvin	K
degree Rankine			÷ 1.8	= kelvin	K
degree Fahrenheit	−	32	÷ 1.8	= degree Celsius	

TABLE 4. (cont'd) CONVERSION TO SI UNITS FOR PHYSICAL QUANTITIES FREQUENTLY USED IN WATER RESOURCES PRACTICE

To Convert From:	Multiply By:	To Obtain:	Units
Time			
day	× 8.640 E+4	= second	s
hour	× 3 600	= second	s
minute	× 60.00	= second	s
month (mean calendar)	× 2.628 E+6	= second	s
year (calendar)	× 3.154 E+7	= second	s
Torque (see Bending Moment)			
Velocity or Speed			
foot/second	× 0.304 8	= meter/second	m/s
foot/minute	× 0.005 080	= meter/second	m/s
foot/hour	× 8.467 E−5	= meter/second	m/s
kilometer/hour	× 0.277 8	= meter/second	m/s
mile/hour	× 0.447 0	= meter/second	m/s
Viscosity			
centipose	× 0.001 000	= pascal−second	Pa·s
centistokes	× 1.000 E−6	= meter²/second	m²/s
foot²/second	× 0.092 90	= meter²/second	m²/s
poise	× 0.100 0	= pascal−second	Pa·s
pound$_{force}$−second/foot²	× 47.88	= pascal−second	Pa·s
slug/foot-second	× 47.88	= pascal−second	Pa·s
stokes	× 1.000 E−4	= meter²/second	m²/s
Volume or Capacity			
acre-foot	× 1 233	= meter³	m³
~~barrel (oil, 42 gal)~~	~~× 0.159 0~~	~~= meter³~~	m³
board foot	× 0.002 360	= meter³	m³
bushel (U.S.)	× 0.035 24	= meter³	m³
foot³	× 0.028 32	= meter³	m³
gallon (U.S. liquid)	× 0.003 785	= meter³	m³
inch³	× 1.639 E−5	= meter³	m³
liter	× 0.001 000	= meter³	m³
yard³	× 0.764 6	= meter³	m³
Volume/Time or Discharge or Flow Rate			
foot³/second	× 0.028 32	= meter³/second	m³/s
foot³/minute	× 4.719 E−4	= meter³/second	m³/s
gallon/minute	× 6.309 E−5	= meter³/second	m³/s
gallon/day	× 4.381 E−8	= meter³/second	m³/s
Work (see Energy)			

TABLE 5. COMMONLY USED EQUIVALENT UNITS IN WATER RESOURCES[1]

A. LENGTH:

Unit	Equivalent[a] [b]					
	millimeter	inch	foot	meter[c]	kilometer	mile
millimeter	1	0.039 37	0.003 281	0.001 000	1 E-6	0.621 4 E-6
inch	25.40	1	0.083 3	0.025 40	25.40 E-6	15.78 E-6
foot	304.8	12	1	0.304 8	304.8 E-6	189.4 E-6
meter[c]	1 000	39.37	3.281	1	0.001	621.4 E-6
kilometer	1 000 000	39 370	3 281	1 000	1	0.621 4
mile	1 609 000	63 360	5 280	1 609	1.609	1

[1]Footnotes for all parts of Table 5

(a) Equivalent values are shown to 4 significant figures.

(b) Multiply the numerical amount of the given unit by the equivalent value shown (per single amount of given unit) to obtain the numerical amount of the equivalent unit (e.g.: 5 inches × 0.025 40 m/inch = 0.127 0 m).

(c) This is the SI expression, in base units or derived units, for the physical quantity.

B. AREA

Unit	Equivalent[a] [b]						
	sq. inch	sq. foot	sq. meter[c]	acre	hectare	sq. kilometer	sq. mile
sq. inch	1	0.006 944	645.2 E-6	0.159 4 E-6	64.52 E-9	645.2 E-12	249.1 E-12
sq. Foot	144	1	0.092 90	22.96 E-6	9.290 E-9	92.90 E-9	35.87 E-9
sq. meter[c]	1 550	10.76	1	247.1 E-6	1 E-4	1 E-6	386.1 E-9
acre	6 273 000	43 560	4 047	1	0.404 7	0.004 047	0.001 563
hectare	15 500 000	107 600	10 000	2.471	1	0.01	0.003 861
sq. kilometer	1.550 E+9	10 764 000	1 000 000	247.1	100	1	0.386 1
sq. mile	4.014 E+9	27 880 000	2 590 000	640	259	2.590	1

C. VOLUME:

Unit	Equivalent[a] [b]							
	cu. inch	liter	U.S gallon	cu. foot	cu. yard	cu. meter[c]	acre-foot	sec-foot-day
cubic inch	1	0.016 39	0.004 329	578.7 E-6	21.43 E-6	16.39 E-6	13.29 E-9	6.698 E-9
liter	61.02	1	0.264 2	0.035 31	0.001 308	0.001	810.6 E-9	408.7 E-9
U.S. gallon	231.0	3.785	1	0.133 7	0.004 951	0.003 785	3.068 E-6	1.547 E-6
cubic foot	1 728	28.32	7.481	1	0.037 04	0.028 32	22.96 E-6	11.57 E-6
cubic yard	46 660	764.6	202.0	27	1	0.764 6	619.8 E-6	312.5 E-6
cubic meter[c]	61 020	1 000	264.2	35.31	1.308	1	810.6 E-6	408.7 E-6
acre-foot	75.27 E+6	1 233 000	325 900	43 560	1 613	1 233	1	0.504 2
second-foot-day	149.3 E+6	2 447 000	646 400	86 400	3 200	2 447	1.983	1

D. DISCHARGE (FLOW RATE, VOLUME/TIME):

Unit	Equivalent[a] [b]					
	gallon/min	liter/sec	acre-foot/day	foot³/sec	million gal/day	meter³sec[c]
gallon/minute	1	0.063 09	0.004 419	0.002 228	0.001 440	63.09 E-6
liter/second	15.85	1	0.070 05	0.035 31	0.022 82	0.001
acre-foot/day	226.3	14.28	1	0.504 2	0.325 9	0.014 28
foot³/second	448.8	28.32	1.983	1	0.646.3	0.028 32
million gallons/day	694.4	43.81	3.069	1.547	1	0.043 81
meter³/second[c]	15 850	1 000	70.04	35.31	22.82	1

E. VELOCITY:

Unit	Equivalent[a] [b]				
	foot/day	kilometer/hour	foot/sec	mile/hour	meter/sec[c]
foot/day	1	12.70 E-6	11.57 E-6	7.891 E-6	3.528 E-6
kilometer/hour	78 740	1	0.911 3	0.621 4	0.277 8
foot/second	86 400	1.097	1	0.681 8	0.304 8
mile/hour	126 700	1.609	1.467	1	0.447 0
meter/second[c]	283 500	3.600	3.281	2.237	1

F. MASS:

Unit	Equivalent[a] [b]					
	$pound_{mass}$	kilogram[c]	metric slug	slug	metric ton	long ton
$pound_{mass}$(avoird.)	1	0.453 6	0.046 25	0.031 08	453.6 E-6	446.4 E-6
kilogram[c]	2.205	1	0.102 0	0.068 52	0.001	984.2 E-6
metric slug	21.62	9.807	1	0.672 1	0.009 807	0.009 651
slug	32.17	14.59	1.490	1	0.014 59	0.014 36
metric ton	2 205	1 000	102.0	68.52	1	0.984 2
long ton	2 240	1 016	103.7	69.63	1.016	1

G. FORCE:

Unit	Equivalent[a] [b]			
	dyne	newton[c]	$pound_{force}$	$kilogram_{force}$
dyne	1	1 E-5	2.248 E-6	1.020 E-6
newton[c]	100 000	1	0.224 8	0.102 0
$pound_{force}$	444 800	4.448	1	0.453 6
$kilogram_{force}$	980 700	9.807	2.205	1

TABLE 6. COMMONLY USED NUMERICAL VALUES FOR PHISICAL PROPERTIES

Quantity	U.S. FLT System	SI System
Gravitational acceleration g (std., free fall)	32.2 ft/s²	9.807 m/s²
Density of water, ρ_{H_2O} @ 50°F/10°C	1.94 slugs/ft³	1 000 kg/m³
Specific weight of water, γ_{H_2O} @ 50°F/10°C	62.4 lb/ft³	9.807 kN/m³
Dynamic viscosity of water, μ_{H_2O} @ 50°F/10°C	2.73×10^{-5} lb·s/ft²	1.30×10^{-3} Pa·s
@ 70°F/20°C	2.05×10^{-5} lb·s/ft²	1.00×10^{-3} Pa·s
Kinematic viscosity of water, ν_{H_2O} @ 50°F/10°C	1.41×10^{-5} ft²/s	1.30×10^{-6} m²/s
@ 70°F/20°C	1.06×10^{-5} ft²/s	1.00×10^{-6} m²/s
Atmospheric pressure, p (std.)	14.70 psia 2 116 psfa	101.3 kP/a²

APPENDIX B

ANSWERS TO PROBLEMS

CHAPTER I STORAGE DAMS

2. FB = 4.1 m

4. b = 101.2 m

5. f = 281.2 kN/m^2 at toe; 291.0 kN/m^2 at heel

6. f = 483.6 kN/m^2 at toe; 56.7 kN/m^2 at heel; no tension, therefore safe

7. f_{max} = 3031 kN/m^2; f = 0.527

8. 2.99 horizontal to 1 vertical

9. a) f = 2001 kN/m^2 and 354 kN/m^2

   b) f = 234.6 kN/m^2 tension at the heel, therefore unsafe

10. $\alpha = 0.12$

13. t = 6.4 m

CHAPTER II SPILLWAYS

1. q = 26.39 $m^3/s/m$

2. R_b = 2.30 m; 2x = 135.8 m

4. d = 0.997 m

6. d_1 = 1.35 m

7. d_1 = 1.015 m

8. x = 17.07 m and y = 2.85 m

10. H_d = 1.843 m

13. d_3 = 5.12 m (compared to 6.11 m for normal jump)

14. L_b = 15 m

16. $W = 67.35$ m; $H = 5.33$ m; $P = 1.33$ m; $t = 1.05$ m; $Le = 10.67$ m; crest elev. 91.17 m; basin elev. 69.0 m; $L_b = 27$ m

17. 40.9 m^3/s; 2.65 m

18. a) $Q = 9.41\ m^3/s$

    b) $Q = 16.15\ m^3/s$

20. Radius too short, $P/\gamma = -6$ m

21. $L = 37.5$ m, $D_o = 1.377$ m; $H_w = 0.788$ m; $R_c = 0.275$ m; $S_o = 1.5\%$

CHAPTER III OUTLET WORKS

3. $P/\gamma = -1.04$ m

4. -7.63 m, unsafe

5. 10% increase

7. $B_1 = 3.21$ m; $L_b = 5.27$ m; $R = 0.141$; $B_2 = 4.69$ m; $L_t = 6.77$ m

8. 4.91 m; 3.09 m lower

CHAPTER IV GATES AND VALVES

2. $d = 5.35$ cm

4. $P/\gamma = 0.90$ m of water

5. $R = 1059.5$ kN/pin, horizontal

9. $A_A = 0.21\ m^2$

14. $F = 150.5$ kN

CHAPTER V DIVERSION WORKS

1. $D = 7.70$ m; $U = 340$ kN/m

3. $L/h = 8.16$ (approx.)

4. $P/\gamma = 8.25$ m of water

5. a) $B = 2.86$ m

   b) $d_2 = 2.17$ m required; 2.9 m available, therefore OK

7. a) $Q \approx 0.45\ m^3/s$; A H 0.31 m^2

   b) 500 fish/h

CHAPTER VI DROP STRUCTURES

1. 1.71 km apart

5. $v_1 = 7.76$ m/s

6. a) $P = 1.574$ m

   b) $E_1 = 7.78$ m

7. $L = 11.87$ m

9. 9 orifices each 1.00 m in diameter

10. $P/\gamma = h_j = 2.324$ m of water; $h_w = 4.4$ m; $U = 412.7$ kN/m

CHAPTER VII STONE STRUCTURES

1. $h_w = 0.63$ m; $d_m = 303$ mm

2. $d_m = 60$ mm on straight; $d_m = 234$ mm on outside of curve

3. $d_t = 0.64$ m

4. $X = 2.60$ m; $Y = 2.52$ m

CHAPTER VIII FLEXIBLE CHANNEL LININGS FOR EROSION CONTROL

7. $\tau_b = 29.4\ N/m^2$; allowable 45 N/m^2

8. $B = 8$ m; **F** = 1.52

9. $q = 3.98\ m^2/s$; $q_s = 1.78\ m^2/s$ (44%)

CHAPTER IX CONVEYANCE AND CONTROL STRUCTURES

3. $d = 0.554\ Q_{0.4}$

4. $b = 1.882$ m

5. $h_g = 0.285$ m

6. $h = 0.803$ m

8. $P/\gamma = -2.203$ m

11. D will increase due to the force F_b, so, h will decrease. Therefore must use a rounded lip to increase C_d and compensate for reduced head.

12. $W_g = 9.76$ m; $L_b = 7.56$ m

13. $L_b = 4.20$ m

CHAPTER X CULVERT HYDRAULICS

2. 2.21 m^3/s; RCP 1000 mm will discharge 2.22 m^3/s

3. 4.95 m^3/s; RCP 1000 mm will dishcarge 5.07 m^3/s

4. 1.84 m^3/s; RCP 1000 mm will discharge 1.87 m^3/s

5. CSP V = 3.18 m/s; $d_m = 0.193$ m; L = 5.4 m;

   RCP V = 3.51 m/s; $d_m = 0.235$ m; L = 6.3 m

6. CSP $d_m = 0.051$ m; L = 2.3 m

   RCP $d_m = 0.104$ m; L = 4.1 m

7. a) 2.73 m^3/s

   b) 3.09 m^3/s

   c) 3.08 m^3/s

8. Helical CSP 1.57 m^3/s (15% reduction)

9. 0.030

10. Use expansion: $V_2 = 2.42$ m/s; new h = 0.69 m

CHAPTER X I FLOW MEASUREMENT

3. $Q = 17.2 \text{ m}^3/\text{s}$

4. $Q = 9.30 \text{ m}^3/\text{s}$

5. $b = 0.13 \text{ m}$

6. a) $W = 9.72 \text{ m}$

   b) $Q = 2.56 \text{ m}^3/\text{s}$

7. $Q = 1.61 \text{ m}^3/\text{s}$; $d_t = 0.802 \text{ m}$

B 6

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