TECHNICAL ADVANCEMENTS IN SPILLWAY DESIGN

Progress and Innovations from 1985 to 2015

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Progress and Innovations from 1985 to 2015

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Progress and Innovations from 1985 to 2015

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Initially, the Bulletin included an additional chapter dedicated to the discussions of Debris and High Sediment Flow in Spillway Operation. This chapter was moved to another bulletin in a combined preparation by the Committee on Hydraulics for Dams and the Committee on Flood Control. The chapter was written by R. Wark (Australia) and received inputs by S. P. Sen (India).

1. INTRODUCTION

1.1 PURPOSE AND SCOPE OF THE BULLETIN

The purpose of this bulletin is to provide to the dam engineering community information about the technical features of different types of spillways developed and implemented during the last four decades and some hydraulic features associated with these structures. The information presented in this bulletin does not supersede the contents of previous ICOLD bulletin on this issue, but rather complements that information with recent technical developments.

Although spillways as such, can be built with many different types of structures and arrangements, the approach used for this bulletin selected some specific types of structures – stepped spillways, labyrinth, PKW and tunnel spillways – and analysed conventional types of structures operating in special conditions – very large flows, very high head, very cold climate. The information was complemented with a chapter on application of design techniques using numerical and physical models, and a chapter on economics and cost issues. The very important issue of debris and sedimentation control affecting spillway design and operation was moved to another ICOLD bulletin that will include dam safety issues resulting from floods.

In each chapter, the subjects treated focused on the main questions that affect the specific issues or structures considered with the objective of providing information for the designer and for the project owner for the choice and development of the project. The subjects exposed were generally based on recent physical and experimental hydraulic studies and on real examples of works built in different parts of the world.

The recent evolution of the technologies related to the design, construction and operation of spillways has focused mainly on safety and economics. Spillways are the key elements of a dam project to ensure the necessary protection of the project against the destructive action of floods and flows in excess to the discharge capacity of other dam features. However each dam project has individual characteristics that require the selection of specific types of spillways and this has led to the development of different approaches to select and define the best type of spillway for each project. This bulletin aims to present and discuss the most recent results of this evolution.

1.2 RELATION TO EARLIER BULLETINS

ICOLD has produced various bulletins related to the design and operation of spillways. As mentioned, the present bulletin does not intend to supersede these publications, although it is recognised that certain subjects like energy dissipation deserved a more ample treatment. In any case, the aggregated of the following bulletins comprise a significant volume of valuable technical information on the hydraulic design of spillways:

Nº	BULLETIN TITLE	YEAR
49A	OPERATION OF HYDRAULIC STRUCTURES OF DAMS	1986
58	SPILLWAY FOR DAMS	1987
81	SPILLWAYS – SHOCK WAVES AND AIR ENTRAINEMENT	1992
82	SELECTION OF THE DESIGN FLOOD – CURRENT METHODS	1992

108	COST OF FLOOD CONTROLS IN DAMS	1997
125	DAMS AND FLOODS – GUIDELINES AND CASE HISTORIES	2003
130	RISK ASSESSMENT IN DAM SAFETY MANAGEMENT	2005
142	SAFE PASSAGE OF EXTREME FLOODS	2012
156	INTEGRATED FLOOD RISK MANAGEMENT	2014

1.3 CONTENT OF THE BULLETIN

A summary description of the content of the bulletin follows:

Chapter 2 – LARGE CAPACITY SPILLWAYS

This chapter addresses the design and specific arrangements of spillways with capacities varying from 20 000 m^3/s to the present largest existing spillway designed for 110 000 m^3/s. It includes references and questions related to gated and ungated spillways highlighting the issue of the general use of very large radial gates. It also addresses the questions of structures for energy dissipation and some references of operation of large spillway and the intrinsic risk associated with these structures.

Chapter 3 - HIGH HEAD SPILLWAYS – THE CHALLENGE OF ENERGY DISSIPATION AND SCOUR CONTROL DOWNSTREAM

This chapter deals with the problem of energy dissipation by jets issuing from high head spillways and the corresponding scour process developed in the downstream rock floor. A physical description of the scour process is presented as well as information on semi-empirical formulae and physically based evaluation methods used for estimating the size of the scour hole. Methods for controlling the scour are also presented including the discussion of the pre-excavation of plunge pools, its sizing and a discussion on the conditions and criteria related to its use.

Chapter 4 – STEPPED SPILLWAYS

This chapter describes the main applications of stepped spillways, their hydraulics and expected performance and the aspects that require specific attention during the design phase. The chapter presents a comprehensive review of the hydraulics of stepped spillways with information of recent investigations and from these, criteria for design purposes. Case examples are also presented to illustrate the performance of real structures.

Chapter 5 – LABYRINTH SPILLWAYS

Design information for labyrinth spillways is presented in this chapter. An ample review of different structure geometries and corresponding hydraulic parameters is presented. Hydraulic design criteria and suggested design procedures are included. Literature references for additional details and significant case examples are also included.

Chapter 6 – PKW SPILLWAYS

This chapter contains information on Piano-key Weirs (PKW), a variation of traditional labyrinth spillways developed in the last decade. Information comprises description of the structure, hydraulic performance as observed in laboratories and in actual projects, hydraulic design criteria and design parameters, some information on structural and construction issues and actual cases and variations in the arrangements. Extensive literature references are included.

Chapter 7 – TUNNEL, SHAFT AND VORTEX SPILLWAYS

This chapter considers different kinds of tunnel spillways, such as high-level, low-level (or low outlet), vortex, shaft and orifice tunnels. The general arrangement strategies, control structure, conveyance structure, terminal structure, operation issues, risk analysis and considerations are discussed. Some new developments, innovations and applications on tunnel spillway design for high dams are specially given which have been achieved in past 20 to 30 years. Some cases are presented as examples.

Chapter 8 - SPECIAL PROBLEMS OF SPILLWAYS IN VERY COLD CLIMATE

This chapter deals mainly with the serious problems of snow and ice blocking, partly or fully, spillways passages and thus reducing their water conveyance capacity in cold regions. It also covers the issue of negative temperatures during construction and operation of spillway structures that impose specific requirements to the design of structures and construction material. In addition other challenges concerning regulation of gates and special loads and impacts from ice on structures, are discussed as well.

Chapter 9 - COMPOSITE MODELLING OF HYDRAULIC STRUCTURES

Composite modelling is the effective utilization of both physical and numerical models, used either in series or in parallel, to solve difficult hydraulic problems. This chapter discuss the use, advantages and limitations of physical and numerical modelling in hydraulic engineering practice and describes the processes of using composite modelling in the design of spillways and outlet dam structures.

Chapter 10 – ECONOMICS, RISK AND SAFETY ISSUES IN SPILLWAY DESIGN

Spillways are the main safety-assurance feature of any type of dam. For this reason as a rule, direct cost reduction attempts in these structures will normally mean increases in the risk of lowering the safety of the overall dam project. However, in the formulation of the overall dam project layout the analysis of different types of spillway alternatives, for the same safety criteria, may have a significant impact on the overall cost of the project. In addition different spillway configuration, such as single gated structure versus combination of gated and ungated spillways, for example, may lead as well to important cost savings. This chapter discusses these issues.

2. LARGE CAPACITY SPILLWAYS

2.1. INTRODUCTION

The purpose of this chapter is to present and discuss the engineering issues associated with the design, construction and operational features of spillways designed to discharge very large flood flows. Although there are no major differences between the criteria used for the hydraulic design of different sizes of spillways, construction and cost considerations play a significant role in the optimisation of design solutions for large spillways. In addition it is recognised that these cases deserve special considerations due to the inherent operational risks associated with the management of large flows from the reservoir water level to the lower downstream reaches of the river.

For the purpose of this Bulletin, very large spillways are arbitrarily defined as structures designed for discharging flows larger than 20 000 m^3/s and/or with specific discharge in the restitution facility, larger than 130 $m^3/s/m$.

Most of present and future dam projects incorporating very large spillways are being built in Asia, Latin America and Africa, in rivers draining very large basins. Tables shown at the end of this chapter, depicts data for projects incorporating very large spillways built in selected countries. Projects of similar magnitude exist in other countries as well. These projects allowed gathering important experience which forms the basis of the discussions presented hereinafter.

2.2. GENERAL ARRANGEMENT STRATEGIES FOR LARGE CAPACITY SPILLWAYS

2.2.1. The spillway design flood

It is not the purpose of the present document to discuss methodologies for determining the spillway design flood. However, in discussing large capacity spillways, it is pertinent to observe the background on which the design flood magnitude is generally defined.

The characterisation of a spillway design flood generally requires:

- The return period (T_R) of different peak values.
- The volume of water associated with the peak flood values.
- The PMF or Probable Maximum Flood, a deterministic value assumed to be the absolute maximum flood possible to be produced at the project site.

Many modern large spillways were designed to discharge the 1:10 000-year flood, which theoretically corresponds to the natural river flow whose peak has the probability of occurrence of 1% in an expected project life of 100 years. As a rule, these spillways are capable of discharging this flood peak with a reservoir level at its maximum allowable level, which often is higher than the normal maximum operating level of the project but does not encroach into the dam freeboard. If the reservoir area is large enough, the reservoir volume created by this level difference may attenuate the peak flood hydrograph and represents a certain margin of safety which is specifically considered in the strategy for managing the flood to be disposed of.

Instead of using the statistical 10 000-yr flood, the PMF is also widely used as the basic element to define the spillway design flow. However, as a rule, for large projects, both flood values are normally computed and with these, discharge conditions defined. It is common to define the spillway design flood as the 10 000-yr flood associated with the maximum allowable reservoir level, and verify the spillway capacity to discharge the PMF from a reservoir level encroaching into the dam freeboard.

Although these criteria are widely used for the design of large spillways there is an inherent risk of underestimation of the flood for which discharge capacity should be provided, due to the possible non-representativeness of the data used for computation of probabilistic values and for the PMF determination. In fact these criteria do not take into consideration the statistical simplifications of using a limited range of data to compute a return period many times larger, as well as the physical future changes of the site, such as climate changes and watershed modifications affecting runoff. These questions are extensively discussed in the ICOLD Bulletin No.142 *Safe Passage of Extreme Floods* (ICOLD, 2012) and deserve careful attention in the design of large spillways. The determination of confidence intervals for the computed flood value and the provision of additional means of discharge such as emergency spillways may be a prudent consideration.

2.2.2. Location, type and size

The technical characteristics and the cost of a spillway are not only related to the structures and equipment that control the flood releases themselves but are also heavily affected by its location and by the arrangement of other project features, particularly the dam type and height and the overall site characteristics. Many variations of arrangements are possible and cost optimisations are worked out for each project and site considering the flow magnitude, the operational requirements and the topographical and geological characteristics, as the most relevant parameters.

Environmental regulations in many countries require that a continuous steady flow, compatible with the environmental conditions of the downstream stretch of the river, be released from the dam barrier. This may affect the arrangement and operation of the spillway especially in cases where the structures are built in large rivers. The present trend, however, seems to favour locating such release facilities in a separate entity. As an example the Belo Monte Project, presently being built in Brazil, which bypasses a long river curve, will have a spillway designed for 62 000 m^3/s and must release in the bypassed stretch during the dry period, a minimum continuous flow of 700 m^3/s and, during the wet season, flow values that may reach 8 000 m^3/s. Instead of doing this through the spillway these flows will be released through a separate structure which will include an additional powerhouse to use the "environmental" flow. Many projects in various countries are following these criteria.

Spillways discharging large flows are necessarily large structures irrespective of the relative size of the dam or other appurtenant structures. If concrete dams are contemplated, the natural location of the spillway is over or across the dam. Generally the spillway will be placed near or close to the river area, to facilitate the restitution to the river, a location that may have to be optimised when there is a powerhouse foreseen for the same location. In the Itaipu Project, Brazil-Paraguay border, (Fig. 2.1), for example, the spillway was located in one abutment since the whole river valley was fully taken by the 18-generating unit powerhouse.

Fig. 2.1 – Itaipu Project (Brazil-Paraguay border) and its 62 200 m^3/s spillway

Depending on the height and type of dam – embankment or concrete gravity, arch-gravity or arch – many different arrangements and configurations are possible and the creativity of engineers worldwide has provided innumerable examples.

For dams in wide valleys the spillway will normally be built over or through a concrete portion of the dam, whether or not the rest of the structure is an embankment. An example of this case is the Tucurui Project (Fig. 2.2), in Brazil. In all these cases the width of the valley was compatible with the width of the spillway.

Fig. 2.2 - View of Tucurui Project (Brazil), 110 000 m^3/s spillway

However if the dam site is not wide enough, it may be difficult to fit the whole spillway over the dam. Different solutions have been employed, from locating the spillways completely outside the dam structure, to combining surface spillways with intermediate and bottom outlets, as in Ertan Dam (Fig.2.3), in China. This project includes seven passages in the dam crest discharging 6 260 m^3/s, six mid-level outlets with capacity of 6 930 m^3/s, four bottom outlets for 2 084 m^3/s and two tunnels discharging 7 400 m^3/s. The total design capacity is 22 674 m^3/s.

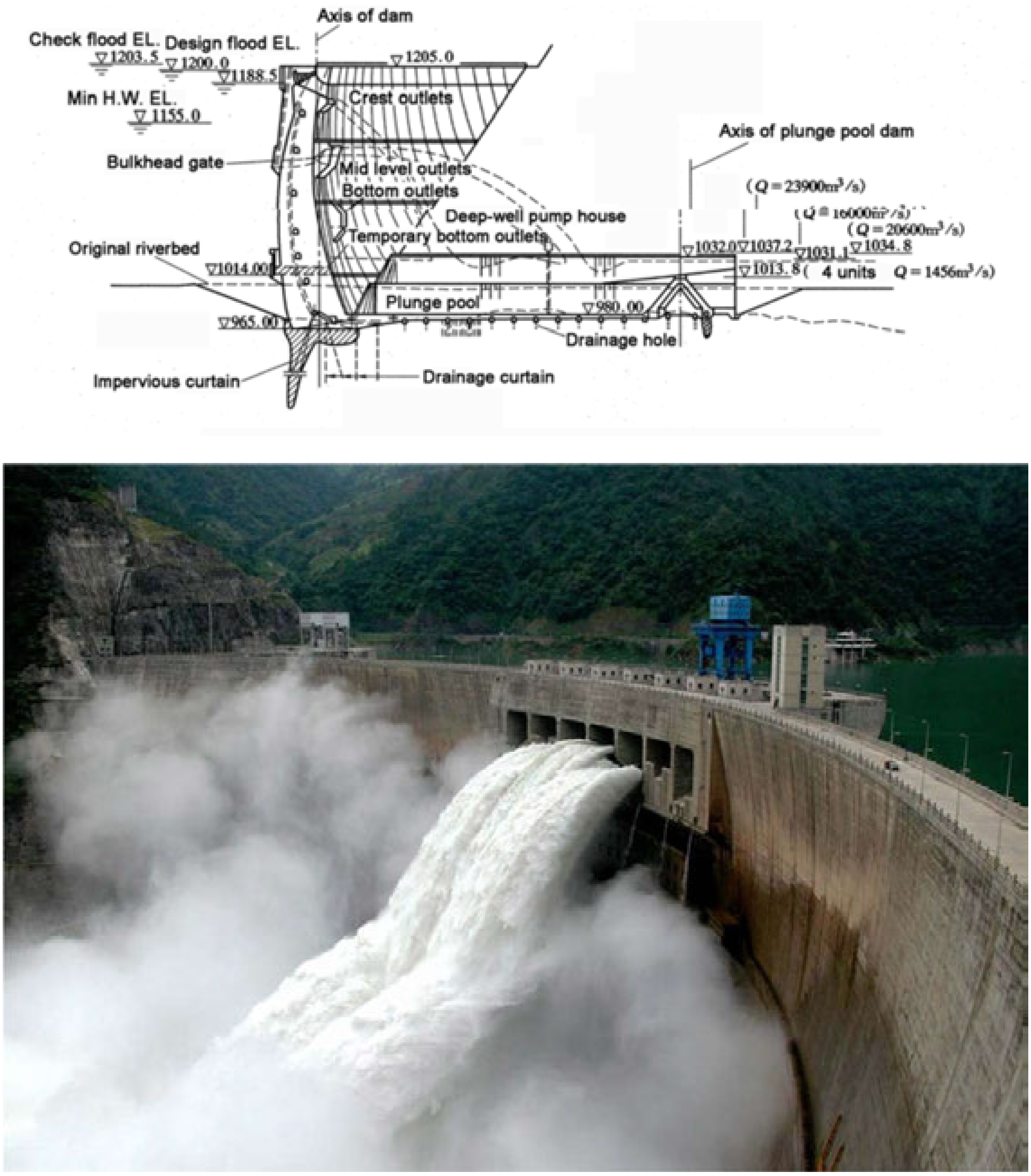


Fig. 2.3 - Cross section and photo of Ertan Dam – China

2.2.3. Flood peak attenuation by reservoir

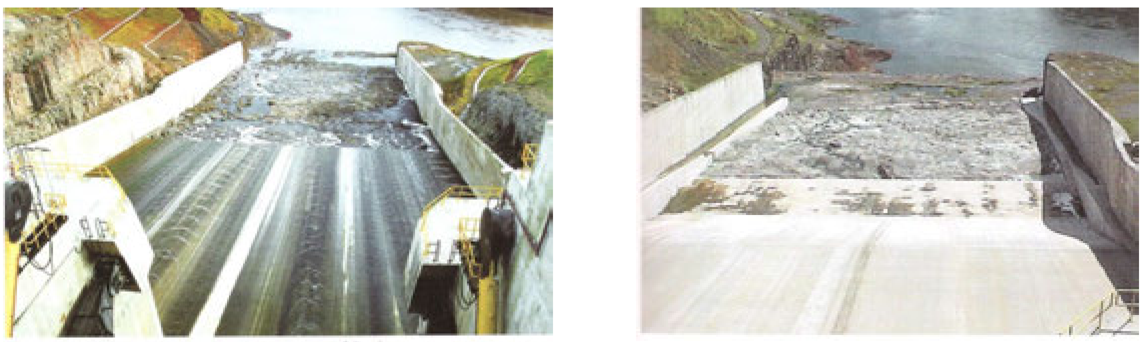
The use of part of the reservoir volume to store part of the flood hydrograph and allow the reduction of the flood peak and reduce the spillway design flood, is a common practice for large spillway projects. Table 2.1 depicts some cases where this practice have been used (compare the columns "Peak inflow" and "Spillway design flood"). In such a case, particular attention has to be taken by the engineer to the elevation reached inside the reservoir to discharge the peak flow of the natural hydrograph.

This practice, however, increases the reservoir area and besides the cost impact – which has to be compared with the spillway and dam cost – may increase the environmental impact of the project. For large reservoirs in rather flat areas, and depending on the environmental legislation of each country, this may pose unsurpassable difficulties.

2.2.4. Single structure vs. different functional components

Spillways designed as a single structure to discharge very large flows will necessarily be long structures and in certain cases it will be economically difficult to fit the whole spillway in the axis of the dam. In these cases it may be convenient to divide the spillway structure in two or more parts, one, to be operated more frequently, discharging the smaller more common floods (referred to as service spillways), and the others (auxiliary spillways) to complement the full capacity of the spillway system. Even when there is no problem of space, the division of the spillway into separate structures can be used to allow savings by building the auxiliary spillways with less stringent technical features since its use is not as frequent as the main structure. Of course this requires that in any case the overall safety of the project is not affected, but it may require continuous inspections on its performance. Some examples illustrate this possibility:

In the Itá Project in Brazil, with two spillways, the total capacity is 52 800 m^3/s. The 275-m long chute of the auxiliary spillway was lined only over the initial 120 m (capacity 20 000 m^3/s, specific discharge 234 m^3/s/m). The key element supporting the decision of using this cost reducing feature was the accessibility to the area and the possibility of remedial work if and when necessary. In fact initial tests carried out on this structure showed that erosion risks were higher than anticipated and the concrete slab was extended 55 m to cover the affected area as shown in Fig. 2.4 (Andrzejewski, 2002).



(a) *Initial lining extension* (b) *Extended lining after erosion tests*

Fig. 2.4 – Auxiliary spillway of the Itá Project (Brazil)

Another example is the Xingó Project, also in Brazil. The project has two parallel spillways for a combined capacity of 33 000 m^3/s. The auxiliary spillway 252-m long chute, designed for a maximum flow of 15 500 m^3/s has a lined concrete slab only along 90 m. This spillway has operated in test conditions up to 4 000 m^3/s. Some localised erosion was observed and backfilled with concrete. After the tests the chute was approved for operation in view of the fact that if higher flows cause additional erosion, fully access to eventual repair will be available (Eigenheer *et al*., 2002).

These two examples show that the use of an auxiliary spillway for infrequent operation with some flexibility in their technical standards can provide initial cost economies but must secure safe operation and this is dependent on continuous inspection and available access to repair eventual damages.

The use of an auxiliary spillway for discharge part of the total design flood should not be confused with the provision of an emergency spillway. This structure is recommended when the confidence on the basic hydrological data and studies, as indicated before, is limited in extension or quality. In that case emergency spillway provisions or other type of facility to retain or divert the excessive flood may be included into the project. This question is also discussed in ICOLD Bulletin 142 (ICOLD, 2012) where there are considerations and recommendations concerning alternative means of handling floods larger than the spillway design flood.

2.3. CONTROL STRUCTURES

2.3.1. Surface spillways

Spillways designed to pass large flows will generally be surface spillways mainly because of the magnitude of the flood to be released. However, low level bottom outlets have been used as well, especially when the structure was also used for river diversion and/or if discharge of accumulated sediment in the reservoir was required.

Most of the large spillways built for hydroelectric or water supply projects are gate controlled because otherwise the length of the spillway and the loss of useful head would impair the benefits of the project. However, the use of gated spillways has sometimes generated concerns about the possibility of gate failure or mal operation in critical circumstances and creating conditions for dam overtopping and collapse. Clearly ungated spillways eliminate this risk.

Whatever the purpose of the project, it is not easy to fit physically and economically a total ungated spillway to discharge very large flows in a project site. Nevertheless, this has been done in some projects where it was possible to use most of the site width for the spillway, locating the water intake or other project facilities in a place not interfering with the spillway. Where the physical possibility exists, the balance between economics and risk assessment is the key decision factor. Fig. 2.5 shows a view of the Burdekin dam, in Australia, designed for a flow of 64 600 m^3/s with a maximum head over the spillway crest of 17 m. If this were a gated spillway it would be possible to have the operation level of the reservoir 17 m higher than in the actual ungated solution. This indicates the economic limitation of large ungated spillways for hydroelectric projects. Additional data on other Australian large ungated spillways are depicted in Table 2.2.

Fig. 2.5 – The 64 600 m^3/s ungated spillway of the Burdekin Dam, Australia

Probably most of the modern gated spillways designed to discharge large flows are equipped with large radial or tainter gates. Large vertical lift gates can and are, of course being used but they require a large overhead structure to allow the raising operation and also require lateral slots that cause some disturbance of the issuing flow. These gates are normally operated with cable hoists because if hydraulic cylinders would be used, the height of the overhead structure would have to double. On the other hand, vertical-lift gates suspended by cables are prone to generate serious vibration problems when the gate is lowered in high velocity flow.

The configuration of the control structure for surface spillways for both types of gates is essentially the same for small and large structures. The orientation of the spillway axis in relation to the approaching flow for large spillways will quite often require hydraulic model studies to avoid flow disturbances and unequal discharge in different passes.

A combination of gated spillways and outlets with ungated or emergency spillways to avoid or minimize the gate failure risk is sometimes used. This is also not easy to do for projects involving very large flood flows. A recent study for the Inga Project in the Congo River in Africa, where the design flood is 60 000 m^3/s, proposes a combination of a conventional surface gated spillway with a PK labyrinth spillway (see chapter 6) to deal with such a large discharge (Lempérière *et al.*, 2012). The combination of PK spillways and gated bottom outlets has already been used for projects with smaller flood flows.

Radial Gates

The use of radial gates for controlling large capacity spillways is an established trend. Very large radial gates up to more than 440 square metres in area were installed and are operating successfully in some Brazilian surface spillways. Gates twenty metres wide by more than twenty metres high were used in Itaipu, Tucurui, Estreito, Santo Antonio and Jirau projects, in Brazil as indicated in Table 2.1. This will cause, of course, higher values for the specific discharge in the downstream spillway channel but designs had cope successfully with this.

In general, the main reason to use such large gates is to achieve a maximum reduction of the dam length. In many very low head projects built in large rivers, there is a limitation on the power output of the generating units, which means increasing the number of units to meet the total available hydraulic power, and therefore the length of the power house becomes significant, leaving a reduced space for the spillway. The spillway design floods in these projects are also very large and the result is the need of saving space by using large gates. As an example, the Jirau Project on the Madeira River, in Brazil (3 750 MW), has 50 generating units operating under a maximum gross head of 15.7 m, and a spillway design flow of 82 600 m^3/s with 18 radial gates, 20.0 m wide by 22.8 m high (Fig 2.6). The spillway has a length of 445.0 m and forced the powerhouse to be divided into two parts located on each bank of the river.

In the Yaciretá-Apipé Project, between Argentina and Paraguay, the power-plant houses 20 units and is 816 m long, while the spillway design flood of 95 000 m^3/s had to be divided into two independent structures (Fig.2.7) one with 18 radial gates 15.0 m wide by 19.5 m height and the other with 16 gates 15.0 m wide by 15.5 m high.

Fig. 2.6 - 82 600 m^3/s Jirau Project Spillway under construction – Madeira River, Brazil

Fig. 2.7 - Yaciretá-Apipé Project – 816 m long powerhouse on the Paraná River next to the 55 000 m^3/s main spillway - Argentina/Paraguay Border

Although radial gates are generally considered to be very reliable and safe equipment, there have been some events in which radial gates failed causing at least concerns in the wide use of this type of equipment. A general discussion of issues related to gates in general was the subject of Question 79 of ICOLD Twentieth Congress. The General Report for this question (Cassidy, 2000) pointed out failure of gates caused by design deficiencies related to earthquakes, flow induced vibrations and fabrication and erection errors. Considering the size of gates and the importance of flows in large spillways, special attention to these issues by designers and owners is very important.

It is not, of course, the object of this report to treat extensively the issue of gate operation and safety but to call the attention of designers and owners of large spillways to the need of considering in the design and operation of these structures, details that many times are not included in the conception of large projects.

As an illustration of the kind of problems that may occur with radial gates, the following five cases indicate the diversity of situations that can happen:

- Folson Dam Spillway, in the United States - One of its 8 spillway radial gates failed in 1995. The gate was 12.8 m wide by 15.2 m high, and failed when being raised with the reservoir practically full (Todd, 1997). Investigations examined the possibility of vibrations affecting the gate structure but concluded that the gate failed because of friction of the trunnion causing the failure of a lower strut transmitting the water load to the trunnion. It was discovered that the friction force in the trunnion was not considered in the structural design of the gate and besides that the reduced frequency of lubrication and lack of weather protection resulted in corrosion which increased the friction over time. The failure of the gate with reservoir full released 1 132 m^3/s.

- Itaipu Dam Spillway, in Brazil (Lima da Silva *et al.,* 2000) – The rod of the one hydraulic hoist cylinder used to move one of the gates broke in July 1994 after 12 years of safe operation. There are 14 gates at Itaipu, each 20.0 m wide by 21.4 m high, each operated by two cylinders. The rod brakeage occurred while lowering the gate during a programmed maintenance operation with no water flowing in the passage. Investigations of the failed equipment concluded that the failure originated from a transverse crack associated with a corrosion process and triggered by the vibration originated in the friction of the side seals of the gate to the lateral walls without water. After the rod failure, the broken cylinder was discarded and replaced by a new one and all the 27 remaining hydraulic cylinders were dismantled, machined and polished to remove cracks and corrosion. A detailed programme of maintenance was then formulated and implemented.
- Salto Osório Dam Spillway, in Brazil – One of the 9 radial gates of this project, each 15.3 m wide by 20.77 m high, failed in September 2011, after 37 years of safe operation. The gate movement was carried out by cables moved by winches and automatically controlled by switch relays. A command for closure was issued but a failure of the limit switch relay did not stop the hoist and continue to order the turning of the winch drum, winding the cable in reverse direction in the hoist pulley, and fully opening the gate. The winding occurred in more than one layer and cause rubbing the cable against the concrete, severing the cable by friction. The gate closed violently by gravity and was then carried by the flow with the bearings being ripped off the concrete piers. Two thousand cubic metres per second were released, fortunately without any specific consequence. The reservoir level was lowered, a stop-log placed in the passage and a new gate installed.
- Shiroro Dam Spillway, in Nigeria (Epko and Adegunwa, 2011) – This project, commissioned in 1984 has a spillway with 4 radial gates each 15.0 m wide by 16.85 m high, designed for 7 500 m^3/s. The gates are moved with hydraulic hoists. In spite of the existence of slots for stop-logs, no stop-logs were provided. In 2005 maintenance actions were carried out including replacement of the seals. However gate No.4 could not be opened more than ¼ of its course. Investigation at this time detect that the top right-hand pier had moved and closed the width at the top by 57 mm, at half height by 46 mm and at the base by 9 mm. This prevented the full opening of the gate and of course limited the capacity of discharging the spillway design flood. Reports indicate that so far (2011) the reason for this problem has not yet been found, although it is probably related to alkali-aggregate reaction. No solution has been proposed or implemented so far.
- Tarbela Spillway, in Pakistan (Khalio Khan and A-Siddiqui, 1994) – One of the 7 gates of the service spillway of this project, each 15.2 m wide by 18.6 m high, failed after 17 years of problem-free operation. The accident was described as the gate getting stuck during a lowering operation and then falling down breaking two hoist ropes and damaging the hoist deck. The event happened when the gates were open with the spillway discharging 2 475 m^3/s and the operator started the closing operation of the extreme right gate. As reported, the motor of this gate hoist device tripped and the gate, after falling from an undetermined height, got stuck leaving an opening of 112 mm from the bottom sill resulting in high velocity discharge. After detailed investigation of

the accident, it was concluded that the gate got stuck because insufficient clearances between the side sealing plates on the pier and the clamping bar of rubber seal on the gate. The original clearance was 17 mm and over the years was reduced to as little as 2 mm. The investigation of the cause of this reduction concluded that it was the combination of the gate thermal expansion and slippage of the seal holding device due to the loosening of the bolts along the years of operation. The gate fell down by its weight when freed by hydraulic vibrations after cooling of the skin plate by the evening lower temperatures.

In spite of such problems, the percentage of radial gates that suffered failure is relatively small. The US Bureau of Reclamation, which is the owner of Folson Dam inform that they have 314 radial gates in their works with 18 000 gate-years of operation and only one gate (Folson) failed (USBR, 2011). The US Army Corps of Engineers have 90 radial gates in various projects and although some technical incidents and design inadequacies have been found, there were no major failures in any of them (Ebner and Craig, 2012). In Brazil, there are 330 radial gates installed in spillway projects whose capacity is larger than 20 000 m^3/s and besides the Itaipu and Salto Osório gate problems mentioned above, there was only one other problem in one gate of the Furnas project, which was related to crystalline corrosion of the steel rods anchoring the trunnion bean. This problem had no consequences on the gate operation.

Very important feature of a spillway controlled by radial gates are the provisions for the use of stop-logs in all gate bays and of course their availability. The cases listed above illustrate the important role of stop-logs to overcome an accident with the gate, but they are also very important to allow maintenance operation of the gate in the dry. As mentioned in the general report of ICOLD Question 79 (Cassidy, 2000) the lack of stop-logs in Folson dam, created a very difficult problem to correct the gate failure accident. Other example of problems due to unavailability of stop-logs is the recovering of a gate in La Villita dam in Mexico, damaged by the sudden failure of the gate steel reinforcement.

Present practice for large gates indicates that gate movers based on servo-motors (hydraulic cylinders) rather than cable or chains have an increased reliability. Many modern large radial gates are designed in this way, with two cylinders per gate. As in Itaipu considerations that only one of them would be able to close or open the gate is a sound criterion.

The available registers of incidents and accidents involving radial gates indicate that in a significant number of cases, faulty operation of the gate, and not a problem with the gate proper, was the primary cause. These include failure in the electricity supply to move the gates and difficulties in access to the spillway area under emergency conditions. This kind of problems (for radial and vertical-lift gates) occurring during heavy rains and flooding conditions, were the cause of very well known dam collapsing cases by overtopping (among which, Euclides da Cunha Dam in Brazil, Tous Dam in Spain and Belci Dam in Bulgaria). In large spillways, designed to control very large flows, it is of utmost importance to build redundant facilities in electric supply and alternative means of unrestricted access to gates even under heavy rain or flood conditions.

It has also been suggested, and sometimes applied, the so-called N-1 criterion in which the number of gated passages of the spillway is determined so that the total spillway design flood can be discharged by all gates except one. This rule does not have a universal consensus mainly because of its economic impact. However one rational approach to the issue would be related to the size of the incoming flood and the speed of increase of the reservoir level. For the very large reservoirs generally associated with large spillway projects the raising of the reservoir level caused by the jamming of one gate is normally slow and takes one or more days before it reaches a critical level. During this time it should be possible to reach the jammed gate and manually force its opening. Of course this implies that access to the damaged gate is available at all times. In such conditions there would be no need to provide an additional gate bay to the spillway. However, for reservoirs with small areas in which large floods would cause a rapid increase in the reservoir level elevation, the application of the N-1 criterion seems to be a justifiable approach if no other economically alternative emergency discharge feature can be provided.

2.3.2. Bottom and intermediate level spillways

The exclusive use of intermediate level orifice spillways and/or bottom outlets to discharge very large flows is relatively limited mainly because of the limitation in capacity of the maximum individual passages, where the flow is proportional to the square root of the head, as compared to surface weirs where the flow varies with the 1.5 power of the head. This of course requires a larger head in orifice spillways to offset the flow capacity difference. However, in many projects, a large head orifice is necessary anyway to allow a wider control of the reservoir level which may be convenient for project operation and also for the control of flood routing in the reservoir.

Large bottom outlets are normally used when the flushing of sediment is required or when the river diversion during construction requires a low level passage which can be more easily controlled during river closure and reservoir impounding. The exclusive use of bottom outlets for discharging large floods is limited to dams of small or medium height, and generally where the combination of spillway function with sediment disposal or diversion control is required or convenient.

However in many projects with concrete dams a combination of bottom and orifice outlets or with surface spillways is rather common. The Ertan Project, in China, mentioned above and the Three Gorges Project in China are significant examples of the use in large rivers of permanent and provisional (used for river diversion) orifices and bottom outlets.

Figure 2.8 depicts a cross section of the spillway block of the Three Gorges Dam, which incorporates a very large flood discharge system able to pass a maximum flood of 102 500 m^3/s with the reservoir at the check flood level at El. 180.4 m. As shown in the figure it contains two levels of permanent discharging facilities (surface [7] and a deep orifice outlet [8]) in addition to a bottom diversion outlet [9] used during construction and lately plugged.

The surface spillway is formed by 22 surface vertical lift gates, 8 m wide, with a sill level at El. 158. The deep orifice outlet ([8], in the figure) comprises 23 passages controlled by radial gates, with intake at El.90, each 7 m wide by 9 m high. The designed single bottom diversion outlet capacity is 2 117 m^3/s, operating under 85 m of head, and produces a specific discharge of 302 $m^3/s/m$. The total designed discharge capacity of the deep orifice outlet is 48 691 m^3/s, with reservoir level at El.175. This is one of the largest intermediate orifice outlet facility and is clearly the result of fitting the project layout to the various needs of the undertaking, both during and after construction.

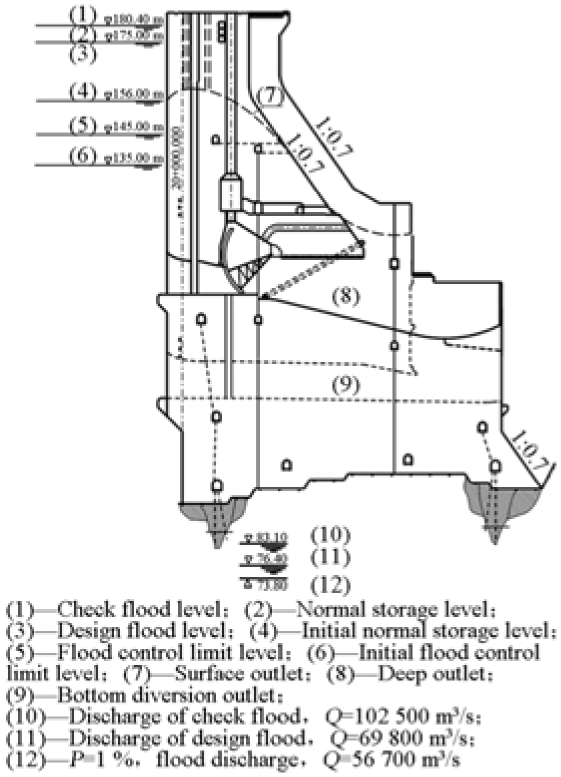


Fig. 2.8 -Three Gorges Project – Cross section of the spillway block (Wang *et al*, 2011)

Very large spillways have also being built in India. Representative of these structures is the Chamera I Project spillway designed for 26 500 m^3/s (Fig.2.9), which contains 8 orifice outlets 10.0 m wide by 12.8 m high, controlled by radial gates and 4 bottom outlets 4.0 m wide by 5.4 m high controlled by sluice gates.

Fig. 2.9 – Orifice and bottom outlets of the spillway of Chamera I Project, in India

Another large bottom outlet spillway is the on Jupiá Project, in the Paraná River, in Brazil (Fig. 2.10). As indicated in Table 1, this project contains four surface spillway passages and 37 bottom spillway passages, each 10.00 m in width by 7.61 m height, with a nominal capacity of 44 000 m^3/s. The reason for this kind of solution was to facilitate the control of the river during construction and the impounding of the reservoir.

Fig. 2.10 - Jupiá Project – 44 000 m^3/s bottom outlet spillway during construction (1967)

The bottom outlet passages in this project are controlled by radial gates. The project was commissioned in 1968 and during 43 years of operation its performance has been good, with localised problems of erosion particularly in the downstream part of the floor slab. The maximum daily flow recorded since 1968 was 28 943 m^3/s but about 20% of this flow was discharged by the four surface spillway passages.

The hydraulic design of bottom outlets is extensively treated in the technical literature, both in their general terms and for specific cases. It is however appropriate to stress the importance of paying thorough attention to the risk of cavitation and the need of providing adequate air supply to the region immediately downstream of the controlling gate.

Bottom and intermediate dam outlets can also be provided through tunnels. Specific references to these works are given in chapter 7 of this bulletin. In many cases these outlets are built in tunnels that were used for river diversion during construction. In these cases special attention should be paid to the schedule of the overall works, since transformation from a provisional construction feature into a permanent outfit of the project after its use as a river diversion feature, may incur time delays and economic impacts.

2.4. CONVEYANCE STRUCTURES

The return of the flow to the river depends on the type and height of the dam, the form of the valley and its geological configuration. In general, for medium-height dams, very large spillways discharging large flows will be surface spillways, returning the flow to the river through a short steep chute or long chute channels. In both cases the chutes normally end either in a flip bucket, if the height of the dam allows it, or through a hydraulic jump or roller-bucket stilling basins. Very low dams will normally have a hydraulic jump stilling basin immediately following the crest structure. As a rule, underground tunnel chutes, as a single outflow facility, will not fit the flow magnitude of large spillways.

As discussed above, large spillways will quite often be gated structures resulting in a rather high specific discharge for the issuing flow. This affects the design criteria used for the dissipation of the energy of the issuing flow.

2.4.1. Chute Channels

In projects with high embankment dams built in valleys with narrow to moderate widths, long chute spillways built in the dam abutments, ending in a ski jump and discharging over a plunge pool, is the rather standard solution both for small or large spillways. There are some examples of ungated chute spillways built over embankment dams, but only for small design floods, among which Crotty Dam, in Australia, Ahning Dam, in Malaysia (Cooke, 1985) and Tongbai Dam, in China (Zhao and Yuyan, 2006).

When the valley width is wide enough, a concrete stretch of the dam to accommodate the spillway, with a steep short chute and flip bucket, can be designed, as in the Tucuruí Project (Fig. 2.2). In these cases a hydraulic jump stilling basin could also be provided as in Sardar-Sarovar Project (Fig.2.12) overflow concrete dam. This 163 m high dam has a very large spillway system including a service spillway with 23 surface radial gates, 16.78 m wide by 18,30 m high, an auxiliary spillway with 7 surface radial gates 18.30 m wide by 18.30 m high and 4 bottom outlets 2.4 m wide by 3.6 m high. The dam also had 10 bottom sluices 2.15 m wide by 2.75 m high used for construction and ultimately closed.

Fig. 2.12 – Sardar-Sarovar 87 000 m^3/s spillway in India (piers and radial gates not yet installed)

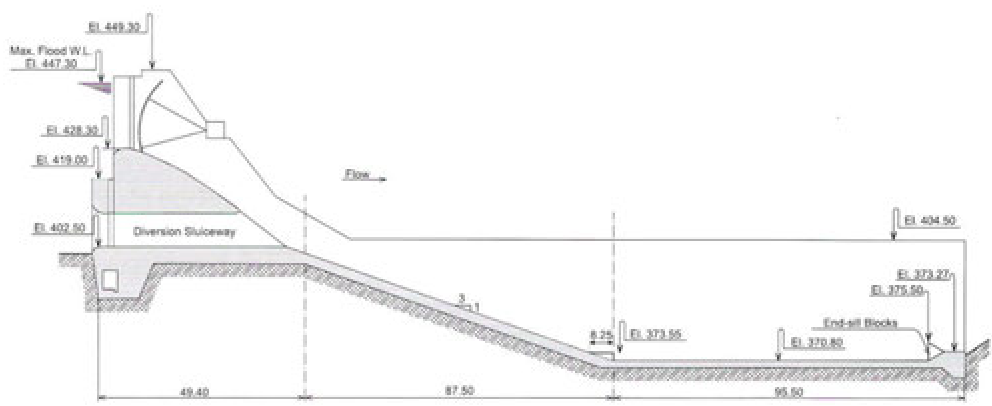
The design of a long chute spillway in an embankment dam abutment depends on the magnitude of the flood and on the topography and geology of the site. For large spillways this scheme often requires very large excavations which have to be wasted or may be used as a source of material for the embankment. However, sometimes the balance of quantities or the planning schedule of the project or even the geotechnical stability of excavation pits, create difficulties to fit a single very large spillway in one abutment. This may lead to the division of the spillway in two or more separate structures, a service and auxiliary spillways, a solution that may have already been envisaged by other reasons, as indicated before.

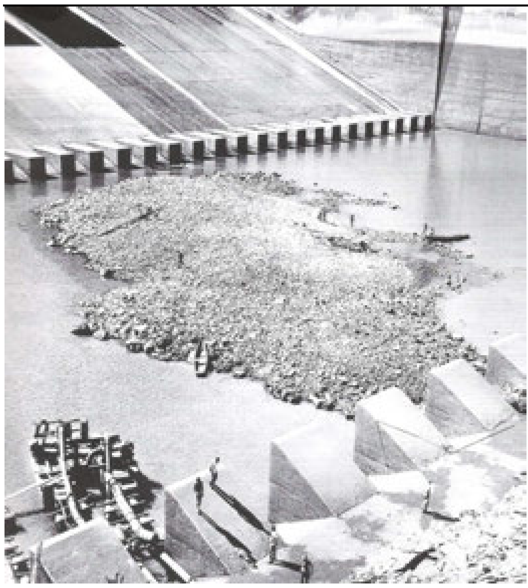
The location and the profile of the chute will, for these reasons, be designed to minimise excavations. In plan, the best hydraulic solution is to have a straight chute with constant width although converging chutes are also common. It is important to locate the flip bucket of the ski jump above the maximum tail-water level and in such way that the river swirling flow does not affect the downstream toe of the dam. It is often convenient and most of the time necessary to carry out hydraulic model investigations to define the final design of this feature, and specially the geometry and performance of the plunge pool.

2.4.2. Hydraulic jump stilling basins

Hydraulic jump stilling basins are necessarily used for very low dams but can also be used in higher dams, as indicated above. For medium-high dams there is no universally accepted criteria for selecting a stilling basin solution against flip bucket and plunge pool. Economic considerations, local geological setting and the usual practice of different countries, usually define the type of solution.

For large spillways with large specific discharges, the basin design must consider the possibility of downstream erosion caused by the occurrence of the jump outside the basin and the asymmetrical operation of the gates causing swirling currents depositing rocks and debris into the basin causing abrasion of the concrete. This is a rather common problem with stilling basins. One illustrative example of this kind of damage occurred in the stilling basin of the 21 400 m^3/s Marimbondo Project spillway, (Carvalho, 2002), in Brazil (Fig 2.13).





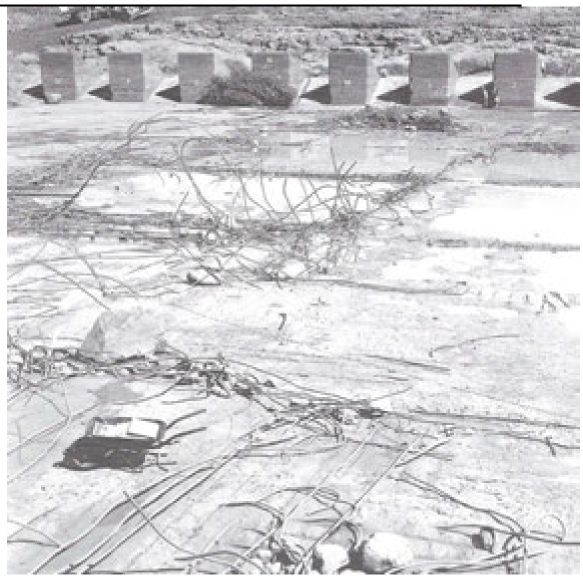
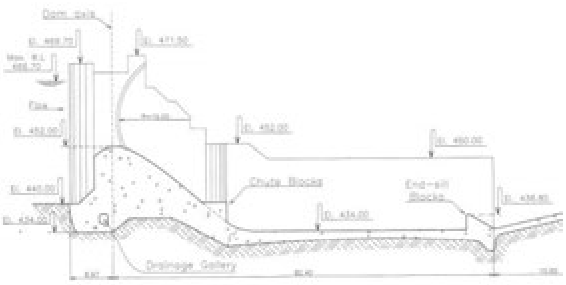


Fig. 2.13 –Marimbondo Spillway stilling basin damages from asymmetric operation

(a) Spillway profile
(b) Solid material deposited in the basin due to asymmetrical gate openings
(c) Damaged steelwork in basin

The use of chute or baffles blocks, as it is illustrated in the Marimbondo Project, is not recommended specially for basins with high specific discharge flows, since they are prone to produce serious cavitation damages. An illustrative example of this fact occurred in the Porto Colombia Project, also in Brazil. The 16 000 m^3/s spillway was originally built with a chute and end-sill baffle blocks, as illustrated in Fig. 2.14 (a). After 10 years in operation, very serious cavitation damage occurred adjacent to the chute blocks and to a lesser degree on the end-sill blocks. To correct these problems the blocks were removed and the end-sill made continuous, as also shown on Fig. 2.14. After these measures, no further damages were observed (Carvalho, 2002a).



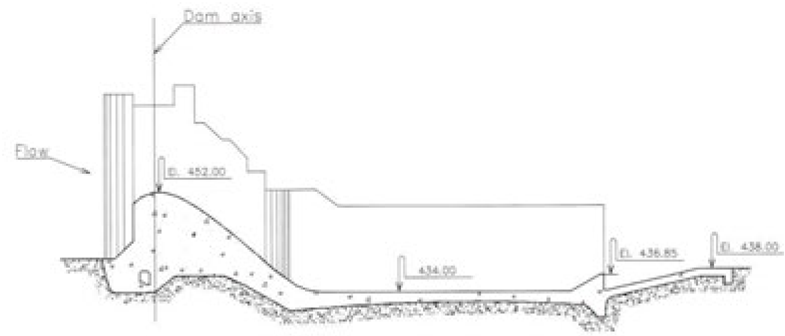


Fig. 2.14 - Porto Colombia spillway: cavitation damage and repairs
(a) *Original spillway profile*
(b) *Cavitation damage downstream of chute blocks*
(c) *Refurbished spillway profile without chute and end-sill blocks*

Besides these types of problems, the integrity of the basin may also be affected by fluctuating uplift forces whose peaks overcome the weight of the concrete slab and the resistance of the steel anchors that are provided to incorporate the weight of the underlying rock. Examples of serious accidents caused by these fluctuating forces are Malpaso, Mexico (Sanches-Bribiesca and Viscaino, 1973) and Karnafully, Bangladesh (Bowers and Toso, 1988). The effect of these fluctuating uplift forces results from the transfer of dynamic pressures generated by the turbulent hydraulic jump to the underside of the basin slabs, through joints and drainage system pipes. A comprehensive summary of present knowledge of the problem and recommendations for design procedures can be found in Bollaert, (2009).

In many cases seeking economy, designs consider the occurrence of the jump within the basin only for frequent floods, allowing it to happen downstream for very large and infrequent floods. Of course this should only be allowed if the downstream rock is considered sound and resistant to erosion caused by such non frequent floods.

2.4.3. Aerator facilities

Aerator facilities to prevent cavitation damages to the chute and flip bucket are often used when the average flow velocity reaches about 30 m/s which corresponds roughly to a cavitation index of 0.25. This of course applies to spillways small and large and is dependent on the velocity and not on flow.

It is not the purpose of this bulletin to discuss cavitation and the applicability of aeration facilities, since this question is not strictly associated only with large spillways. A comprehensive treatment of the subject can be found in ICOLD Bulletin 81 (ICOLD, 1992) and in Falvey, (1990).

2.4.4. Energy dissipation and downstream erosion

The dissipation of the energy from the flow issuing from a spillway and the control of the downstream erosion that may be caused by the remaining energy of the flow, are major issues in the design and operation of large spillways. This is especially true due to the normally high values of the specific discharge of the issuing flows. It can be seen in the data shown in the Tables presented at the end of this chapter, and from examples mentioned above, that values of the specific discharge above 150 m^3/s/m and up to more than 300 m^3/s/m, are rather frequent in large spillways outflows. In general, high specific discharge in large spillways result from the need of reducing the gated surface spillway length, as discussed before, or in dams built in narrow valleys with surface and bottom outlets.

The main common types of energy dissipaters are the hydraulic jump / roller bucket stilling basin, the ski jump and plunge pool arrangement, and the free fall jet into a plunge pool.

In many cases where the evaluation of the quality of the rock allows it, instead of a stilling basin as such, there is simply a concrete slab leading to a bare rock channel. This solution has been used in Brazil in auxiliary spillways in some projects, as mentioned before for the Itá (Fig. 2.4) and Xingó Projects, while in Canada there are a good number of projects in which the single main spillway follows this trend. It is of course an economic solution where the rock is sound, but in any case some erosion damages can occur and it is necessary to have access to the damaged area and a flow regime with time intervals to allow eventually needed repairs. An interesting example of this case is the LG-2 spillway, part of the James Bay Project, in Canada, which has a spillway for 17 500 m^3/s discharging in a bare rock channel excavated in steps (Fig. 2.15). Although in operation since September 1979, its maximum discharge along 30 years has been only 3 500 m^3/s (Nzakimuena and Zulfiquar, 1999). Nevertheless there has been significant scour immediately downstream of the protecting upstream slab.

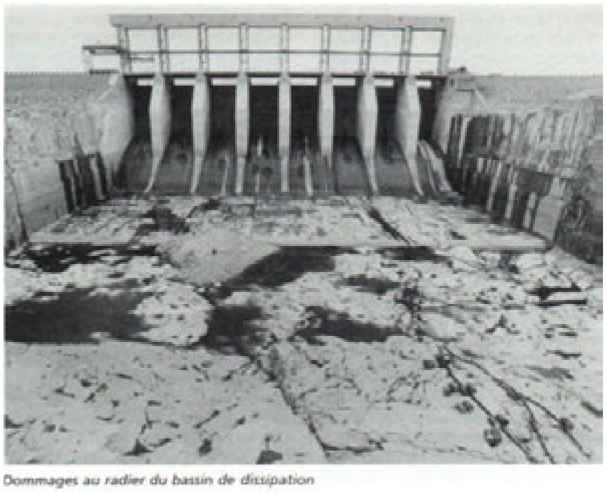


Fig. 2.15 – LG-2 spillway discharging in a bare rock channel and
erosion damages downstream from the control structure.

The issuing of a high specific discharge flow from a stilling basin may create erosion by dislodging and displacing pieces of fractured rock, some of very large size, and creating scour holes. It is very difficult to evaluate the magnitude and extension of this erosion process, which is different from the erosion caused by a plunging jet, which is also difficult to foresee but nevertheless has been better studied. Hydraulic model studies to infer the possibility and extension of scour holes are often necessary together with a careful evaluation of the geological condition of the site. It is of course very important to prevent the evolution of the scour hole into the foundation of the stilling basin. In the Macagua Project 30 000 m^3/s spillway, in Venezuela, the originally designed concrete slab downstream from the control structure, had to be extended 40 m after deep erosion occurred next to the stilling basin (Marcano, 2009). In many cases, especially in very low dams, after the start of operation of the project this additional construction work may be difficult and expensive, because it requires coffer-damming the area where the slab will be extended. It is therefore prudent, in these cases, to be conservative in the original dimensioning of the extension of the stilling basin slab.

For higher embankment and gravity concrete dams, a ski jump at the end of the spillway channel is the normal rule. The flow is discharged from the ski jump creating a jet that impacts on the river bed thereby producing a scour hole, which assists in the process of energy dissipation. The prediction of the scour evolution and the control of the scour hole dimensions is one of the most difficult tasks of the designer of large spillways with high specific discharge, because of the costs and risks associated with it. Chapter 3 of the present bulletin deals specifically with this subject and covers the various theoretical and practical aspects of this question.

Among the issues generally present in most cases of large spillways is the need of a pre-excavated plunge pool if the depth of the downstream natural level is considered to be insufficient to dissipate the issuing flow energy and the project does not allow the construction of a tail pond dam. For large gated spillways with high specific discharges, the amount of energy to be dissipated is substantial.

The determination of the need of a pre-excavated plunge pool and especially its adequate stable dimensions is based on the estimated depth and lateral expansion of the scour hole due to jet action. As also discussed in Chapter 3, this issue has been the object of many studies and researches, with both empirical and physical approaches. The development of the jet erosion scour must avoid the risk of affecting the foundation of the permanent structures and the stability of the valley slopes, but the difficulties of anticipating the length and width of the scour pit and establishing the horizontal dimensioning of the plunge pool, especially in the case of large spillways with high specific discharge values, lead to a defensive and conservative approach. However, as pointed out in the following chapter of this bulletin, to use the maximum spillway flow for this seems to be too conservative, and recommendations are to using 50% of the spillway design flow for that.

As of today, in current practice, probably most cases of estimating scour depths are determined by empirical formulae. The computation with these empirical formulae may give an approximation of the real depth of a scour pit associated with a given flow for the project and provide information for the location of outlet structures and the design of pre-excavated plunge pools.

An interesting example illustrating the approximations and anticipating the depth and extension of jet erosion in a large spillway, is shown in Fig. 2.15 (Sucharov and Fiorini, 2002) produced as part of the analysis of the Itaipu spillway erosion. The data reflects the field information surveyed in 1988, six years after the exceptional floods of 1982. During this whole period the spillway discharged continuously all the incoming flow since the power generating units were still being assembled. The maximum discharge was 40 000 m^3/s roughly equal to the 500-yr flood. In the graph of Fig.2.16 the model and prototype data are shown in comparison with projections based on Veronese empirical formula – which was used for the prediction of erosion depth – with different values of the formula K coefficient, and data from other projects.

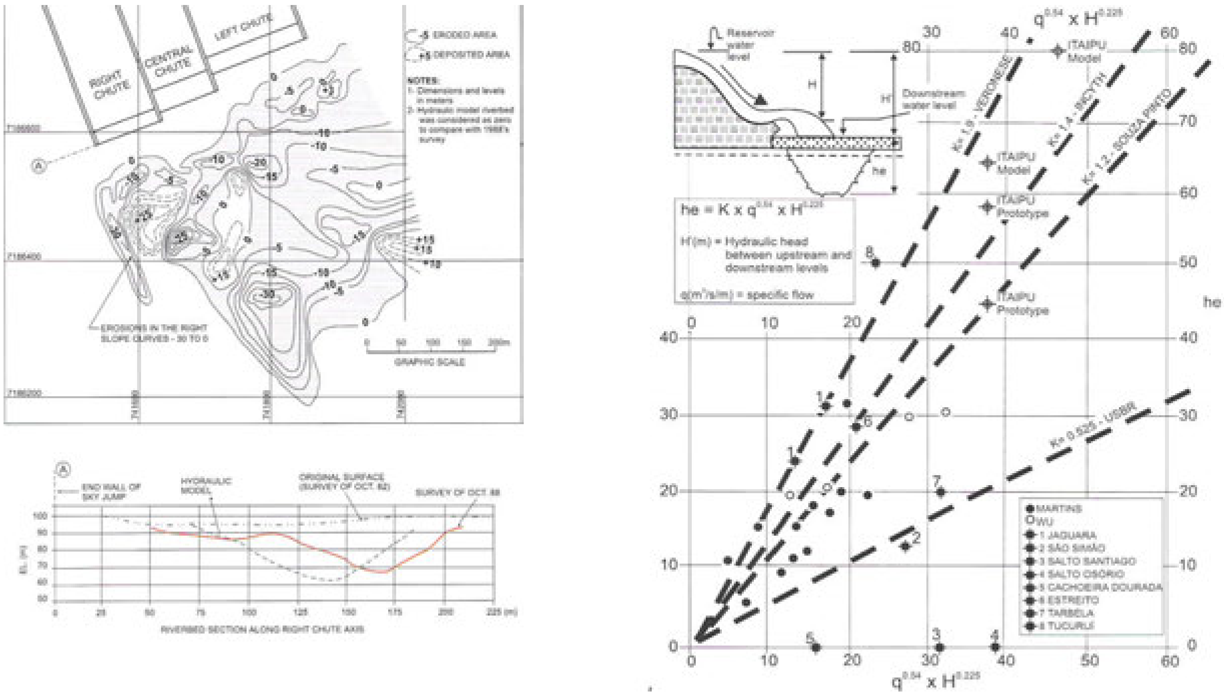


Fig. 2.16 – Itaipu Spillway erosion data. (Sucharov & Fiorini, 2002)

One important and controversial issue which substantially affects the cost and schedule of a project is related to the need of lining the plunge pool. If the rock is sound and free of fractures the general practice has been to leave the excavated plunge pool unlined. Plunge pools at the Tucuruí and Itaipu Projects, mentioned before, were unlined with good results. However in order to prevent the evolution of the scour to the permanent structures, concrete lining and or rock bolting and anchoring have been considered necessary in some projects. As discussed in Chapter 3 of this bulletin, the requirements to obtain a problem-free lining are very stringent, difficult and expensive. However in some projects plunge pools have been totally lined with concrete slabs anchored to the rock, as shown in Fig.2.17, for the 15 000 m^3/s Karun III spillway, in Iran, to prevent any detrimental consequences on the high arch dam. In this case the plunge pool lining required 250 000 m^3 of reinforced concrete and 135 000 m of rock bolting, anchoring and doweling (IWPC, 2004).

Left: Plunge pool lining under construction
Above: View of the completed works

Fig. 2.17 – Karun III spillway (15 000 m^3/s) concrete lined plunge-pool

2.5. OPERATIONAL ISSUES AND RISK MANAGEMENT

Very large spillways are associated with large rivers and with large floods. As discussed above these spillways will normally be gated structures and, as such, require strict operation rules to avoid incidents and accidents related to the passage of extreme floods over the dam barrier. This of course is true for any size of gated spillway, but the magnitude of the damage caused by operational failure of a large spillway can have much more serious consequences.

The technical literature and specially ICOLD documents – bulletins, reports and papers presented in congresses and workshops – have dealt extensively with this subject. Bulletins 49 (1986) and 142 (2012) and the general reports of ICOLD Questions 71 (Pinto, 1994) and 79 (Cassidy, 2000) present information and comprehensive summaries of operational issues of spillways.

In most cases, and specially, in projects incorporating very large spillways, the project owner produces an Operation & Maintenance Manual that includes, for a specific project, rules and directives to be followed by operators in normal and in emergency situations. It is of course very important that these rules be enforced and that the operating staffs be trained and qualified to carry out the corresponding activities.

The proper operation of gates in large spillways is the key issue to prevent the risk of dam being overtopped and eventually breached or destroyed. Some of the possible problems with gates have been mentioned before in this document but in general the causes of malfunctioning can be classified as follows:

- failure of the operator to issue the commanding order to open the gates on time;
- failure of the gates to move as commanded or the supporting systems to provide the power or the commanding signals to correctly move the gate; and
- clogging of the passage due to large size debris brought by the flood.

However the evaluation of the true risk of dam overtopping in gated spillway projects, is more complex. For instance, the commonly used Dam Safety concept of Inflow Design Flood (IDF) does not address possibility of ''operational flood'' in which a dam could be overtopped and fail due to a combination of a flood that is much smaller than the IDF and one or more operational faults (e.g. spillway gate failure). The number of possible combinations of unfavourable events causing such a failure is very large and increases with the complexity of the dam or system of dams. Micovic *et al*. (2015) presented results of stochastic flood simulation for a system of three dams and reservoirs in Western Canada, where the focus of flood hazard analysis was shifted from reservoir inflows to corresponding reservoir levels. The authors derived the full probability domain of the peak reservoir level as the combination of probabilities of all factors leading to it, including reservoir inflows, initial reservoir level, reservoir(s) operating rules and spillway gate failures. The results showed that dams were much more likely to be overtopped (and consequently fail) due to a combination of otherwise somewhat common individual events than due to a single extreme flood event such as the IDF. Micovic et al. (2015) also concluded that the importance of spillway gate reliability increases as the size of available surcharge storage decreases. For example, in the case of a small reservoir surcharge storage, simulating possibility of spillway gate failures during the flood increased the resulting probability of dam overtopping by five orders of magnitude over the case in which all spillway gates are assumed operable.

A statistical survey cited by Hinks and Charles (2004) based on Foster *et al* (2000), indicates that 46% of embankment dam failures were due to overtopping, and among these 13% were associated with spillway gate operation. Although there seems to be no figure to indicate which percentage is related to human errors in the operation of gates, this factor is generally recognised as being very important. The risk associated with human errors is generally considered to be possible to mitigate by training of operators and by simple, clear and unambiguous operating rules. There are anyway recommendations that a team of two people be assigned the task of operating the gate system for occupational safety reasons (Barker et al, 2006). Irrespectively of the type of error, in a flood occurring situation it is obvious that adverse physical and psychological conditions may affect the correct action of operators.

Besides the need of comprehensive and extensive training of operators, there are practical recommendations to minimise the risk of operation errors and deficiencies. Among these are the need of avoid overstressing of operators due to the occurrence of failure in operating equipment, and to ensure unobstructed access to gates and reliable communications when an emergency condition happens. To prevent that, the provision of redundancy in equipment, access and communication is mandatory. A systematic inspection of these operational procedures, in which the operators had been included, is also very important (Bister, 2000). ICOLD Bulletin 154 (2013) discusses in more general terms practical measures to deal with operation failures and mitigation measures.

The risk of failure to move the gates or to follow command instructions due to mechanical and electrical problems, were illustrated by the examples mentioned before in this chapter. Redundancy, as mentioned above and specific design solutions may help to minimise this issue. However, the provision of unobstructed access to the gates and the assurance of electric supply to operate them even under extreme flood condition, is very important specially in large gated spillways.

There are examples of complete dam failures which could have possibly been avoided if these conditions existed when the accident occurred. In the Euclides da Cunha Dam accident, in the Pardo River in Brazil, in January 1977, a spillway with 2 surface gates designed for 2 040 m^3/s and one bottom outlet for 300 m^3/s, were not operated properly during a sudden flood peaking 3 670 m^3/s which generated an incoming flood wave in the reservoir of approximately 2.000 m^3/s. This happened because the operator lost communication with the central office of the owner from whom he used to receive orders to operate the gates and was afraid to produce a downstream flood affecting some riverside population and, when he decided to open the gates, the gate command was inoperative because of power supply failure and he could not reach the spillway because of the flooded access. Overtopping destroyed the Euclides da Cunha Dam and the next downstream Armando de Salles Oliveira Dam, in the Pardo River. After these events the owner conducted an overall revision of all its practices for dam operation (Siqueira, 1978).

Besides the electro-mechanical issues, civil structures must be operated and maintained as well to remain in good conditions. The questions that generally deserve special attention are cavitation action in chutes and stilling basins, hydraulic problems associated with asymmetrical operation of gates, the transport and deposition of sediment in stilling basins and the expansion of downstream erosion into the foundation of the structures. These questions are extensively treated in the reference papers mentioned in this chapter.

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3. HIGH HEAD SPILLWAYS – THE CHALLENGE OF ENERGY DISSIPATION AND SCOUR CONTROL DOWNSTREAM(*)

3.1. INTRODUCTION

The safety of dams during flood events has to be ensured by an appropriate capacity of the releasing structures. In the case of high head spillways, besides of well-known hydraulic structures problems due to high velocity flows and cavitation risk, one main issue is the challenge of energy dissipation and scour control downstream (Schleiss 2002). High velocity jets can occur which are guided by the releasing structures into the tail-water at a certain distance from the dam. At the zone of impact of these high-energy jets, the riverbed will be scoured. Since scour due to plunging jets can reach considerable depth even in rocky river beds, instability of the valley slopes has to be feared, which can endanger in some cases the foundation and the abutment of the dam itself. Such scour problems occur especially at dams were the spillways are combined with the dam structure itself and, consequently, the impact zone of the high-velocity falling jets is relatively close to the dam. This is typically the case with concrete dams, where high velocity falling jets can be created by gated or ungated crest spillways (arch dams only), chute spillways followed by a ski jump and orifice spillways as well as high capacity bottom outlets. Severe scour conditions occur especially in the case of high concrete arch dams in narrow valleys with high flood discharges.

Such a typical spillway arrangement is shown in Figure 3.1 and Figure 3.2 at the example of Khersan III Dam Project in Iran. Flood handling at the 175 m high double arch dam will be provided by three separate spillway facilities:

- a two-bay chute flip bucket spillway on the right abutment with an ogee crest at El. 1404.5 m controlled by 11.5 m wide by 13.5 m high radial gates (capacity of 4240 m^3/s at PMF flood level El. 1426.3 m);
- an uncontrolled crest spillway with ogee crest divided in 6 bays; two 13.5 m and two 19.5 m wide bays at El. 1418 m and two 12.5 m wide bays at El. 1421 m (total capacity of 3360 m^3/s at PMF flood level El. 1426.3 m);
- two bottom outlets with centerline at El. 1330 m and 1345 m with service gate openings, 3 m wide by 4 m high (total capacity of 395 m^3/s at PMF flood level El. 1426.3 m).

(*)This Chapter corresponds to an updated and enhanced version of Schleiss (2002)

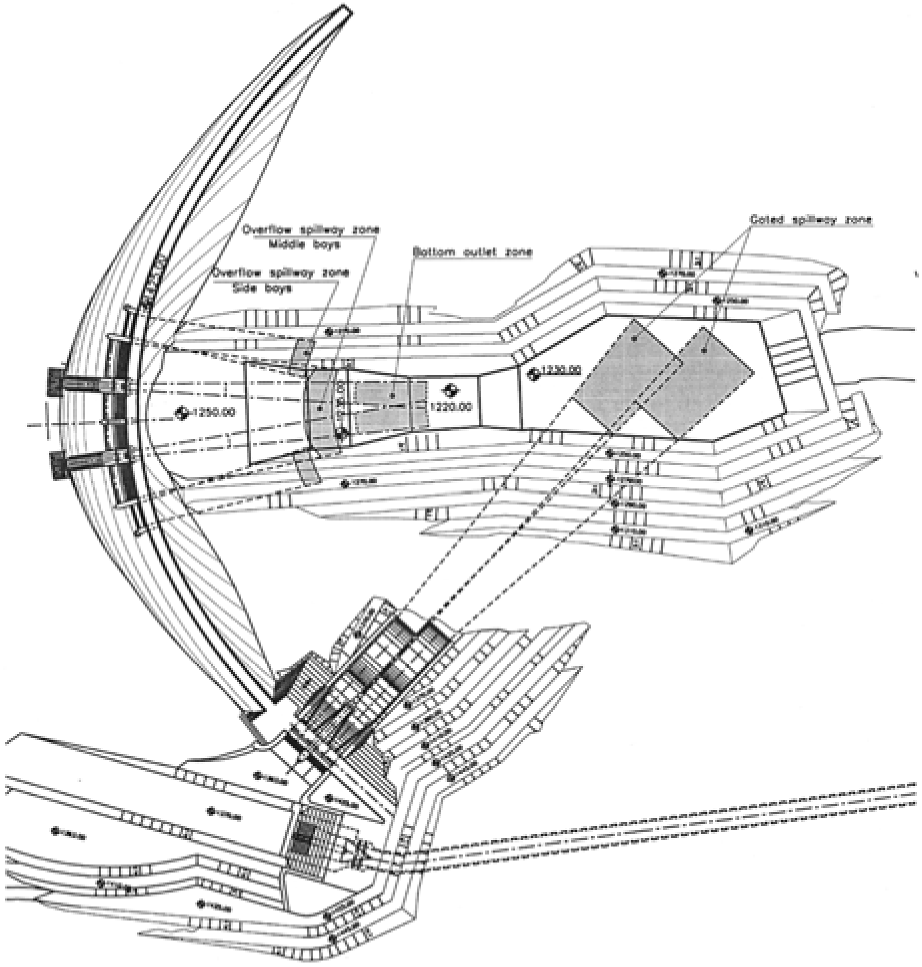


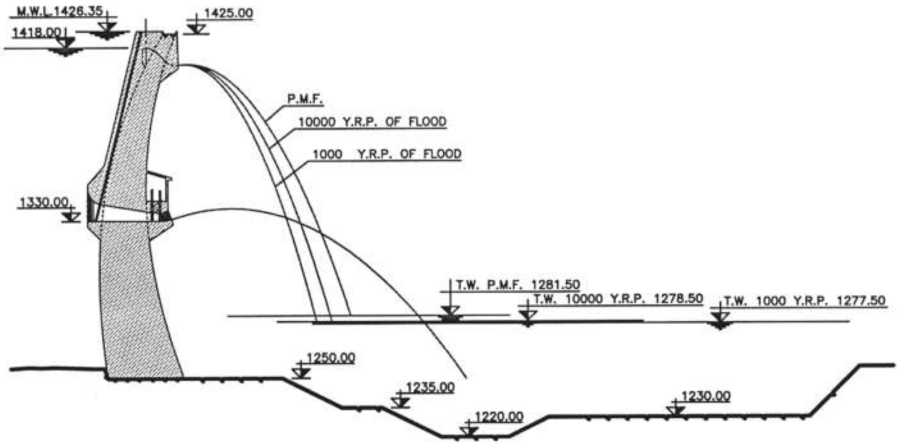
Figure 3.1 - Khersan III Dam Project in Iran. Layout of arch dam with spillway structures and its jet impact zones.

In today's spillway design of dams there is a tendency of increasing the unit discharge of the high velocity jet leaving the appurtenant structures. In gated chute flip bucket spillway, specific discharges in the range of 200 to 300 $m^3/s/m$ are not rare anymore, since cavitation risk in chutes can be mitigated by the help of bottom aerators. Uncontrolled crest spillways for arch dams are designed nowadays for specific discharges up to 70 $m^3/s/m$ and up to 120 $m^3/s/m$ by installing gates on the crest. With the latest high pressure gate technology, low level orifice spillways can evacuate specific discharges in the range of 300 to 400 $m^3/s/m$.

This trend is also confirmed by many high dam projects in China with large discharge flows and built in narrow valleys. Special experiences on high gravity dams, high arch dams and high rockfill dams have been reported by Gao *et al*. (2011).

Besides the hydraulic design questions of the water release structures itself, the challenge regarding energy dissipation and scour control, must answer the following questions:

- What will be the evolution and extent of scour downstream of the dam at the jet impact zone?
- Are the stability of the valley slopes and the foundation of the dam itself endangered?
- Is a tail-pond dam required to create a water cushion and how does it affect the scour depth?
- Is a pre-excavation of the rocky river bed required and what should be its shape
- Has the plunge pool to be lined?
- Is the tail-water level and powerhouse operation influenced by scour formation?



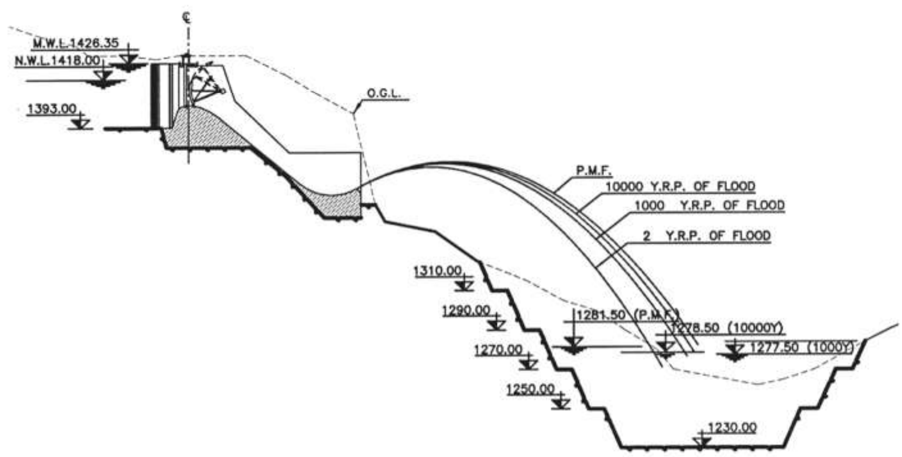


Figure 3.2 - Khersan III Dam Project in Iran. Longitudinal profiles through crest spillway and gated side spillway and plunge pools.

3.2. THE SCOUR PROCESS

3.2.1. The physical processes

Scouring is a complex problem and has been studied since longtime. As illustrated in Figure 3.3, scour can be described by a series of physical processes as (Bollaert 2002, Mason 2011):

a) free falling jet behavior in the air and aerated jet impingement;

b) plunging jet behavior and turbulent flow in the plunge pool;

c) pressure fluctuation at the water-rock interface;

d) propagation of dynamic water pressures into rock joints;

e) hydrodynamic fracturing of closed end rock joints and splitting of rock in rock blocks;

f) ejection of the so formed rock blocks by dynamic uplift into the plunge pool and entrain and circulate the excavated rock around the pool and vertically over the height of the pool;

g) break-up and degrading of the rock blocks by the ball milling effect of the turbulent flow in the plunge pool;

h) ejecting the rocks downstream to a point where they cannot roll back into the pool and thus formation of a downstream mound;

i) displacement downstream of the scoured materials by the flow transport in the river.

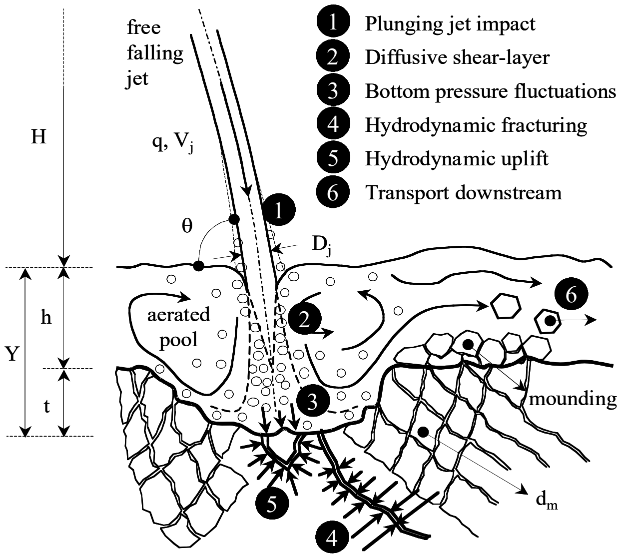


Figure 3.3 - Main parameters and physical-mechanical processes involved in the scouring phenomenon (Bollaert 2002)

3.2.2 Jet behavior in the air

In evaluating the scour caused by free jets, it is at first necessary to predict the jet trajectory so that the location of the jet impingement in the plunge pool and the zone of the scour hole are known (Whittaker & Schleiss 1984). The behavior of an ideal jet can be easily assessed by using ballistic equations. Nevertheless, at prototype jets effects as air drag, disintegration of the jet in the air and initial flow aeration in long chutes have to be considered. A number of researchers have developed equations to predict the jet trajectory, i.e. the jet travel length accounting for these effects (Gun'ko *et al*. 1965, Kamenev 1966, Kawakami 1973, Zvorykin *et al*. 1975, Taraimovich 1980, Martins 1977).

The spread of the jet during fall is also an issue and has been addressed on an empirical basis by Taraimovich, 1980 and U.S.B.R. 1978. More recently, the lateral spread has been related to its initial turbulence intensity (Ervine & Falvey, 1987; Ervine *et al*. 1997). Typical angles of jet spread are 3 to 4 % for roughly turbulent jets. Turbulence intensity for free over-fall jets is less than 3 %, for flip bucket jets between 3 and 5 % and for orifice jets between 3 and 8 % (Bollaert 2002). Based on experimental results and an extensive literature survey, jet issuance parameters relevant for engineering practice can be found in Manso *et al*. (2008).

3.2.3 Jet behavior in the plunge pool and pressure fluctuations

As the jet plunges into the pool, a considerable amount of air is entrained, corresponding to an air concentration of 40 to 60% at typical jet velocities of 30 m/s at impingement (Bollaert 2002). Several expressions defining the air content at the point of impact of the jet in the plunge pool have been developed by scale model tests. Some of them can be reasonably extended towards prototype velocities (Van de Sande 1973, Bin 1984, Ervine *et al*. 1980, Ervine 1998).

The flow conditions in the plunge pool can be characterized by a high-velocity, two-phase turbulent shear layer flow and a macro-turbulent flow outside of this zone (Bollaert 2002, Manso 2066). The shear layer produces severe pressure fluctuations at the water-rock interface and is highly influenced by aeration. Existing theories on two-dimensional vertical jet diffusion in a semi-infinite or bounded medium define the outer limits of the water-rock interface that is directly subjected to these pressures. The diffusion of 2D-jet has been investigated by many researchers (Bollaert & Schleiss 2003). The concept of a jet of uniform velocity field penetrating into a stagnant fluid is based on the progressive growing of the thickness of the related boundary layer by exchange of momentum. In this shear layer the total cross section of the jet increases whereas the non-viscous core of the jet decreases. The inner angle of diffusion is about 8° for highly turbulent impinging jets. When the plunge pool is deep enough, the core jet disappears and a fully developed jet occurs. The angle of diffusion of the jet through the pool depends on the degree of turbulence and aeration of the jet at impact and can be estimated at around 15° (Ervine & Falvey 1987). The most severe hydrodynamic action on the plunge pool bottom occurs in the impingement region, where the hydrostatic pressure of the free jet region is progressively transformed into fluctuating high stagnation pressures and into an important bottom shear stress.

Knowledge on the statistical characteristics and of the spatial distribution of the pressure fluctuations has been acquired by physical modelling of hydraulic jumps and plunging jets (Tos &

Bowers 1988, Ballio *et al*. 1992, Bollaert & Schleiss 2003, 2005, Manso 2006). The pressure patterns generated by core jet impact are generally quite constant with high values in the core and much lower values directly outside the core. The pressures for developed jet impact are completely different from pressures generated by core jet impact. Due to the high turbulence level and the two phase character of the shear layer, developed jet impact conditions can generate much more severe dynamic pressures at the pool bottom. Therefore, not every water cushion has a retarding effect on the scour formation. The spectral energy content of a high velocity plunging jet extends over a much wider frequency range (> 100Hz) than what is generally assumed for macro-turbulent flows in plunge pools (up to 25 Hz).

Systematic experimental investigations in plunge pools with different lateral confinements showed that the pool geometry influences not only plunging jet diffusion and air entrainment in the pool, but also impact pressures at the water-rock interface and inside the fissured rock mass (Manso et al. 2009). Favourable plunge pool geometries may limit the pressure fluctuations at water rock interface and therefore the scour potential is reduced (Bollaert *et al*. 2012).

3.2.4 Propagation of dynamic water pressures into rock joints, hydrodynamic fracturing and dynamic uplift

The transfer of pool bottom pressures into rock joints results in transient flow that is governed by the propagation of pressure waves. For closed-end rock joints, as encountered in a partially jointed rock mass, the reflection and superposition of pressure waves generate a hydrodynamic loading at the tip of the joint. If the corresponding stresses at the tip of the joint exceed the fracture toughness and the initial compressive stresses in the rock, the rock will crack and the joint can further grow. In the case of open-end rock joints in a fully jointed rock mass, the pressure waves inside the joints create a significant dynamic uplift force on the rock blocks. This dynamic uplift force will break up the remaining rock bridges in the joints by fatigue and, if high enough, eject the so formed rock blocks from the rock mass into the macro-turbulent plunge pool flow.

The presence of such pressure waves in rock joints and significant amplifications due to resonance phenomena could be observed and measured with an experimental study at near prototype scale (Bollaert 2002, Bollaert & Schleiss 2003, 2005, Manso 2006, Federspiel 2011, Duarte *et al*. 2012).

3.2.5 Ball milling effect of the turbulent flow in the plunge pool and formation of a downstream mound

Once the rock blocks are formed and ejected by the dynamic uplift from the surrounding rock mass into the pool, they can be taken up by the macro-turbulent eddies. If the block is too big to be transported by the flow, it will be broken-up after some time by the ball milling effect of the eddies in the plunge pool. Having attained the required minimum size, the rock blocks will be displaced downstream by the flow and deposited on the mound or carried away by sediment transport in the river. The mound may limit the depth of scour but also raise the tail-water level. Exceeding a critical level, the increased tail-water may interfere with the operation of bottom outlets or reduce the net head of the powerhouse, if it is located upstream of the mound. If the mound does limit the depth of the scour, the scour is considered to have attained a so-called dynamic limit. However if the mound is removed and the scour proceeds to a maximum possible extent, it is considered to have attained the ultimate static limit (Eggenberger & Müller 1944).

3.3. SCOUR EVALUATION METHODS

3.3.1. General overview

The existing scour evaluation methods can be grouped as follows (Bollaert & Schleiss 2003):

- empirical approaches based on laboratory and field observations;
- analytical-empirical methods combining laboratory and field observations with some physics;
- approaches based on extreme values of fluctuating pressures at the plunge pool bottom;
- techniques based on time-mean and instantaneous pressure differences and accounting for rock characteristics;
- scour model based on fully transient water pressures in rock joints.

The most common methods used for scour evaluation due to falling, high velocity jets are illustrated in Figure 3.4 for the considered physical parameters involved in the scour process which can be related to the three phases water, rock and air. Time evolution is a further parameter.

3.3.2 Empirical formulae

A large number of empirical equations have been developed for predicting the scour from plunging jets. These empirical formulae were mostly derived from hydraulic model tests in the laboratory but also from prototype observation and are widely used in practice for design purposes. Some expressions are of general applicability, others are specific to free over-fall jets, ski-jumps or orifice spillways. A comprehensive overview and comparison of most of the known formula have been given by Whittaker & Schleiss (1984), Mason & Arumugam (1985) and Bollaert (2002).

The complex scouring process is reduced by the empirical formulae to a few parameters. The ultimate total scour depth Y measured from tail-water level is thought to be a function of:

- the specific discharge q (discharge per unit width of jet);
- the fall height H;
- the tailwater depth h (measured from initial river bed level);
- the characteristic sediment size or rock block diameter d.

Most of the formulas are written in the form

$$Y = t + h = K \cdot \frac{H^y \cdot q^x \cdot h^w}{g^v \cdot d_m^z} \qquad (1)$$

where t is the scour depth below the initial bed level and K a constant.

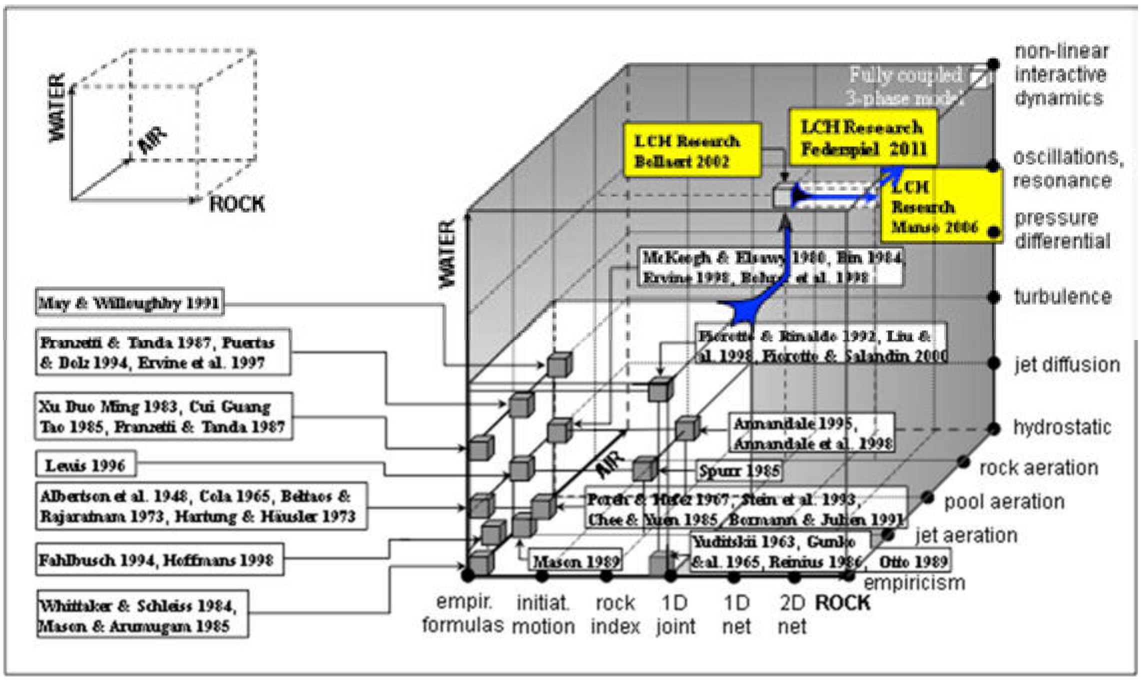


Figure - 3.4. Summary of existing scour evaluation methods considering physical parameters involved in the scour process, which can be related to the three phases: water, rock and air (completed according Bollaert 2002)

Mason & Arumugam (1985) applied this form of formula to a large number of scour data, 26 sets from prototypes and 47 from model tests. Following further work, Mason (1989) suggested the following best fit exponents and constants for both model and prototype conditions:

$$K = (6.42 - 3.10H^{0.10})$$
$$v = 0.30$$
$$w = 0.15$$
$$x = 0.60$$
$$y = 0.05$$
$$z = 0.10$$

This was shown to give the most consistent and accurate prediction for the 46 model test data set and a general upper bound for the prototype data set. It was also shown to dimensionally balance which was a pleasing indicator of probable validity (Mason 1989).

According to the data sets, the formula is applicable for hydraulic model fall heights H between 0.325 m and 2.15 m and between 15.82 m and 109 m for prototypes in the case of free over-fall jet, ski-jump or orifice spillways. The use of the mean particle size d_m gave better results than the use of the d_{90} particle size. For prototype rock, an equivalent particle size d_m = 0.25 m is recommended in the above formula.

3.3.3 Semi-empirical equations

As already mentioned, semi-empirical methods are combining laboratory and field observations with some physics as:

- initiation of motion of the bed material by shear stress;
- energy conservation equations;
- geomechanical characteristics;
- angle of impingement of the jet;
- steady-state two-dimensional jet diffusion theory;
- aeration effects.

A detailed overview of these semi-empirical equations and methods can be found in Whittaker & Schleiss (1984) and Bollaert (2002). The hydrodynamic process of scour is often derived from the two-dimensional jet diffusion theory. The geomechanical behavior of the rock mass is considered by the shear-stress based initiation of motion concept for non-cohesive granular materials or by an index that defines the resistance of the rock mass against erosion. Both hydrodynamic and geomechanical characteristics are for example combined in Spurr's (1985) and Annandale's (1995, 2006) erodibility index methods for rocks and in the momentum conservation equations established by Fahlbush (1994) and Hoffmans (1998) for non-cohesive material.

3.3.4 Approaches based on extreme values of fluctuating pressures at the plunge pool bottom

At the plunge pool bottom, fluctuating, dynamic pressures occur due to the direct jet core impact in the case of relatively small water cushion. For high plunge pool depth (higher than 4 to 6 times the thickness of the incoming jet) a turbulent shear flow or developed jet impact according to the two-dimensional jet diffusion theory is created. These two types of jet impact generate completely different pressure patterns as already mentioned.

The dynamic pressures at the plunge pool bottom can penetrate into fissures of the underlying rock mass. The approaches based on the extreme pool bottom pressures assume that the maximum pressures occurring at the water-rock interface are transferred through the joints underneath the rock blocks. These maximum pressures underneath the rock blocks combined with the minimum pressures at the plunge pool bottom create a net uplift pressure Δp on the rock blocks (Figure 3.5). The ultimate scour is reached when this net pressure difference Δp on the rock block is not able anymore to eject it. Since the maximum and minimum pressures are not occurring at the same moment, the so defined net uplift pressure represents a physical upper limit of dynamic loading conditions and is therefore rather conservative.

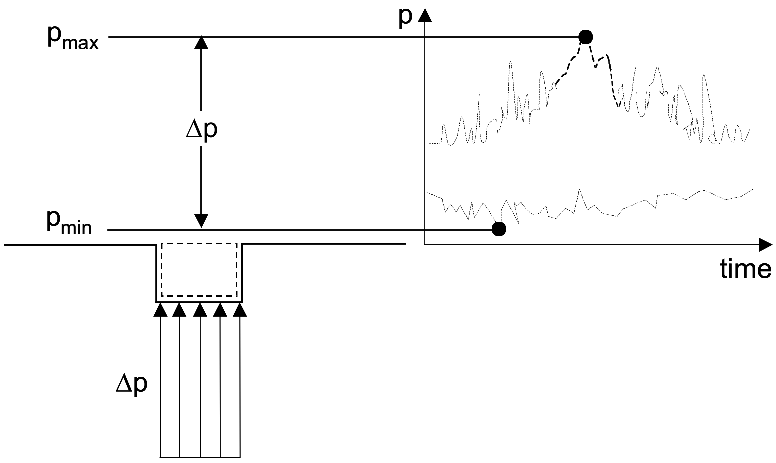


Figure 3.5 - Definition sketch of extreme dynamic pressures at the plunge pool bottom. The maximum and minimum pressures are defined at the center of the block for a long enough time interval (according Bollaert 2002).

Studies on pressure fluctuations in plunge pools have mainly been carried out by Ervine *et al*. (1997), Xu-Duo-Ming (1983) and Franzetti & Tanda (1984, 1987) for circular jet impingement and by Tao *et al*. (1985), Lopardo (1988), Armengou (1991), May & Willoughby (1991) and Puertas & Dolz (1994) on rectangular jets. These studies give useful information on bottom pressure fluctuations but do not describe their propagation inside the joints of the underlying rock mass. The simultaneous application of extreme minimum and maximum bottom pressures above and underneath rock blocks can result in net pressure differences of up to 7 times the root-mean-square value or up to 1.5 - 1.75 times the incoming kinetic energy of the jet. Even if this seems to be a conservative design criterion it has to be noted, that violent transient pressure phenomena, which could occur inside the rock joints, are not considered (Bollaert 2002).

3.3.5 Techniques based on time-mean and instantaneous pressure differences and accounting for rock characteristics

Contrary to the approach presented in the previous section, time-averaged or instantaneous pressure differences occurring at or during a certain time at the surface and underneath the rock blocks, are considered. This means that the fluctuating pressures have not only to be measured at the plunge pool bottom but also inside the rock joints.

Yuditskii (1963) (reported by Gunko et al., 1965) was probably the first who stated that time averaged and pulsating pressures are responsible for rock block uplift in the scour process. He measured the forces on a single rock block on flat plunge pool bottoms due to the impact of a jet produced by a ski-jump spillway in a scale model. Measurements techniques at that time allowed only to obtain time averaged forces. More recently Otto (1989) quantified time-averaged uplift pressures acting on a rock block for plane jets impinging obliquely. Depending on the relative protrusion of the block and the point of jet impact, important surface suction effects occurred, leading to mean uplift pressures of almost the total incoming kinetic energy. Without considering this suction effect, the maximum uplift pressures were still half of the incoming kinetic energy.

The destructive effects of instantaneous pressure differences entering in tiny rock joints was highlighted the first time by Hartung & Häusler (1973). Kirschke (1974) at first performed an analytical and numerical analysis of water hammer propagation in rock joints. Instantaneous pressure differences based on transient flow assumptions were measured and analysed for the first time for the case of concrete slab linings of stilling basins (Fiorotto & Rinaldo 1992, Bellin & Fiorotto (1995), Fiorotto & Salandin 2000, Fiorotto & Caroni 2007). However, the scale of the model tests and the data acquisition rate did not allow measuring any oscillatory or resonance effects in the joint under the slabs.

Annandale *et al*. (1998) simulated the erosion of fractured rock by the use of lightweight concrete blocks, placed in a series of two layers on a 45° dip angle. Jet impingement confirmed their theory that the erosion threshold criterion for rock and soil material can be defined by means of a geomechanical index, the so-called Erodibility Index. The erosive power of the water can be related to the erosion resistance of the material.

Experimental and numerical studies focusing on fluctuating net uplift forces on simulated rock blocks were also performed by Liu *et al*. (1998). A design criterion for rock block uplift was given based on a transient flow model but without considering resonance effects (Liu 1999).

All the mentioned studies on concrete slabs and rock blocks consider the surface pressure field as a function of space and time. But the pressure field underneath is assumed constant over the surface of the element and equal to the pressure at the entrance of the joints i.e. at the surface. Therefore fully transient flow conditions in the joints such as pressure wave reflections and amplifications are neglected by these extreme pressure techniques.

3.3.6 Scour model based on fully transient water pressures in rock joints

Bollaert (2002) measured for the first time the transient pressures in rock joints due to high-velocity jet impact with systematic laboratory tests with and experimental set-up reproducing near prototype conditions and reproduced them in a numerical model. New phenomena could be observed and explained as the reflection and superposition of pressure waves, resonance pressures and quasi-instantaneous air release and resolution due to pressure drops in the joint.

The analysis revealed that the pressure wave velocity is highly influenced by the presence of free air bubbles in the joints. These bubbles can be transported by flow from the plunge pool into the joint but also be released from the water during sudden pressure drop below atmospheric pressure.

In open-end joints instantaneous net uplift pressures of 0.8 to 1.6 times the incoming kinetic energy of the impacting jet has been measured. This is significantly higher than any previous assumptions in literature (see also section 3.3.5) and underlines that transient pressure effects in rock joints are a key physical process for scour formation.

Based on the experimental results and the numerical simulation, a new model for the evaluation of the ultimate scour depth has been developed, the so-called Comprehensive Scour Method (CSM), which represents a comprehensive assessment of the two physical processes: hydrodynamic fracturing of closed-end rock joints and dynamic uplift of rock blocks (Bollaert 2002, 2004, Bollaert & Schleiss 2005). All relevant processes as the characteristics of the free falling jet (velocity and diameter at issuance, initial jet turbulence intensity), the pressure fluctuations at the plunge pool bottom and the hydrodynamic loading inside rock joints are dealt with and compared with the resistance of the rock against crack propagation.

The CSM model was further enhanced by considering geometry of plunge pool and lateral confinement of plunging jet (Manso 2006) as well as the dynamic response of a rock block due to fluid-structure interaction (Federspiel 2011, Asadollahi *et al*. 2011). In a recent research the influence of jet aeration was investigated and implemented in the CSM model (Duarte *et al*. 2012, 2013).

The application of the CSM model needs knowledge on rock parameters as number, spacing, direction and persistency of fracture sets, in-situ stresses in the rock mass, fracture toughness and unconfined compressive strength. These parameters are normally assessed during the geological and geotechnical reconnaissance campaign for any dam foundation project and are therefore available. Nevertheless uncertainties have to be addressed by sensibility analysis and engineering judgment.

3.4. DIFFICULTIES ENCOUNTERED WHEN ESTIMATING SCOUR DEPTH

3.4.1 Which is the appropriate formula or theory?

In the feasibility design stage of a dam project normally the easily applicable empirical and semi-empirical formulae are used to get a first idea of the expected ultimate scour depth occurring downstream of the spillway structures during the project lifetime. Most of the formulae have been developed for a specific case such as a ski-jump, a free crest over-fall or an orifice spillway, while some others are of general applicability. Therefore a careful selection of the appropriate equations should be made for each project. Nevertheless even after this selection, the results of the remaining formulae show very often a wide scatter. In such a case, the engineering decisions can be based on a statistical analysis of the results by using, for example, the average of all formulae or the positive standard deviation in a more conservative way. This analysis is carried out for a certain spillway discharge with the corresponding tail-water level by varying the characteristic rock block size (if considered) according to expected geological conditions.

Sometimes prototype scour measures of an existing dam, situated in similar geological conditions than the foreseen new project, are available. Then the formula predicting best the scour depth can be identified and be applied to the new project.

3.4.2 How model tests should be performed and interpreted?

If free jets impinging on rock underlying a plunge pool have to be modeled in a laboratory, three difficulties may arise (Whittaker & Schleiss 1984):

- the appropriate choice of a material that will behave dynamically in the model as fissured rock does in the prototype;
- the grain size effects;
- the aeration effects.

In most models the disintegration process, that means the hydrodynamic fracturing of closed end rock joints and splitting of rock in rock blocks, is assumed to have taken place. Thus only the ejection of the rock blocks into the macro-turbulent plunge pool flow and the transport of the material out from the scour hole are modelled. Reasonable results may be obtained if fissured rock is modelled by appropriately shaped concrete elements (Martins 1973), but their regular pattern and size does not fully represent a rock mass with several intersecting fracture sets. Nevertheless, crushed rock having at least a grain size distribution similar to the expected rock blocks, should be used instead of round river gravel. In any case, model tests cannot simulate the break-up of the rock blocks by the ball milling effect of the turbulent flow in the plunge pool. Therefore in the model a mound is formed which is higher and more stable than in the prototype. As a result the prototype scour depth is underestimated. This can be compensated to some extent by choosing the material carefully including downscaling. Normally good predictive results for scour depth can be obtained by using non-cohesive material, but the extent of the scour may be not correct because steep and near vertical slopes cannot be reproduced. Therefore the use of slightly cohesive material by adding cement, clay caulk, chalk powder, paraffin wax etc. to the crushed gravel is proposed (Johnson 1977, Gerodetti 1982, Quintela & Da Cruz 1982, Mason 1984).

It is known that for grain sizes smaller than 2 to 5 mm, the ultimate scour depth becomes constant (Veronese 1937, Mirtskhulava *et al*. 1967, Machado 1982). For an acceptable scale of a comprehensive dam model of 1: 50 to 1:70, the smallest prototype rock blocks, which can be reproduced, are in the range of 0.1 to 0.35 m.

Finally air entrainment cannot be scaled appropriately in comprehensive models unless using an unpractical large scale (in the order of 1:10). Air entrainment has a highly random character, which influences the scour process considerably (see section 3.2). Mason (1989) studied systematically the effect of jet aeration on scour in a mobile gravel bed with a specially designed apparatus in the laboratory for head drops of maximum 2 m. Duarte *et al*. (2012, 2013) studied systematically the effect of jet aeration on the dynamic response of a rock block impacted by high velocity jet in an experimental set-up reproducing near prototype conditions.

Through the use of hybrid modelling the behaviour of rock scour can be correctly represented. This was successfully done for defining mitigation measures for the scour of Kariba dam (Noret *et al*. 2013, Duarte *et al*. 2011). Several historical scour geometries were reproduced in a physical model with a fixed mortar surface equipped with pressure sensors. Four different spillway operation dynamic pressures were measured at the water rock interface. These recorded dynamic pressures were then introduced in the comprehensive scour method (CSM) of Bollaert & Schleiss (2005), which can predict how deep rock blocks can be formed by fatigue and ejected into the plunge pool by the dynamic pressures.

3.4.3 How to analyse prototype observations properly?

When analysing prototype observations on scour depth in order to derive an equation for similar conditions, the following questions have to be answered:

- What was the duration of the operation of the spillway for different specific discharges (discharge-duration curve)? (An example of discharge-duration curve of a spillway is given in Figure 3.6).

- Which was the prevailing, specific discharge, which has formed the scour depth?
- Was the duration of this specific discharge long enough to create ultimate scour depth?

Since in practice it is very often difficult to answer precisely to these questions, probably significant uncertainties have been introduced into the existing formulas derived from prototype observations. This may also explain the large scatter when predicting scour for other prototypes.

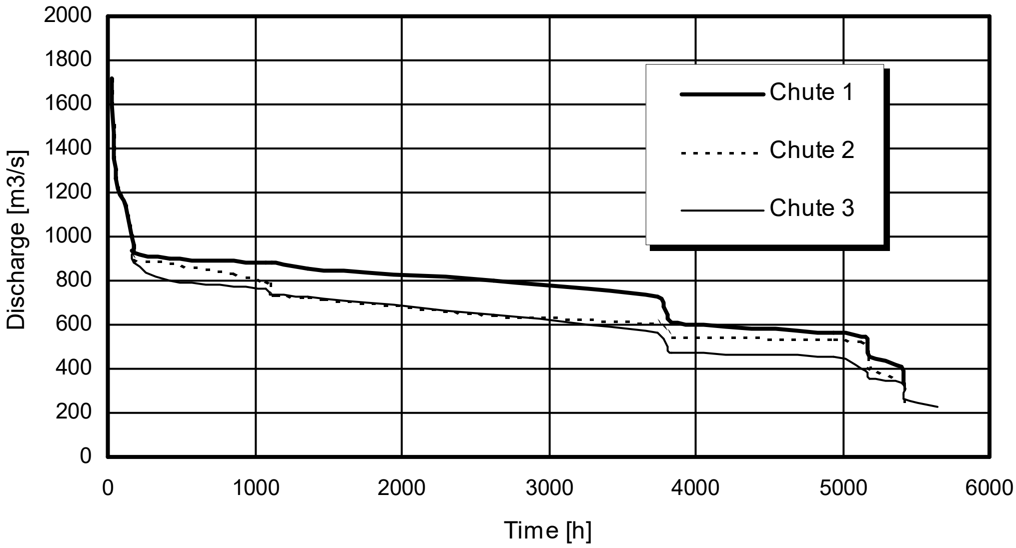


Figure 3.6 - Discharge-duration curve of the chute spillway of Karun I Dam in Iran for the period between March 1980 and July 1988 (width per chute 15 m) (Schleiss 2002).

3.4.4 Can ultimate scour depth be achieved during spillway operation period and what is the scour rate?

In principle, the scour depth estimated by use of empirical and semi-empirical formula will occur only for a long duration of spillway operation, after steady conditions in the scour hole are achieved. This will happen only after a minimum duration of spillway operation, mainly depending on the quality and jointing of the rock mass. Therefore the specific discharge which will have a sufficiently long duration to form the ultimate scour during the technical life of the dam has to be known. Higher and therefore rare discharges are not able to create ultimate scour.

Since plunge pool scour t + h is known to develop at an exponential rate with time T, the scour rate can estimated with the following relationship (Spurr 1985):

$$(t + h)(T) = (t + h)_{end}\ (1 - e^{-aT/T_e}) \qquad (2)$$

where **a** is a site-specific constant. The evaluation of T_e (instant at which equilibrium is attained) depends on how rapidly hydro-fracturing and washing out of the material from the scour hole will occur, taking into account the primary and secondary rock characteristics. Primary rock characteristics comprise RQD, joint spacing, uniaxial compressive strength, and angle of jet impact compared to main faults or bedding planes; the secondary characteristics are the hardness and degree of weathering (Spurr 1985). Knowing the depth of a scour, which occurred during a certain period of operation and estimating the maximum scour depth by one of the formulae, the site specific constant a/T_e can be determined.

As a rough estimation based on some prototype data, ultimate scour is normally attained only after T_e = 100 to 300 hours of spillway operation for a certain discharge considered.

It may be concluded, that the ultimate scour depth for a certain specific discharge occurs only if the duration of this discharge is long enough. Scour for a smaller duration can be estimated with an exponential rate relationship.

3.4.5 Which will be the prevailing discharge for scour formation during a flood event?

A flood event and the corresponding discharge curve of the spillway can be characterized by a hydrograph (Figure 3.7). For all discharges of the hydrograph with duration shorter than the instant T_e at which equilibrium is attained, the ultimate scour will not be reached. Knowing the scour rate relationship [Equation (2)], the prevailing discharge which will produce maximum scour depth during the flood event can be determined. The scour is estimated successively for discharges

q_u ($T = T_e = T_u$), q_1 ($T_1 < T_u$), q_2 ($T_2 < T_1 < T_u$), ..., q_{peak} ($T_{peak} < T_i < T_u$).

The discharge, which gives the deepest scour, is the prevailing discharge.

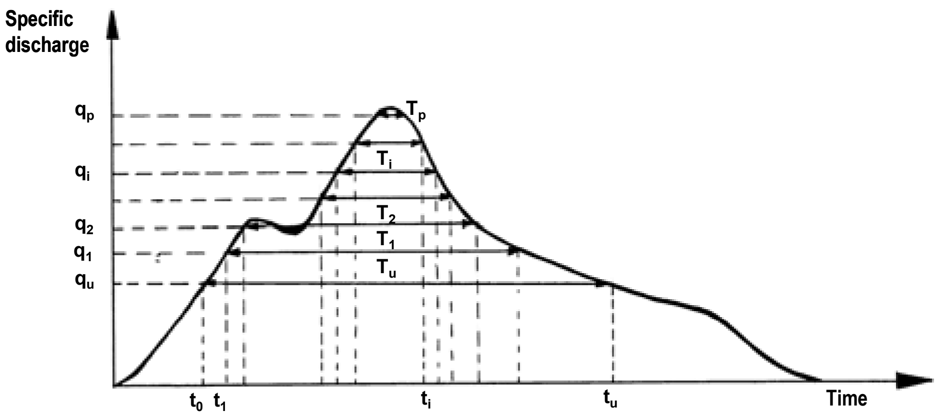


Figure 3.7 - Flood event and the corresponding discharge curve of the spillway showing discharges with a duration shorter than the instant $T_e = T_u$ at which equilibrium of scour and its ultimate depth is attained with the purpose to determine the prevailing discharge (Schleiss 2002).

It has to be noted that these considerations are valid only for ungated free surface spillways. For gated free surface spillways the discharge may not be directly related to the reservoir inflow but be prescribed by operation rules. When lowering the reservoir level during floods, outflow discharges are higher than reservoir inflow. This can also be the case for pressurized orifice spillways

3.5. SPILLWAY DESIGN DISCHARGE AND SCOUR EVALUATION

Spillways are designed for the so-called design or project flood, typically a 1000-year flood, and checked for the so-called safety check flood, typically to a flood between a 10 000-year flood and PMF. The question arises for what flood the scour depth has to be evaluated and the constructive scour control measures have to be based on.

As discussed in section 3.4.4, the ultimate scour depth will occur only after steady conditions in the scour hole are achieved, which will happen only after certain duration of spillway operation. Therefore it is very conservative to base the estimation of the scour depth or the design of mitigation measures on low frequency floods (PMF or 10 000-year flood). It will be very unlikely that during the technical life of the dam, these rare floods can produce ultimate scour depth.

Therefore for each flood event with a certain return period, the prevailing discharge and the maximum scour depth has first to be determined according section 3.4.5. Furthermore it has to be decided, for which flood return period the maximum scour depth during the technical life of the dam, will be estimated. The probability of the occurrence of a flood with a given return period during the useful life of a dam is as follows:

$$r = 1 - (1 - 1/n)^m \qquad (3)$$

where r is the risk or the probability of occurrence, n the return period of the flood (years) and m the useful life of the dam.

In Figure 3.8 the probability of occurrence of floods for different useful lifetime of the dam is illustrated. It can be seen, that the probability of the occurrence of a 200-year flood during 200 years of operation is 63 %, whereas for a 1000-years flood is only 20 %.

It seems reasonable to choose a design discharge with a probability of occurrence of about 50 % during the useful lifetime of a dam for the scour evaluation and the protection measures. Higher design discharges with lower probability of occurrence are too conservative.

It has to be noted, that in the case of gated, surface and orifice spillways, high discharges can be released at any time by opening the gates. Furthermore for low-level outlet spillways the core impact velocity of the jet is nearly independent from discharge.

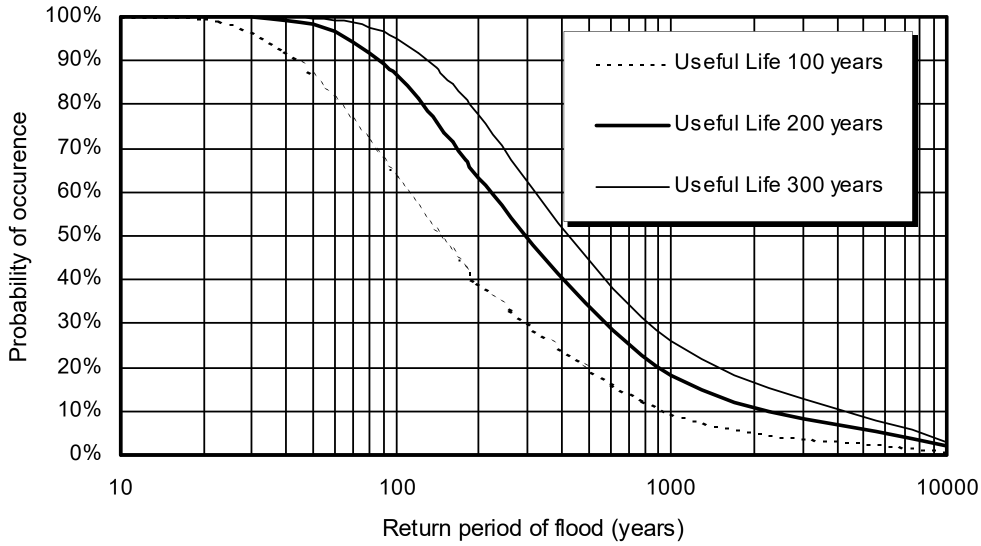


Figure 3.8 - Probability of occurrence of a flood with a given return period for different useful lifetimes of the dam (Schleiss 2002).

3.6. MEASURES FOR SCOUR CONTROL

3.6.1 Overview

To avoid scour damage, three active options may be considered:

- avoid scour formation completely;
- design water release structures such that the scour occurs far from dam foundation and abutments;
- limit the scour extent.

Since structures for scour control are rather expensive, only the two latter are normally economically feasible (Ramos 1982). Besides elongating as much as possible the impact zone of the jets by an appropriate design of the water release structures, the extent of the scour can be influenced by the following measures:

- limitation of the specific spillway discharge;
- forced aeration and splitting of jets leaving spillway structures;
- increasing tailwater depth by a tailpond dam;
- pre-excavation of the plunge pool.

The location of the scour depends on the selected type of spillway and its design.

To avoid scour completely, structural measures as lined plunge pools are required. Besides the active options, scour damage can be prevented also by passive measures, for example by protecting dam abutments with anchors against instability due to scour formation.

3.6.2 Limitation of the specific spillway discharge

This measure is mainly important in the case of arch dams and free ogee crest spillways with jet impacts rather close to the dam. The jet can be guided by an appropriate crest lip design for a given specific discharge at a certain distance from the dam toe. If the dam foundation can be endangered by the scour the discharge per unit length of the ogee crest has to be limited. But by reducing the specific discharge, the available velocity at the crest lip and therefore the travel distance of the jet is also reduced.

In the case of gated ski-jump spillways and low-level outlets, the specific discharge depends on the size of the outlet openings. Since the available velocity at the outlet is high enough to divert the jet far away from the dam and its foundation, the limitation of the specific discharge is normally less important than for free crest spillways.

3.6.3 Forced aeration and splitting of jets

In order to split and aerate the jets leaving flip buckets and crest lips, they are often equipped with baffle blocks, splitters and deflectors. Furthermore high velocity flows in spillway chutes are normally aerated by aeration ramps and slots along the chute. All these measures will increase air entrainment, which will reduce the scouring capacity of the plunging jets. Nevertheless the amount of air entrained is difficult to estimate. Because of scale effects the efficiency of these measures can only be checked qualitatively by hydraulic model tests.

Martins (1973) suggested a reduction of 25 % of the calculated scour depth in the case of high air entrainment and 10 % for intermediate air entrainment. Mason (1989) proposed an empirical expression considering the volumetric air-to-water ratio β. The proposed empirical equation based on spillway models does not depend on the fall height H, since he used a direct relationship between β and H as developed by Ervine (1976). The empirical formula is accurate for model data and seems to give a reasonable upper bound of scour depth for prototype conditions.

Nevertheless in the case of prototype scour in fractured rock, air strongly influences water hammer velocity and consequently resonance effects of pressure waves in rock joints. Recent research reveals that forced aeration may reduce pressure fluctuations at the water rock interface but may increase resonance effects due to reduced pressure wave celerity (Duarte et al. 2012, 2013). Thus forced aeration is not always reducing scour potential but may increase under certain conditions the risk of rock block break up and its ejection into the plunge pool.

3.6.4 Increasing tail-water depth by a tail-pond dam

Another way to control scour from jets is to increase tail-water depth by building a tail-pond dam downstream of the jet impact zone. The efficiency of a water cushion is often overestimated (Häusler 1980). For plunge pool depths smaller than 4 to 6 times the jet diameter, core jet impact (see section 3.2.3) is normally observed at the plunge pool bottom (Bollaert 2002). The jet core is characterized by a constant velocity and is not influenced by the outer two-phase shear layer condition of the impinging jet. The pressures are also constant with low fluctuations, which have significant spectral energy at very high frequencies (up to several 100 Hz). For tail-water depths larger than 4 to 6 times the jet diameter (or thickness), developed jet impact occur at the plunge pool bottom with a different pressure pattern produced by a turbulent two-phase shear layer. Very significant fluctuations are produced with high spectral content at frequencies up to 100 Hz. Maximum fluctuations have been observed for tailwater depths between 5 to 8 times the jet diameter (Bollaert & Schleiss 2003). Substantial high values persist up to tailwater depth of 10 to 11 times the jet diameter. From these observations it may be concluded, that water cushions in the range of 5 to 11 times the jet diameter can generate even more severe dynamic pressures at the plunge pool bottom than smaller tail-water depths. Therefore only water cushions deeper than 11 times the diameter of the jet at impact have a retarding effect on the scour formation.

This tendency was also confirmed by the empirical scour equation of Martins (1973), which gives a maximum scour depth for a certain tail-water depth. Johnson (1967) already found that too small water cushions are even worse than no cushion since the material can be transported more easily out of the plunge pool.

It has to be noted, that increased tail-water level by the help of a tail-pond dam may interfere with bottom outlets.

3.6.5 Pre-excavation of the plunge pool

In principle, pre-excavation increases the tail-water depth and in view of scour control, the same remarks are valid than given in section 3.6.4.

In general the pre-excavation of the expected scour may be also appropriate for scour control, when the eroded and river transported material can form dangerous deposits downstream, for example near the outlet of the powerhouse. Such deposits could increase the tail-water level and reduce the power production. Such problems normally should not be expected, when the scour is formed about 200 m upstream of the powerhouse outlet.

Pre-excavation of the scour hole is also often considered when instabilities of the valley slopes may be considered. In such cases the excavation has to be stabilized by anchors and other measures. As an alternative, even under such conditions, pre-excavation could be omitted if the valley slopes are stabilized by appropriate measures, in such a way that the slopes remain stable even after ultimate scour formation.

The selection of the design discharge for an excavated geometry of the plunge pool has to be based on considerations similar to the ones discussed in section 3.3.5. In general it is not economically interesting to excavate deeper than the scour depth, which would be formed by 50- to 100-year floods. If instabilities of the valley slopes are considered possible as a result of deeper scour due to higher spillway discharges, rock anchors and pre-stressed tendons can be used, as already mentioned (Figure 3.9).

Instead of full excavation, pre-splitting of plunge pool rock may by adequate under certain conditions, making the rock more readily erodible in certain places. Pre-splitting can therefore be used to influence the eventual geometry of the plunge pool without the expense of complete excavation. In some cases of course, such as where scour affects power production, complete excavation may be more appropriate.

The pre-excavated plunge pool geometry has to be based on the expected natural scour geometry. Several authors proposed empirical formula for the estimation of the length (Martins 1973, Kotoulas 1967) and the width (Martins 1973) of the scour hole. Amaniam & Urroz (1993) performed a number of tests on a model scale flip bucket spillway with a gravel bed plunge pool in order to develop equations to describe the geometry of the scour hole created by the jet impinging into the plunge pool. They observed that the performance of the pre-excavated scour holes is better when they are close to the self-excavated hole for the same flow parameters.

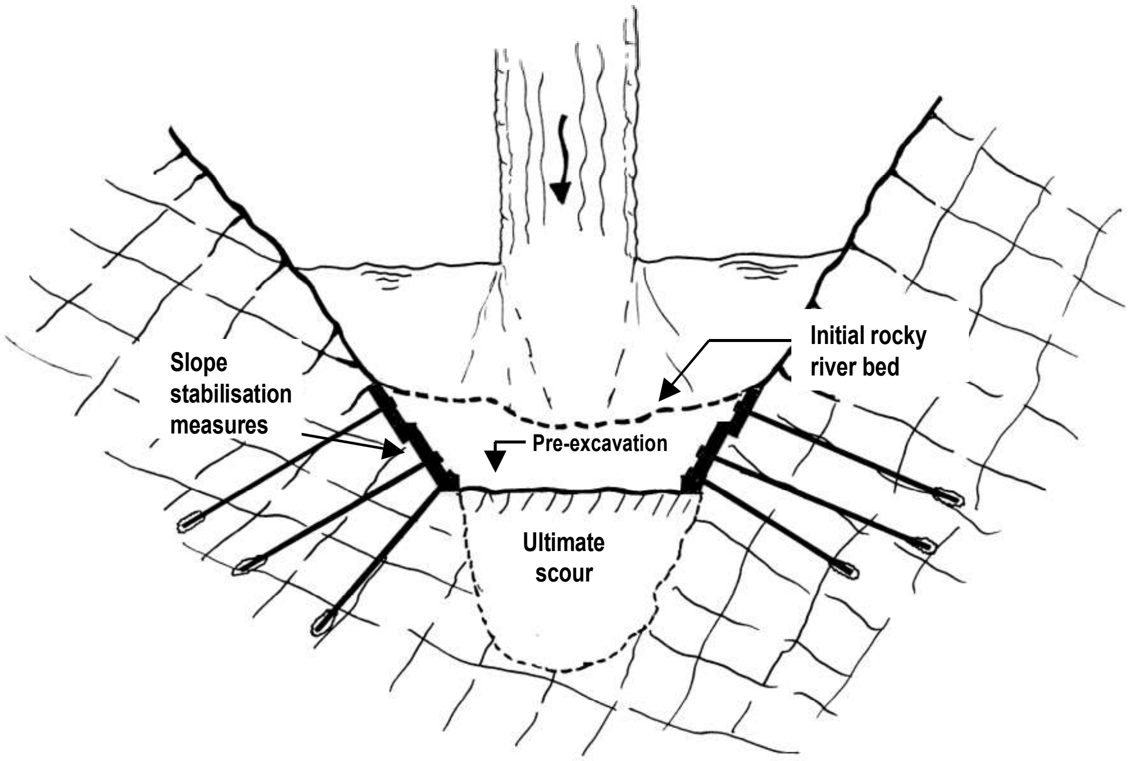


Figure 3.8. Pre-excavation of the plunge pool for a scour depth, which would be formed by a 50- to 100-year flood, and slope stabilization measures in case of ultimate scour formation in order to avoid abutment instabilities (Schleiss 2002).

For the well-known scour hole case at Kariba dam on Zambezi River, which has reached a depth of more than 80 m below the tail-water level, an innovative solution has been developed by reshaping the existing scour hole with an excavation mainly in downstream direction (Noret *et al*. 2013). This reduces significantly the dynamic pressures at water rock interface due to jet impact. With hybrid modelling techniques it could be proved that the reshaped scour hole will not progress anymore in future (Duarte *et al*. 2011).

3.6.7 Concrete lined plunge pools

If absolutely no scour formation in the rock downstream of a dam can is required, the plunge pool has to be reinforced and tightened by a concrete lining. Since the thickness of the lining is limited by construction and economic reasons, normally high tension or pre-stressed rock anchors are required to ensure the lining stability in view of the high dynamic loading. Furthermore the surface of the lining has to be protected with reinforced (wire mesh, steel fibers) and high tensile concrete having also high resistance against abrasion. Construction joints have to be sealed with efficient water stops. In addition the stability of the lining against static uplift pressure during dewatering of the plunge pool has to be guaranteed by a drainage system. This can also limit dynamic uplift pressures in case of limited cracking of the lining.

The design of plunge pools linings has to be based on the following sequence of events (Fiorotto & Rinaldo 1992, Fiorotto & Salandin 2000):

- pulsating pressures can damage the joint seals between slabs (construction joints);
- through these joint seals, extreme pressure values may propagate from the upper to the lower surface of the slabs;
- instantaneous pressure differentials between the upper and lower surfaces of the slabs can reach high values;
- the resultant force stemming from the pressure differential may exceed the weight of the slab and the anchor's resistance.

Furthermore the propagation of dynamic pressures through fissures in the lining reveals the presence of water hammer effects, which can amplify the pulsating uplift pressures underneath the concrete slabs (Bollaert 2002, Melo et al. 2006, Liu 2007, Fiorotto & Caroni 2007).

Since cracking of the plunge pool lining cannot be excluded, the assumptions of an absolutely tight lining and neglecting dynamic uplift are on the very unsafe side. If the high dynamic pressures can propagate through a small, local fissure from the upper to the lower surface of a concrete slab, dynamic pressures from underneath will lift the slab locally, which finally results in a progressive failure of the whole plunge pool lining. Thus the concept of a tight plunge pool lining is as risky as a chain concept: the system's resistance is given by the weakest link, i.e. the local permeability of the concrete slab.

Furthermore fluctuating pressures due to the plunging jets into the plunge pool are high compared to the proper weight of the slab and thus will result in vibration of the concrete slab and consequently in the development of cracks in the concrete and in possible fatigue failure of the anchors.

Cracking of a concrete slab before operation cannot be ruled out fully even using sophisticated construction joints and water-stops because of temperature effects when filling the plunge pool with water and of deformation of the underground. Therefore the design criteria of the plunge pool liner have to take into account the following:

- the load case of dynamic uplift pressures during operation;

- reinforcement in the concrete slab has to be designed for crack width limitation for possible dynamic vibration modes (depending on anchor pattern and stiffness);
- the grouted and pre-stressed rock anchors have to be designed for fatigue.

The drainage system under the plunge pool lining is of very high importance since it increases considerably the safety against dynamic uplift pressures. Nevertheless, since limited cracking of the lining cannot be excluded, as already mentioned, pressure waves can be transferred through the cracks into the drainage system. The response of the drainage system in view of these dynamic pressures with a wide range of frequencies at the entrance of a possible crack has to be controlled by a transient analysis, in order to be sure that no amplification of the pressures in the drainage system occurs (Mahzari et al. 2002). These amplifications of the pressures in the drainage system or any dynamic pressure underneath the slabs have to be considered in the design of the anchors (Mahzari & Schleiss 2010).

As a conclusion, since plunge pool linings are a risky concept as already mentioned before, a "belt-and-braces" approach for prototype design should be used with the following recommendations:

- slabs are sized generously so that localized instantaneous fluctuations average over larger area and the slabs will generally be reinforced;
- water bars are provided between slabs to prevent dynamic pressure communicating with the underside of the slabs;
- as a further precaution anchor bars are used and sized based on mean pressure differentials across different parts of the apron and on the assumption that some water bars may have failed;
- finally, generously sized under drainage is provided so that any pressure fluctuations feeding through small joints between the slabs can be dissipated in generously sized drainage zones.

3.7. CONCLUSIONS

Although the physical understanding of the scour process has considerably improved during the last 10 to 20 years, the scour evaluation still remains a challenge for dam designers. Scour models are now available which take into account the pressure fluctuations in the plunge pool and the propagation of transient water pressures into the joints of the underlying rock mass such as the Comprehensive Scour Method. It allows modelling the physical process of ejection of rock blocks due to dynamic uplift and the influence of air bubbles in erosion process in the plunge pool.

The rock parameters, which have to be assessed during the reconnaissance campaign for the dam foundation together with the hydrological conditions during the considered lifetime of the dam, influence the evolution of scour with time. The latter results in uncertainties which have to be overcome by engineering judgment.

In order to check and calibrate complex scour models, more detailed prototype data on scour evolution with fully documented discharge records are still needed. These observations are essential for a continuous safety assessment of a dam and to allow predicting the scour evolution.

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4. STEPPED SPILLWAYS

4.1. INTRODUCTION

The use of stepped chutes in dams and channels is not novel; it was common in ancient hydraulic constructions, especially those constructed with masonry (Chanson, 1994, 2002). The introduction of the Roller Compacted Concrete (RCC) technique on dams has revitalized the use and the study of stepped spillways. This modern period and the ancient period have in common that the stepped surface came as an output of the construction technique. The intense research that has been carried out in the modern time makes the difference between these periods. As a consequence of this, knowledge on hydraulic performance, cavitation risk, durability and energy dissipation has significantly improved.

Stepped spillways have two practical advantages: the first is that they fit very well with the RCC construction procedures, where formworks are placed vertically to provide workspace, resulting in vertical or stepped faces; and the second is that a higher energy dissipation is achieved due to the steps' influence on the flow, allowing to construct smaller energy dissipation structures.

The high energy dissipation rate that is produced by these chutes is now opening a new period, in which stepped spillways are designed regardless of the construction technique.

The main uses of stepped spillways are on RCC gravity dams (Fig. 4.1), where the downstream stepped face is used as the spillway; and on embankment dams with limited hydrological safety, for which an overtopping protection may be placed armouring the downstream shoulder (Fig. 4.2).

Figure 4.1.- La Breña II Dam (Spain). Gravity RCC dam. (ACUAES)

Figure 4.2.- Yellow River No.14 Dam (USA). Overtopping RCC spillway. (Golder Associates)

Another recent use of the stepped spillways is in combination with embankment dams, where stepped chutes excavated along the abutments are being constructed (Fig. 4.3). These may be lined or unlined, and which may have a variable geometry (step height and chute slope) according to the prevailing topography (Baumann *et al*., 2006).

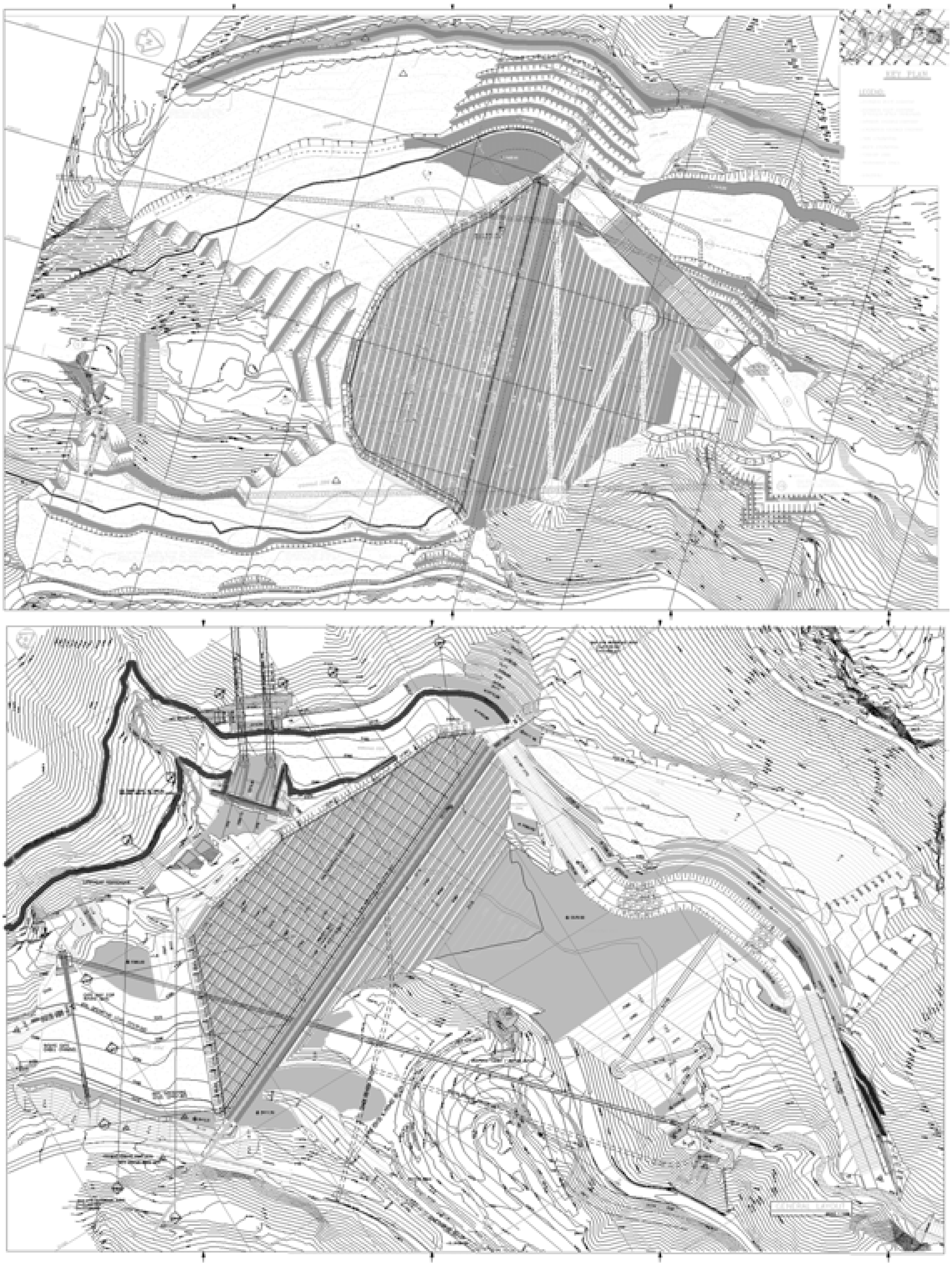

Figure 4.3.- Siah Bishe pumped storage power plant (Iran). Lower Dam (top) and Upper Dam (bottom). Stepped spillways along embankment dam abutments. (Courtesy of Prof. A.J. Schleiss)

There are also many older dams, which are still in use, where stepped masonry spillways were used in conjunction with embankment dams. These still require regular maintenance and assessment to ensure their continued successful use.

This chapter describes the main applications of stepped spillways (section 4.2), their hydraulics and expected performance (section 4.3) and the aspects that require specific attention during the design phase (section 4.4).

4.2. MAIN USES OF STEPPED SPILLWAYS

4.2.1. Stepped spillways on RCC gravity dams

One of the main uses of the stepped chutes is on RCC gravity dams. The RCC construction technique has proved itself as a time and cost-effective solution for developing large hydraulic projects. At the initial stage of the RCC technique many designs were conservative, with conventional vibrated concrete faces and smooth chutes used. Nowadays, most RCC gravity dams have stepped downstream faces as a result of the construction methods. The use of stepped chutes increased worldwide as it complements the construction method and because they provide higher energy dissipation.

Research on their hydraulic performance and experience gained on existing prototypes has contributed to consolidate the use of this type of spillways. According to the studies and the experience, the unit design flow has been increasing throughout the years (Table 4.1). Earlier designs were restricted to relatively small unit discharges (up to 10 $m^3/s/m$) (Sorensen, 1985), because of the uncertainty about the spillway performance and the risk of cavitation, but nowadays higher discharges are being considered. Boes (2012) indicates that designs for discharges below 30 $m^3/s/m$ can be considered conventional, while those exceeding that value require special attention.

The layout of these spillways is similar to that of smooth ones on typical concrete gravity dams: control structure, spillway channel and termination structure. However, the design of each component has to be adapted to the hydraulic performance of the stepped chute: the crest-to-chute transition has to be designed carefully to avoid flow deflection and spray, the sidewall height has to account for the flow bulking; the risk of cavitation has to be analysed and aeration provided if needed; and the terminal structure has to account for the air-water flow characteristics and the energy dissipation along the chute. These peculiarities are discussed further in section 4.4.

4.2.2. Overflow RCC stepped spillway on embankment dams

The overtopping protection of embankment dams with a RCC overlay is another important application of stepped spillways. They are typically used for existing dams with limited hydrological safety, where overflow spillways are designed for supplementing the service spillway to discharge large floods (Berga, 1995; PCA, 2002; FEMA, 2014; Toledo *et al*., 2015). In small dams (<15 m high) this alternative is cost-effective when compared to other measures, such as: enlarging of the existing spillway, constructing a new spillway separated from the dam body, or heightening the dam (Hansen, 2003; Bass *et al*., 2012). The statistics of 109 overflow RCC spillways in the USA (FEMA, 2014) show that these range in height from 5 to 20 m, and that the maximum height up-to-date is 35 m; and regarding the design unit discharge the average is below 3 $m^3/s/m$ and the maximum around 10 $m^3/s/m$.

Table 4.1.
Evolution of the design unit discharge of stepped spillways in gravity dams.
(Adapted and updated from Chanson (2002) and Matos (2003))

Dam (Country)	**Year**	**Design unit discharge ($m^3/s/m$)**	**Observations**
Monskvile (USA)	1986	9.3	
De Mist Kraal (South Africa)	1986	10.3	
Upper Stillwater (USA)	1987	11.4	
Les Olivettes (France)	1987	6.6	
Wolwedans (South Africa)	1990	12.4	
La Puebla de Cazalla (Spain)	1991	9.0	
Shuidong (China)	1994	100	Unconventional stepped spillway. Use of flaring piers to dissipate energy before the spillage over the stepped face.
Rambla del Boquerón (Spain)	1997	17.8	
Dona Francisca (Brazil)	2001	32	Maximum unit discharge.
Dachaosan (China)	2002	165	Unconventional stepped spillway. Use of flaring piers to dissipate energy before the spillage over the stepped face.
Pedrógão (Portugal)	2005	39,9	
La Breña II (Spain)	2008	22.3	Use of rounded edges in the crest-to-chute transition to reduce spray.
Boguchany (Russia)	2012	44	Unit discharge capacity for the Full Supply Level. Use of a large step (3.6 m) at the end of the piers to improve aeration.
Enlarged Cotter (Australia)	2013	48	Design unit discharge of the primary spillway. Use of a deflector -with air-inlets- to provide aeration.
De Hoop (South Africa)	2014	40	Maximum unit discharge. Use of triangular protrusions on the top third to improve aeration.
El Zapotillo (Mexico)	Under const.	38	Use of a large step to provide aeration.

This type of spillway consists of a concrete slab that armours the downstream slope and makes overflowing possible. The RCC technique is often used for constructing these slabs. RCC is typically placed in horizontal layers, and the downstream face, which constitutes the spillway, becomes stepped.

Although these spillways are also constructed with RCC, the design and construction conditions are different than those of gravity dams. Hence, the size and the finishing of the steps vary. The usual step height is 0.30 m (1 lift) and the facing may be formed or unformed. When unformed, the step face is not compacted and it has to be considered as a sacrificial concrete. Additionally, the hydraulic performance of the unformed protections would differ from the formed stepped ones and less energy dissipation would occur (FEMA, 2014). The lift width is determined by construction and structural requirements. Special care has to be taken with the uplift pressures that could occur beneath the slab, and a drainage system has to be provided. The minimum lift width is around 2.5 m when it is limited by the vibrating roller size. Larger width is required for large unit flows and for flat slopes, where a minimum slab thickness of 0.6 m is recommended (PCA, 2002).

The specific design peculiarities of these spillways, including the crest shape, the terminal structure or the sidewalls design are further discussed in section 4.4.

4.2.3. Stepped spillways along embankment dam abutments

The introduction of the stepped chutes in spillways located along the embankment dam abutments is a recent application that may be used in future projects with the aim of improving energy dissipation. This use benefits from the research about the hydraulic performance of stepped chutes on moderate and flat slopes (Ohtsu *et al*., 2001; 2004; André, 2004; González and Chanson, 2007; Meireles and Matos, 2009).

A characteristic of these spillways is the necessary adaptation of the chute slopes and the step height to the prevailing topography. In such cases the use of transition channels may be used to keep the step height constant over distances as long as possible (Figure 4.3) (Baumann *et al*., 2006; Boes *et al*. 2015).

4.2.4. Stepped masonry spillways

Prior to the era of modern concrete, stepped spillways constructed using masonry often accompanied the construction of earth embankment dams. The slopes of such spillways are typically in the range of 18.4° (3H/1V) to 5.7° (10H/1V). Major failures in the UK of two such spillways, at Boltby Dam in 2005 and Ulley Dam in 2007, led to investigations in their possible causes (Winter *et al*. 2010). Variations in the hydro-dynamic pressures on the sidewalls were highlighted as a significant vulnerability and the need for continued good maintenance of both the masonry and the mortar was highlighted as one essential aspect for these spillways. This is discussed in section 4.3.7.

4.2.5. Gabion structures and other stepped chutes

Stepped chutes are also found on small gabion dams. This type of dam has some interesting applications, among which are: check dams for solid detention and small weirs for slope correction and erosion prevention. Stepped gabion spillways may be also used on small agricultural reservoirs and dams with small catchment and low unit discharges (up to 3 $m^3/s/m$) (Peyras *et al.*, 1991, 1992; Rice and Kadavy, 1997). A practical further application is the use of porous gabion stepped chutes for noise abatement at municipal sites where low noise is crucial. (Boes and Schmid, 2003).

Another type of stepped chute results from the use of wedge-shaped blocks. Precast blocks are assembled overlapping each other, forming a stepped surface. The blocks benefit from the suction that is produced on the vertical face of the step, to improve its adherence with the foundation, resulting in slimmer pieces. It is a solution which has been used in small dams and agricultural ponds, with unit discharges of up to 4 $m^3/s/m$ (Pravdivets and Bramley, 1989; Hewlett *et al.*, 1997; FEMA, 2014; Morán and Toledo, 2014).

The stepped spillways directly excavated on rock should also be referenced. These features could be found on ancient hydraulic constructions and also in recent works as the spillways along the embankment dam abutments (Baumann *et al.*, 2006; Lutz *et al.*, 2015; Scarella and Pagliara, 2015). The chutes are characterised by larger, irregular steps, up to 2-3 m height or even more, which result from the necessary adaptation to the ground geology and morphology (Chanson, 1994, 2002; Felder and Chanson, 2011). In addition to the study of the hydraulic performance, it is important to check the rock resistance to erosion and scour, as the loss of material may produce changes in geometry and reduce the energy dissipated along the chute, leading to a poor spillway performance.

4.3. HYDRAULICS OF STEPPED SPILLWAYS

The hydraulic characterisation of flow is essential for the spillway design. The hydraulic design of any spillway requires the determination of the flow regime, depth, velocity distribution and residual energy. On stepped spillways it is also important to know where and how the aeration develops and to characterize the pressure field over the steps, in order to prevent cavitation and damage.

4.3.1. Flow regimes

The hydraulics of stepped spillways is driven by several variables with the most important being the critical depth (h_c) (which is a function of the unit discharge (q) and the cross-section), the step height (s) and the chute slope (Φ). The flow could be classified into three different types (Figure 4.4), which basically depend on the ratio between the flow depth and the step height. When this ratio is small (the step size predominates) the flow occurs as *nappe flow*, consisting of a succession of small cascades, with the stream falling from step to step (Essery and Horner, 1978). When the ratio is large (the flow depth is predominant) the flow is called *skimming flow*. This flow is characterised by the formation of a coherent main stream that skims over the edges of the steps (Essery and Horner, 1978; Sorensen, 1985; Rajaratman, 1990). The intermediate situation, when flow is unstable, partly nappe, partly skimming, is called *transition flow* (Ohtsu and Yasuda, 1997; Chanson and Toombes, 2004).

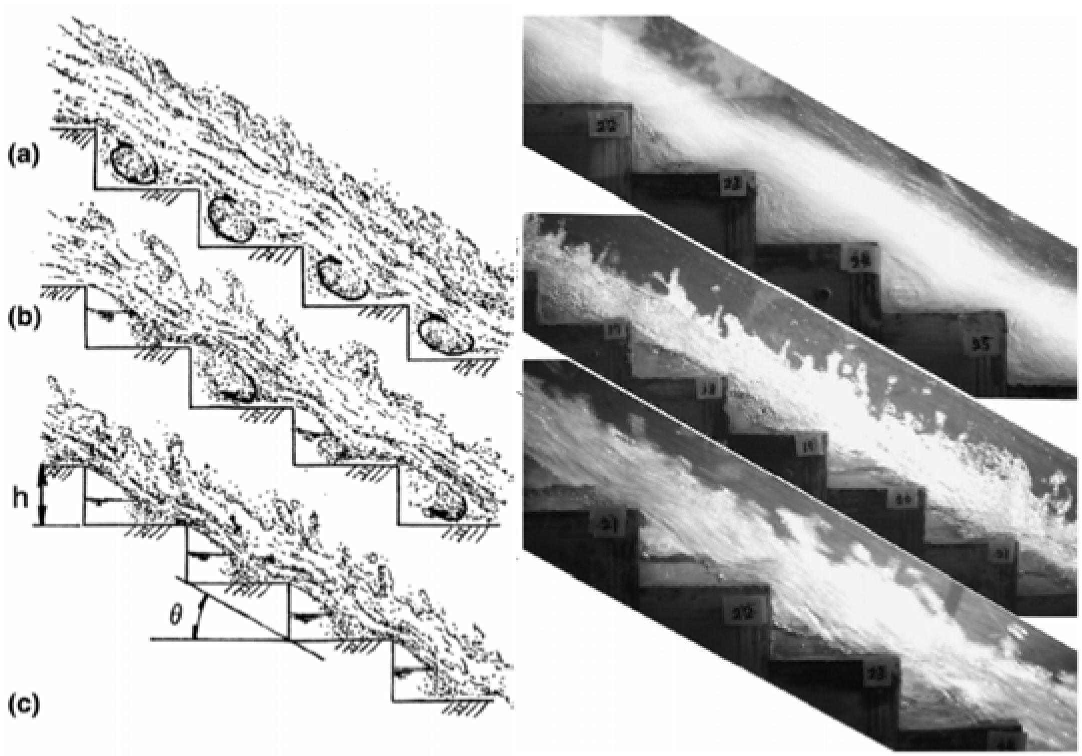

Figure 4.4.- Nappe flow (bottom), transition flow (centre) and skimming flow (top). (Othsu *et al.*, 2001)

Nappe flow develops when the critical depth is small in comparison to the step height for a given chute slope. In ungated spillways this is the flow that develops at the beginning and at the end of the routing process, and in times when the discharge is relatively small. When the flow depth is small in relation to the step size the flow jumps from one step to another. The jet impacts on the horizontal face of the step, causing the formation of a small hydraulic jump, prior to jump onto the next step, so that energy is gradually dissipated on each drop. The hydraulic jump could be fully or partially developed depending on the discharge and the step geometry (Chanson, 1994, 2002). The air pocket bounded by the vertical face of the step and the nappe of the falling jet is a characteristic of this flow type.

Skimming flow develops when the critical depth is larger than about 80% of the step height for a given chute slope (Essery and Horner, 1978). In such conditions, a main compact stream flows over the steps, which constitute a rough bottom. The cavity defined by the stream underside and the step faces, is completely filled with water. The shear stresses between the main flow and the water that fills the step lead to the formation of a recirculating vortex. A continuous momentum transfer feeds the vortex circular movement; this transfer is the main cause of the energy dissipation on stepped spillways.

Ohtsu *et al*. (2004) studied a wide range of slopes, from 5.7º (10H/1V) to 55º (0.7H/1V). They observed that skimming flow may be sub-classified into two categories. For the slopes between 19º (slope 2.9H/1V) and 55º (slope 0,7H/1V) the water surface is parallel to the pseudo-bottom defined by the step edges. This type of flow is the one which develops in most stepped spillways, those located on gravity dams and on many embankment dams. For the mildest slopes, from 5.7º (10H/1V) to 19º (2.9H/1V), the flow surface is not completely parallel to the pseudo-bottom, the horizontal face is so long that the influence of gravity on the flow causes a flow-deflection and an impact before the edge. This effect is reproduced by the water surface which has a wavy pattern.

A detailed description of the different types and subtypes of flow can be found in Othsu *et al.* (2004) and in González and Chanson (2007).

4.3.2. The use of skimming flow for design

As mentioned before, the flow on stepped chutes occurs either as nappe or skimming flow, with a transition regime between them. The determination of the limits and the conditions of each flow has been a traditional research topic. This characterisation is an important issue, since it is advisable to ensure that the design flow is distinctly nappe or skimming, but not transition flow. The transition regime must be avoided for design purposes, given that it is characterized by significant hydrodynamic instabilities, which may lead to high spray and poor spillway performance (Chanson and Toombes, 2004).

Both, the nappe and the skimming flow were studied in depth, but more analysis has been focussed on the skimming flow as it is usually selected for the hydraulic design. Under this condition –the design flood is discharged in the skimming regime– the extreme flood would also be spilled in the skimming regime. For smaller flows, which occur for lower recurrence floods and during the beginning and the end of the routing process, the spillage would be inevitably in nappe flow and possibly in transition flow. As a consequence of that, despite the adjustment of the design parameters for large floods, it is necessary to check whether a good hydraulic performance for smaller discharges takes place as well.

Typically the onset of the transition flow should occur near a 100-year recurrence interval. This is reasonable since for the more frequent floods the nappe flow regime with high dissipation energy will occur.

The development of either one type of flow or the other depends basically on the ratio between the step height and the critical depth, and also on the chute slope. The onset of the skimming flow may be estimated by:

(Boes and Hager, 2003b) For slopes between 26º<Φ<55º.

$$\frac{\mathrm{h_c}}{\mathrm{s}} = 0.91 - 0.14 \tan \Phi$$

(Ohtsu *et al.*, 2004) For slopes between 5.7º<Φ<55º.

$$\frac{\mathrm{h_c}}{\mathrm{s}} = 0.857\ (\tan \Phi)^{-0.1667}$$

(André, 2004) For slopes between 18.6º<Φ<30º.

$$\frac{\mathrm{h_c}}{\mathrm{s}} = 0.939\ (\tan \Phi)^{-0.364}$$

(Amador *et al.*, 2006) For gravity dams (Φ~51º).

$$\frac{h_c}{s} = 0.854\ (\tan \Phi)^{-0.169}$$

Table 4.2 shows the unit discharge threshold for the formation of skimming flow for typical slopes and step heights, using the referred methods.

Table 4.2.

Unit discharge (in $m^3/s/m$) for the onset of skimming flow, under different typical stepped spillway configurations.

Method and scope*		**Gravity dams**			**Embankment dams**			
		(0,8H/1V) φ=51.3°			**(1,5H/1V) φ=33.7°**	**(2H/1V) φ=26.6°**	**(3H/1V) φ=18.4°**	**(4H/1V) φ=14.0°**
		s=0.9 m	**s=1.2 m**	**s=1.5 m**	**s=0.3 m**	**s=0.3 m**	**s=0.3 m**	**s=0.3 m**
Boes and Hager (2003b)	26°<φ<55°	1.7	2.6	3.6	0.4	0.4	---	---
Ohtsu et al. (2004)	5.7°<φ<55°	2.0	3.1	4.3	0.4	0.5	0.5	0.6
André (2004)	18.6°<φ<30°	---	---	---	---	0.7	0.9	---
Amador (2006)	φ~51°	2.0	3.1	4.3	---	---	---	---
* As defined by authors.								

Among the three variables which influence the flow type, two of them (i.e. step height and chute slope) are usually difficult to modify since they are conditioned by other factors such as the construction method, dam type or ground morphology. Therefore, to adjust the flow type, it would be necessary to modify the critical depth, which in turn may be adjusted by changing the crest length.

4.3.3. Location and characteristics of the inception point

Two regions differing in aeration are distinguished for skimming flow (Fig. 4.5). In the upstream part of the spillway, nearby the crest, the flow is non-aerated, this region is also known as black-water or clear-water region. Downstream, typically in the medium and lower parts of the chute, the flow is aerated, this region is also named white-water region. In the non-aerated region the flow features a compact, transparent look, without bubbles. The approach flow to the spillway crest is laminar. The turbulent boundary layer starts growing from the crest. When the boundary layer reaches the surface, the air-water friction is large enough to cause surface irregularities, creating air pockets that are incepted and rapidly distributed within the flow (Amador *et al.* 2009). Downstream of this *inception point* the flow is aerated.

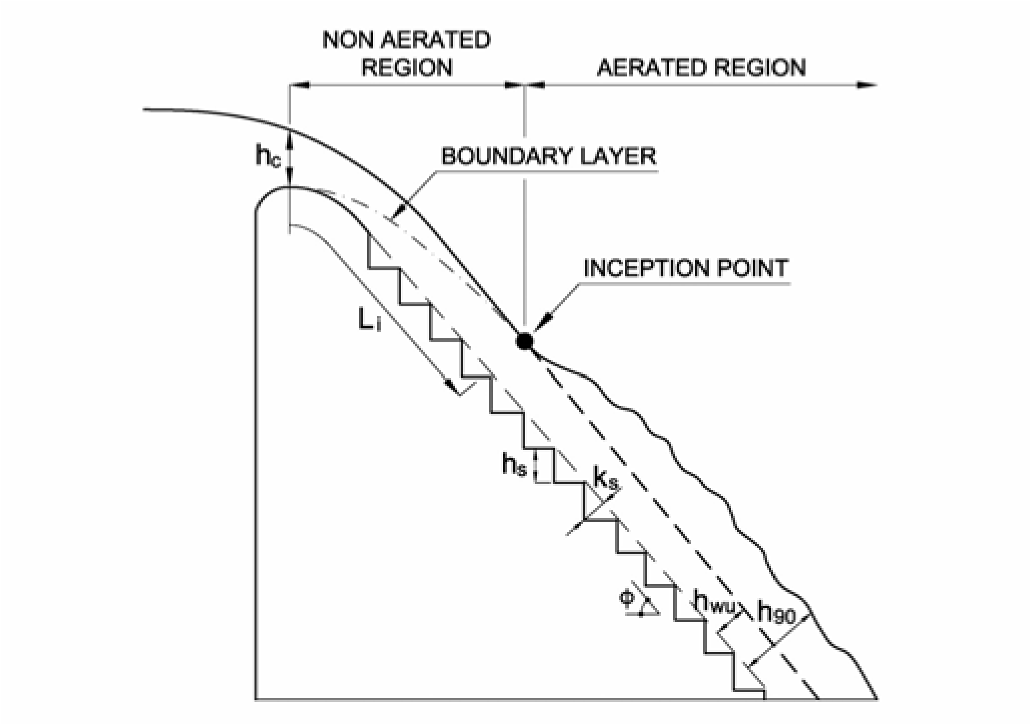

Figure 4.5.- Skimming flow development. Non-aerated and aerated regions. Design variables.

In turn, three different regions may be differentiated within the aerated zone regarding aeration and flow development (Ohtsu *et al*., 2001; Amador *et al*. 2006) (Fig. 4.6). In the first, located immediately downstream of the inception point, the air is rapidly distributed by the turbulence effect and the flow is rapidly varied. This region lasts until the aeration process stabilizes. The flow in the second aerated region is gradually varied until quasi-uniform flow is attained, which constitutes the third aerated flow region.

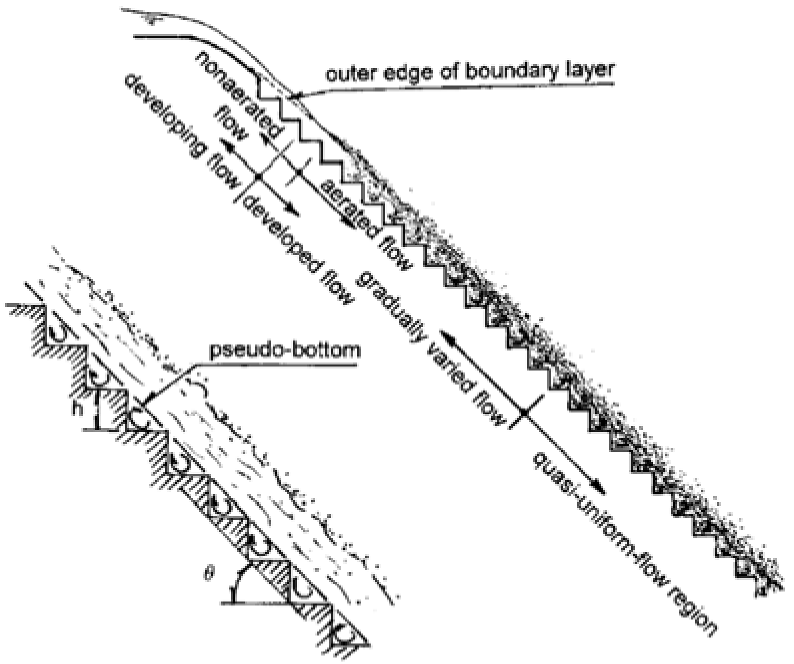

Figure 4.6.- Sub-regions of the aerated zone. (Ohtsu *et al.*, 2001)

As explained above, the inception point defines where the air-entrainment process begins. Upstream of this point the flow is un-aerated. The location of the inception point is important to assess the cavitation risk. The area which is more prone to develop cavitation is around the inception point, as it is the non-aerated zone with the highest flow velocities and therefore the highest sub-atmospheric pressures. Downstream of this area the risk of cavitation is not significant, since the flow is highly aerated.

Some equations which may be used for determining the location of the inception point are shown below.

(Boes and Hager, 2003a) for slopes between 26º<Φ<55º:

$$L_i = \frac{5.90\, h_c^{6/5}}{(\sin\Phi)^{7/5}\ s^{1/5}}$$

where L_i is the distance from the crest to the inception point.

The analysis of this formula illustrates the influence of the different factors on the non-aerated region. To be precise, larger critical depths (say larger unit flows) lead to larger distances of the non-aerated flow, milder slopes produce larger distances (for instance, embankment dams have larger un-aerated areas than gravity dams), and smaller steps results also in larger distances. The location of the inception point is more sensitive to the critical depth and to the chute slope than to the step height.

(Amador *et al.*, 2009) for gravity dams (Φ~51º):

$$\frac{L_i}{K_s} = 5.982\, F_*^{0.84}$$

where K_s is the bottom roughness, that is computed as the step height in the stream direction; and F_* is the roughness Froude number:

$$K_s = s\cos\Phi$$

$$F_* = \frac{q}{\sqrt{g\sin\Phi\ K_s^3}}$$

(Meireles *et al.*, 2012) for gravity dams (Φ~53º):

$$\frac{L_i}{K_s} = 6.75\,F_*^{0.76}$$

(André, 2004) for spillways over embankment dams (18.6º<Φ<30º):

$$\frac{L_i}{K_s} = \frac{8}{\tan\Phi}\,F_{*\Phi}^{0.73}$$

Where $F_{*\phi}$ is the roughness Froude number corrected for accounting with the slope effect:

$$F_{*\Phi} = \frac{q}{\sqrt{g\cos\Phi\ K_s^3}}$$

(Meireles and Matos, 2009) for spillways over embankment dams (16º<Φ<26.6º):

$$\frac{L_i}{K_s} = 5.25\,F_*^{0.95}$$

Hunt and Kadavy (2013) have analysed the influence of the roughness Froude number, noting that for large discharges (F_*>28) the chute has a different performance; in which the effect of the steps is less as the distance increases to the location of the inception point from the crest. These authors proposed the following equations for broad crested stepped spillways over embankment dams (Φ≤26.6º):

$$\frac{L_i}{K_s} = 5.19\,F_*^{0.89} \qquad 0.1<F_*<28$$

$$\frac{L_i}{K_s} = 7.48\,F_*^{0.78} \qquad 28<F_*<10^5$$

Table 4.3 includes L_i values for typical dam configurations (slopes and step heights) and unit discharges, using the referred methods.

The flow depth at the inception point (h_i) has also been analysed in different studies, with some of the formulas proposed are:

(Boes and Hager, 2003a) for slopes between 26º<Φ<55º:

$$\frac{h_i}{s} = 0.40\,F_{*1}^{0.6}$$

Note that these authors defined the roughness Froude number (F_{*1}) using the step height (s) instead of the bottom roughness (K_s).

$$F_{*1} = \frac{q}{\sqrt{g\sin\Phi\ s^3}}$$

Table 4.3.

Location of the inception point (in m) from the spillway crest, under different typical stepped spillway configurations and for different unit discharges.

Method and scope*		**Gravity dams**			**Embankment dams**					
		(0.8H/1V) φ=51.3º s=1.2 m			**(2H/1V) φ=26.6º s=0.3 m**			**(3H/1V) φ=18.4º s=0.3 m**		
		q=5 m³/s/m	**q=10 m³/s/m**	**q=20 m³/s/m**	**q=1 m³/s/m**	**q=3 m³/s/m**	**q=5 m³/s/m**	**q=1 m³/s/m**	**q=3 m³/s/m**	**q=5 m³/s/m**
Boes and Hager (2003b)	26º<φ<55º	11.6	20.3	35.5	9.3	22.3	33.4	---	---	---
Amador et al. (2009)	φ~51º	10.6	18.9	33.9	---	---	---	---	---	---
Meireles et al. (2012)	φ~53º	11.0	18.6	31.5	---	---	---	---	---	---
André (2004)	18.6º<φ<30º	---	---	---	8.3	18.4	26.8	12.1	26.9	39.0
Meireles and Matos (2009) (1)	16º<φ<26.6º 1.9<F_*<10	---	---	---	4.6	13.1	---	5.3	15.0	---

* As defined by authors.

(1) This formula has been developed with a broad crested weir model and the location of the inception point (L_i) is referred to the downstream edge of the broad crested weir.

(Amador *et al.*, 2009) for gravity dams (Φ~51º):

$$\frac{h_i}{K_s} = 0.385\, F_*^{0.58}$$

(Meireles and Matos, 2009) for spillways over embankment dams (16º<Φ<26.6º):

$$\frac{h_i}{K_s} = 0.28\, F_*^{0.68}$$

After Boes and Hager (2003a) the averaged air concentration at the inception point ($\overline{C_i}$) could be determined as:

$$\overline{C_i} = 1.2 \cdot 10^{-3}(240° - \Phi)$$

4.3.4. Attainment and characteristics of uniform flow

As aforementioned, the aerated region may be divided into three areas depending on the aeration and the flow development (Fig. 4.6). The first, located immediately downstream of the inception point, is where the aeration process develops, and where the flow depth increases significantly due to the highly turbulent aeration process. Downstream of this region the stream is a two-phase flow emulsion of air and water (Boes and Hager, 2003a) where the flow gradually varies towards the becoming uniform flow. The mixture flow depth increases progressively until reaching a constant value under uniform flow conditions. Uniform flow may not be reached in some cases as the chute may not be long enough. Such condition could be written in terms of the chute height (H_{chute}) as:

(Boes and Hager, 2003b) required relative chute height (H_{chute}/h_c) to reach uniform flow:

$$\frac{H_{chute}}{h_c} \sim 24\ (\sin\Phi)^{2/3}$$

In accordance with this equation, the steeper the slope is, the higher the chute should be to attain uniform flow. For instance, the relative chute height to reach uniform flow on a typical gravity dam (slope 0.8H/1V; Φ= 51.34º) is H_{chute}/h_c = 20.35; while for an embankment dam (slope 3H/1V; Φ= 18.43º) it is 11.14. Hence, spillways over gravity dams require larger chute heights to attain uniform flow than spillways overlying embankment dams, for a given unit discharge.

The equivalent clear water depth of uniform flow (h_{wu}) (i.e. equivalent to the flow depth on a smooth conventional chute) may be determined as follows:

(Boes and Hager, 2003b)

$$\frac{h_{wu}}{h_c} = 0.215\ (\sin\Phi)^{-1/3}$$

Skimming flow on stepped spillways is highly aerated, resulting in larger depths than those which correspond to a smooth chute. The high turbulence in skimming flow makes it difficult to exactly determine the air-water mixture flow depth. To cope with this limitation the depth where the local air concentration is 90% is considered as the characteristic mixture flow depth (h_{90}) (Chamani and Rajaratnam, 1999). This variable, that considers flow bulking, is often used for the sidewall design, including a safety freeboard to avoid lateral spilling. This is discussed further in section 4.4.3. The characteristic mixture flow depth can be determined with the following methods:

(Chanson, 1994, 2002) Characteristic mixture depth in uniform flow (h_{90u})

$$h_{90u} = h_c \cdot \sqrt[3]{\frac{f_e}{8\,(1 - C_e)^3\ \sin\Phi}}$$

where f_e is the equivalent Darcy friction factor for the air-water mixture and C_e is the equilibrium air concentration for uniform flow:

$$\frac{f_e}{f} = 0.5 \cdot \left(1 + \tanh\left(2.5 \cdot \frac{0.5 - C_e}{C_e(1 - C_e)}\right)\right)$$

$$C_e = 0.9 \sin\Phi \text{ (for } \Phi\text{<50º)}$$

where f is the Darcy friction factor for non-aerated flows, which could be computed as:

$$\frac{1}{f} = 1.42\ \ln\left(\frac{D_h}{K_s}\right) - 1.25$$

with D_h being the hydraulic diameter $D_h = 4A/P$; where A is the cross-sectional area of flow and P the wetted perimeter.

(Boes and Hager, 2003b) characteristic mixture depth in uniform flow (h_{90u}):

$$\frac{h_{90u}}{s} = 0.5\, F_{*1}^{(0.1\tan\Phi + 0.5)}$$

An approximation of the differential equation of the backwater curve by Boes and Minor (2000) could be used for computing the characteristic mixture flow depth (h_{90}) in the gradually varied flow region:

$$h_{90}(x) = 0.55\left(\frac{q^2 s}{g\sin\Phi}\right)^{1/4} \cdot \tanh\left(\frac{\sqrt{gs\sin\Phi}}{3q}\cdot(x - L_i)\right) + 0.42\left(\frac{q^{10}s^3}{(g\sin\Phi)^5}\right)^{1/18}$$

where x is the distance along the spillway from the crest.

A review into the hydraulics of stepped chutes by the UK Environment Agency (Winter *et al.*, 2010) indicated that the Chanson approach is likely to give the most accurate results for the relatively shallow sloping chutes associated with earth embankment dams, while the Boes and Hager, as well as the Boes and Minor, equations are applicable to the more steep sloping chutes associated with RCC dams and for which they were originally developed.

4.3.5. Energy dissipation

Friction factor

The large energy dissipation rate provided by stepped spillways is one of the main advantages of stepped spillways. A significant part of the initial head is dissipated as a consequence of the momentum transfer to the vortices that form between the steps and the pseudo-bottom. This is fundamental for the design of the dissipation structures, thus several research studies and analyses have been carried out regarding this topic.

Energy dissipation on a chute depends on the friction factor of bottom roughness (f_b). In a stepped chute this is related to the bottom roughness (K_s) and the spacing between the step edges. The factor of friction could be determined analytically using the Darcy-Weisbach formula by making a shape correction to a rectangular open channel, and deducting the lateral effect of the smooth sidewalls. Apart from this approach, it should be noted that, adjusting a friction factor formula for stepped spillways is complex due to the highly turbulence in which the two-phase flow develops. Boes and Hager (2003b) proposed the following equation to determine the friction factor bottom roughness for slopes between 19º < Φ < 55º.

$$f_b = (0.5 - 0.42\sin(2\Phi))\left(\frac{K_s}{D_h}\right)^{0.2}$$

When $0.1 < K_s/D_h < 1.0$, this equation may be rewritten as:

$$\frac{1}{\sqrt{f_b}} = \frac{1}{\sqrt{0.5 - 0.42\sin(2\Phi)}}\left(1.0 - 0.25\log\left(\frac{K_s}{D_h}\right)\right)$$

Another important feature that should be taken into account when studying the friction factor is the influence of aeration. The air bubbles have a lubricant effect, limiting the shear stresses between the main stream and the recirculating vortices; hence reducing the energy dissipation that is achieved along the chute (Chanson, 1994). Accounting for this effect Boes and Hager (2003b) recommended to compute the hydraulic diameter using the equivalent clear water depth (h_w) instead of the characteristic flow depth (h_{90}). This parameter is named D_{hw}. According to these authors the use of the characteristic flow depth may lead to large friction factors, which in turn may produce an overestimation of the energy dissipation and a non-conservative design of the terminal structure.

<u>Residual energy</u>

The energy dissipation depends on the flow type. Mateos and Elviro (2000) found that the energy dissipation rate becomes significant once the flow is close to attaining the uniform flow. According to these authors for relative chute heights $H_{chute}/h_c < 10$, energy dissipation is similar to that of smooth chutes.

Boes and Hager (2003b) proposed two formulas for calculating the residual energy (H_{res}) depending on the type of flow attained:

(Boes and Hager, 2003b) for $H_{chute}/h_c < 15\text{-}20$ (i.e. uniform flow is not attained):

$$\frac{H_{res}}{H_{max}} = \exp\left[\left(-0.045\left(\frac{K_s}{D_{hw}}\right)^{0.1}(\sin\Phi)^{-0.8}\right)\frac{H_{chute}}{h_c}\right]$$

where H_{max} represents the maximum head –potential energy of water in the reservoir– which could be estimated as $H_{max} = H_{chute} + 1.5h_c$.

(Boes and Hager, 2003b) for $H_{chute}/h_c \geq 15\text{-}20$ (i.e. uniform flow is attained):

$$\frac{H_{res}}{H_{max}} = \frac{E}{\frac{H_{chute}}{h_c} + E}$$

with E equal to:

$$E = \left(\frac{f_b}{8\sin\Phi}\right)^{1/3}\cos\Phi + \frac{\alpha}{2}\left(\frac{f_b}{8\sin\Phi}\right)^{-2/3}$$

where α is the kinetic energy corrector coefficient, which could be assumed as α =1.1.

According to André (2004) the residual energy at the base of a moderate sloped chute (Φ=30º) is given by:

$$\frac{H_{res}}{H_{max}} = \frac{1.5\ \tan\Phi}{10}\exp\left(25.26\frac{h_c}{N_s \cdot s}\right)$$

where N_s is the number of steps.

Table 4.4 summarizes the energy dissipation potential for typical dam configurations (slopes and step heights), dam heights and unit discharges.

Table 4.4.

Normalized residual energy (H_{res}/H_{max}) and energy dissipation rate (%), under different typical stepped spillway configurations and for different unit discharges.

Chute height	**Gravity dams (0.8H/1V) φ=51.3º**								
	s=0.9 m			**s=1.2 m**			**s=1.5 m**		
	q=5 m³/s/m	**q=10 m³/s/m**	**q=20 m³/s/m**	**q=5 m³/s/m**	**q=10 m³/s/m**	**q=20 m³/s/m**	**q=5 m³/s/m**	**q=10 m³/s/m**	**q=20 m³/s/m**
H_{chute}=30 m	0.32 68%	0.52 48%	0.68 32%	0.32 68%	0.51 49%	0.68 32%	0.31 69%	0.51 49%	0.67 33%
H_{chute}=50 m	0.22 78%	0.33 67%	0.52 48%	0.22 78%	0.32 68%	0.51 49%	0.21 79%	0.31 69%	0.50 50%
H_{chute}=100 m	0.13 87%	0.19 81%	0.28 72%	0.12 88%	0.19 81%	0.28 72%	0.12 88%	0.18 82%	0.28 72%
H_{chute}=150 m	0.09 91%	0.14 86%	0.21 79%	0.08 92%	0.13 87%	0.21 79%	0.08 92%	0.13 87%	0.20 80%
Method: Boes and Hager (2003b) The grey cells indicate that the uniform flow is attained at the chute end.									

Chute height	**Embankment dams s=0.3 m**							
	(3H/1V) φ=18.4º			**(2H/1V) φ=26.6º**			**(3H/1V) φ=26.6º**	
	q=1 m³/s/m	**q=3 m³/s/m**	**q=5 m³/s/m**	**q=1 m³/s/m**	**q=3 m³/s/m**	**q=5 m³/s/m**	**q=0,20 m³/s/m**	**q=0,30 m³/s/m**
H_{chute}=15 m	0.09 91%	0.19 81%	0.35 65%	0.14 86%	0.26 74%	0.45 55%	0.10 90%	0.11 89%
H_{chute}=30 m	0.05 95%	0.10 90%	0.14 86%	0.07 93%	0.15 85%	0.21 79%	0.09 91%	0.09 91%
H_{chute}=50 m	0.03 97%	0.06 94%	0.09 81%	0.04 96%	0.10 90%	0.14 86%	0.08 92%	0.08 92%
Method	Boes and Hager (2003b)			Boes and Hager (2003b)			André (2004)	
In the calculations done with the Boes and Hager equations (2003b), the grey cells indicate that the uniform flow is attained at the chute end.								

Energy dissipation could be enhanced introducing end-sills, blocks and protrusions. André *et al*. (2003; 2004) studied the effect of end-sills and blocks on stepped spillways and concluded that the latter can increase energy dissipation by 5 to 8% and also reduce the risk of cavitation at the beginning of the chute. Wright (2010) has investigated the use of triangular protrusions, obtaining similar conclusions. This type of block, which has been used in De Hoop Dam in South Africa, has been shown to be an effective measure to increase the friction factor, and to improve aeration and energy dissipation.

4.3.6. Pressure field on the steps

Another important issue of stepped spillways hydraulics is the description of the hydrodynamic pressure field. It is important to identify the areas with cavitation risk. The works by André *et al*. (2004), Amador *et al*. (2006), Sánchez-Juny *et al*. (2007) and Amador *et al*. (2009) describe in detail the development and fluctuations of the pressure field on the horizontal and vertical faces of the steps, and provide equations for their computation on gravity dams, both on the horizontal and vertical faces. These two regions which have to be distinguished to describe the pressure field are the external zone located near the step edge, which is influenced by the main stream; and the internal part located around the inside corner, which is most influenced by the recirculating vortices.

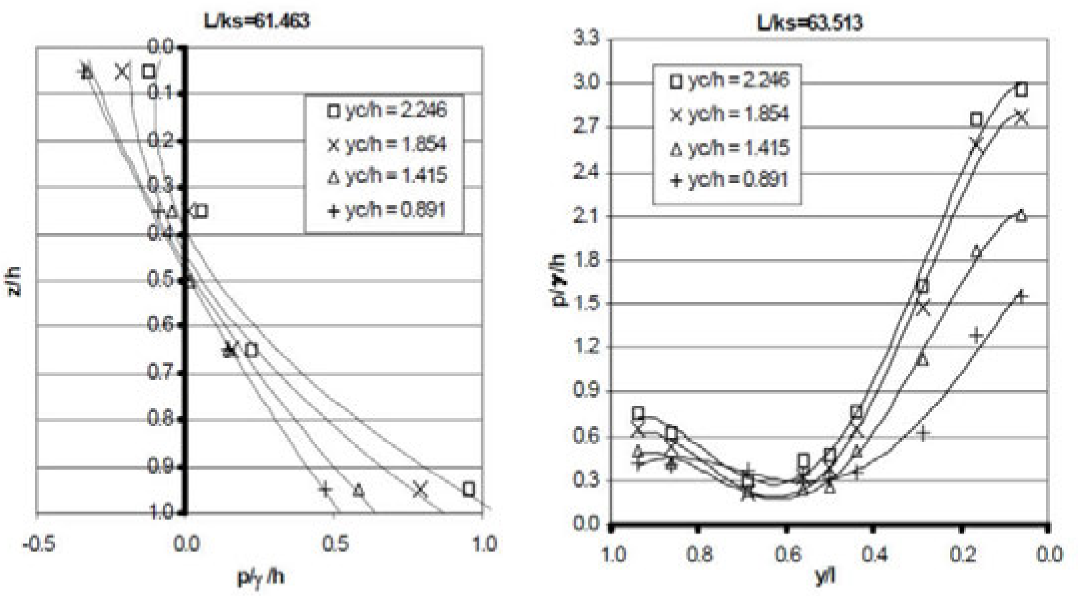

Figure 4.7.- Profiles of mean pressure on the vertical and the horizontal faces of the steps. (Sánchez-Juny *et al.* 2007)

On the horizontal face the maximum pressure is located on the external zone, due to the impact of the main flow, which is influenced by gravitational action, resulting in a pressure decrease towards the interior of the step (Fig. 4.7). On the vertical face the minimum pressure is located on the external zone, due to the drag effect caused by the main stream. Pressure has a positive gradient to the interior corner, where its value is similar to that of the horizontal face (Fig. 4.7). Fluctuation on both faces is very high, being more pronounced near the step edges and for higher discharges.

Regarding the evolution of the pressure field along the spillway, researchers agree that the most delicate area is that located in the non-aerated region, while the aerated region is not prone to cavitation damage (Zhang *et al*., 2012). The minimum and maximum pressures were recorded around the inception point; downstream of this zone the presence of air bubbles in the flow produces a cushion effect that reduces the pressure and its fluctuation (Amador *et al*. 2005). Research regarding the design of aerators at the beginning of stepped spillways is ongoing.

It has to be noted that pressure measurements performed on scaled stepped spillway scale models do not show high frequency pressures as they occur on prototype spillways. Therefore model results should be transferred to prototype spillways with care and only if having similar slopes because of scale effects. Scale effects could also impact other variables such as the air concentration and, in turn, influence the friction factor and the residual energy estimation. Boes (2000) suggested a minimum Froude scale between 1:10 and 1:15 for hydraulic models to minimise these scale effects.

4.3.7. Pressure field on the sidewalls

As mentioned in section 4.2.4, the collapse of two old stepped masonry spillways in the UK, at Boltby Dam in 2005 and at Ulley Dam in 2007, led to a research programme into hydrodynamic effects on stepped chutes (Winter *et al*., 2010; Winter, 2010). These studies revealed that at high skimming flows the centres of the horizontal vortices at each step can generate high negative pressures which in turn act directly on the associated local zones of the sidewall. Moreover this will occur adjacent to other wall zones subject to high positive pressures due to step impact (Fig. 4.8). Poor maintenance and cracks in the mortar can, therefore, lead to the high back-pressurisation of masonry in zones subject to external negative pressures. This results in blocks being "plucked" from the wall, a local turbulence increase and, in extreme cases, complete wall collapse.

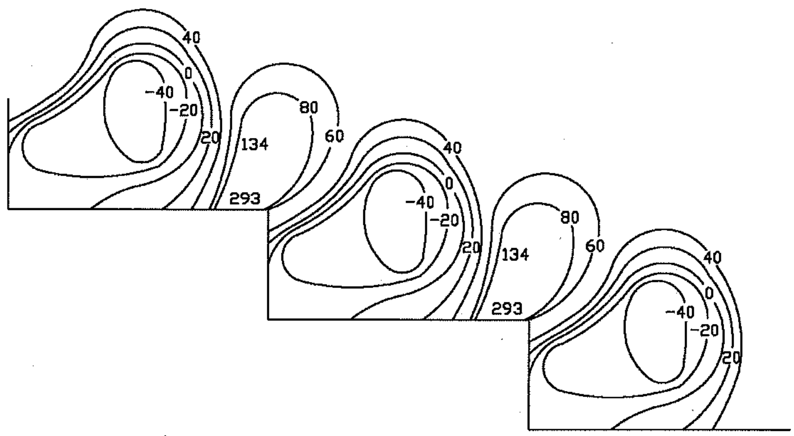

Figure 4.8.- Hydrodynamic pressure variation on the sidewall of a typical stepped chute under skimming flow conditions. (Winter *et al*., 2010)

4.4. DESIGN FEATURES

4.4.1. Step height

There is an optimal relation between the step height and the energy dissipation that is around $s/h_c \geq 0.3$ (Tozzi, 1992) and $s/h_c \geq 0.25$ (Othsu *et al.* 2004). However, it is important to remark that the step height is mainly determined by the existing construction techniques. On RCC stepped spillways the step height is a multiple of the concrete layer thickness. The standard thickness is 0.3 m. Initially smaller thicknesses were used on the pioneering RCC dams, but nowadays 0.3 m is predominant. For example: 23 out of the 27 existing RCC dams in Spain were constructed using 0.3 m thick layers (de Cea *et al.*, 2012). In gravity dams it is usual to lift the formwork every four layers, resulting in steps heights of 1.2 m, whereas, in embankment overtopping RCC protection the most usual step height is 0.3 m. This is due to the fact that smaller steps adapt better to milder slopes (otherwise the step length would be too large and concrete is wasted) and because the steps are not always formed. As the step heights are largely influenced by the construction technology, layer thickness and step height that are common nowadays may slightly change in future depending on further RCC development.

New developments where the stepped slope does not come as a direct output of the construction method, such as in conventional gravity dams, like the Boguchany Dam in Russia (Bellendir *et al.*, 2012), or in side spillways of embankment dams like the Siah Bishe Lower and Upper Dams in Iran (Baumann *et al.*, 2006), step heights are not determined by the construction technique and may be more flexible to be adjusted for maximising energy dissipation.

Some authors indicate that the step height may be adjusted during the design phase to ensure that design flow is discharged distinctively as skimming flow. It should be emphasised that the step height is not the most adequate parameter to do so, since it is highly dependent on the construction method. As discussed in section 4.3.2, the most flexible parameter is the critical depth, which could be modified by means of changing the crest length (if possible).

4.4.2. Crest shape and transition to chute

Most gravity dam spillways have an ogee crest (as WES or Creager). This crest shape fits very well with the typical triangular cross section and slopes of gravity dams. The transition between the crest and the spillway chute should be designed in such a way to prevent flow deflection and impact on the initial steps, which later results in spray and bad performance for small discharges, and cavitation risk for the larger flow. The work carried out by CEDEX (Madrid) largely focussed on that design issue. Mateos and Elviro (1995; 2000) proposed a stepped transition which follows the crest profile, by means of steps of increasing height and length (Fig. 4.9).

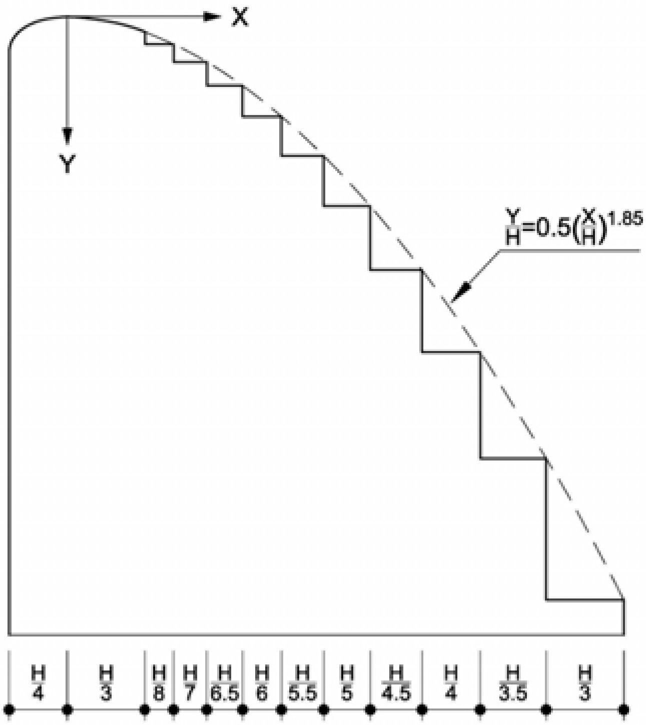

Figure 4.9.- CEDEX's proposal of crest-to-slope transition, with H being the design head on spillway crest. (Mateos and Elviro, 1995)

Another alternative that has been proposed to reduce impact and spray is modifying the impact angle (Pfister *et al*., 2006a). When impact occurs, the flow is deflected with an angle that is approximately symmetrical to the impacting flow. Thus, chamfering the edge of the first steps reduces spray. This action may also be reproduced by means of an insert or rounding the steps edges, as has been done on the first three steps of La Breña II Dam (Fig. 4.10) (Elviro and Balairón, 2008).

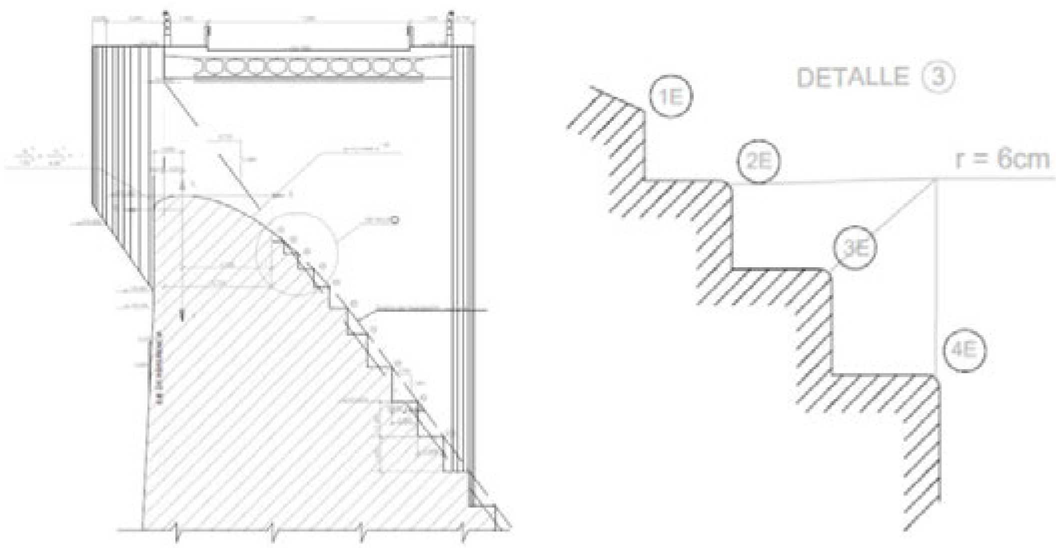

Figure 4.10.- La Breña II Dam (Spain). Use of rounded edges for reducing spray. (Elviro and Balairón, 2008)

Other crest alternatives for RCC gravity dams are broad-crested weirs, precast ogee-type weirs and Piano Key Weirs. Those infrequent alternatives may be considered when special conditions, such as economic, time or hydrological restrictions, are required to be met. In these cases performance of those alternatives should be checked with specific studies.

In the case of stepped spillways over embankment dams it is common to make use of broad-crested weirs over the dam crest (Hansen, 2003). This alternative fits well with the trapezoidal cross section of embankment dams (Fig. 4.11). Although the discharge capacity of this crest-type is smaller, that feature is not usually a problem, as it is cost-effective in most cases to increase the spillway width. Typically, these spillways span over the entire dam. If larger discharges are required a sharp crested weir, a labyrinth-type weir or an ogee weir may be used (Bass *et al.* 2012). The transition for these types of spillways has not been an object of specific research. On usual designs the stepped chute directly starts after the crest (Fig. 4.11). The problems of jump and spray that arise on gravity dam spillways do not normally receive specific attention, due to the fact that these overtopping protections are in many cases emergency spillways with low unit discharges.

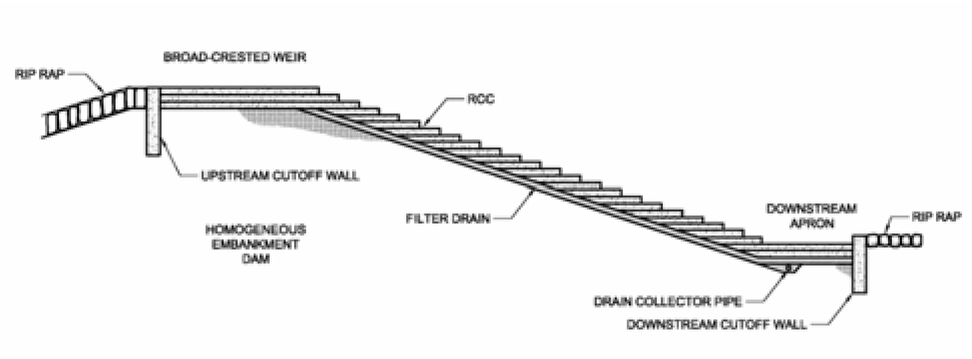

Figure 4.11.- Typical cross-section of an embankment dam with RCC overtopping spillway. (Adapted from Bass *et al*. (2012))

Gated stepped spillways are not common since they usually imply larger unit discharges, thus many gated spillways on RCC dams have smooth chutes. Specific design features such as enlarging the smooth part to fully cover the non-aerated flow region (Amador *et al*., 2006) or providing aeration (Guo *et al*. 2003) should be considered on gated spillways.

Dachaosan Dam (China) spillway is a gated stepped spillway which has been designed for a very large unit discharge (165 $m^3/s/m$). The upstream part of this spillway is smooth and the air entrainment is ensured by means of flaring piers in conjunction with a small flip bucket. The flaring piers concentrate the flow, turning it into a vertical position, achieving a high lateral aeration due to friction with air. Dachaosan Dam has discharged up to 6.173 m^3/s, a unit discharge of 93 $m^3/s/m$, showing good performance and no significant damages reported (Shen, 2003). This type of energy dissipater is an interesting alternative for large projects that has been successfully used in other high head/velocity spillways in China.

4.4.3. Cavitation risk and aeration measures

The zone prone to cavitation on stepped spillways is that located on the non-aerated flow region. The most delicate area is the surroundings of the inception point, since the aeration process is not fully developed and velocity is larger than upstream. Boes and Hager (2003a) establish an upper limit for the velocity at the inception point of 20 m/s, based on the air-concentration requirements to avoid cavitation near the inception point. Amador *et al*. (2005) determined a lower velocity threshold upon the inception point of 15 m/s, based on the analysis of the pressure field. Frizell *et al.* (2013) indicate that no cavitation problems or severe damage were reported on stepped spillways to date; hence those limits may be increased with the support of future research (use of low-ambient pressure chambers to induce cavitation) and also with more data analysed from prototypes. It is however advisable to include measures to prevent cavitation when the referred limits are to be surpassed and also for unit discharges over 30 $m^3/s/m$ (Boes, 2012).

The best measure to counter cavitation is providing aeration. Research that has been carried out at VAW of ETH Zurich focussed on the effect of step aerators and deflectors (Pfister *et al*., 2006b; Schiess-Zamora *et al*., 2008). The step aerator is an air-supply conduit that exits at the first vertical face (first step after the smooth upstream part) (Fig. 4.12). The negative pressures on the upper part of the vertical face contribute to drag the air, which is then mixed within the flow. The use of a step aerator enables to increase unit discharges up to 40 $m^3/s/m$ (for usual gravity dam and step heights of s=1.2 m). Larger unit discharges ($q>40$ $m^3/s/m$) require the introduction of a second aerator, that should be located upstream of the inception point.

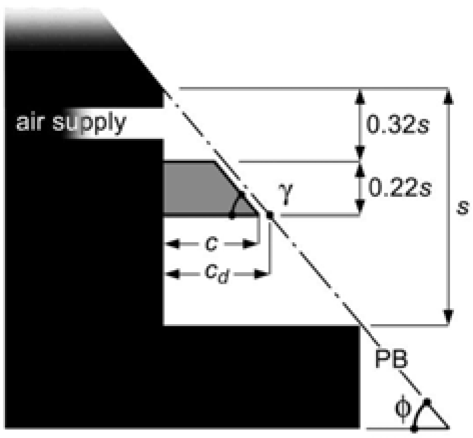

Figure 4.12.- Step aerator for providing aeration. (Schiess-Zamora *et al*. 2008)

A more effective and brisk measure for providing aeration is the use of deflectors. The deflector is located at the beginning the stepped chute, and it produces a jump over the initial steps creating a large air-cavity under the jet (Pfister *et al.*, 2006b). The air is entrained into the flow when the jet impacts the chute downstream. The deflector has to be complemented with air-supply conduits (Fig. 4.13). The design of these devices should take into account that high spray would be produced around the impact point. A higher step designed with the same aim of creating an air-cavity under the flow, has been also employed to enhance aeration as an alternative to deflectors.

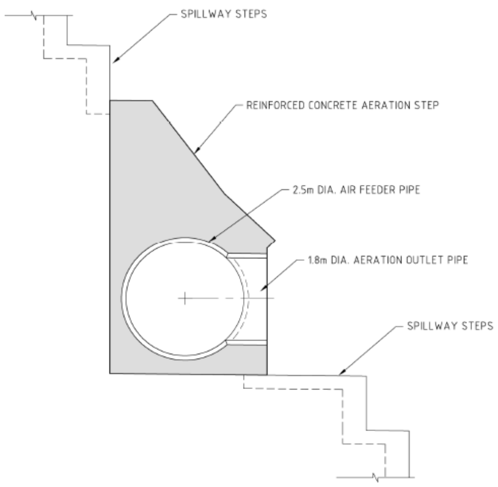

Figure 4.13.- Enlarged Cotter Dam (Australia). Deflector for providing aeration. (Willey *et al.*, 2010)

Furthermore, aeration could be also improved by using splitters and protrusions. The work carried out by Wright (2010) shows that triangular protrusions located on the upstream part of the spillway help to shorten the length of inception and enhances aeration. Thus, for an upper velocity limit of 20 m/s at the inception point a spillway fitted with triangular protrusions will be effective up to 40 $m^3/s/m$.

4.4.4. Sidewall design and behaviour without sidewalls

The sidewall design should take into account the bulking effect of the aerated flow. The flow depth of the two-phase fluid is larger than that of the equivalent discharge over a smooth chute. Hence, in assessing the required wall height a safety factor over and above the calculated height is usually applied depending on the nature of the surrounding topography. Special care is needed if the spillway overflow could affect an adjacent earth embankment dam.

For stepped spillways with parallel sidewalls, the required sidewall height perpendicular to the pseudo-bottom (h_t) could be calculated applying a freeboard coefficient (η) to the characteristic flow depth:

$$h_t = \eta \cdot h_{90}$$

Boes and Hager (2003b) propose the use of η =1.2 for gravity dams and η=1.5 for embankment dams.

The calculation of the sidewall height in converging spillways requires a specific approach (Hunt *et al.*, 2012), since there is a flow concentration near the wall. Those types of converging spillways can be found on narrow valleys and, most commonly, on embankment dams where the spillway overflow protection may also spill over the dam abutments constituting a converging stepped channel.

The performance without sidewalls is a novel approach. Is characterized by the lateral expansion of flow, which results in a lower unit discharges at the dam toe. It could be a cost-effective alternative in cases with low unit discharge and good foundation. The studies that were carried out at UPC (Barcelona) show that unit discharge reduction with such arrangement is between 50% and 70% (Estrella *et al.*, 2012).

4.4.5. Terminal structure

One of the main advantages of the stepped spillways is the energy dissipation that could be achieved along the chute. This renders the design of smaller terminal structures compared to that of a similar smooth chute possible, which results in cost and construction time savings.

In stepped spillways on RCC gravity dams the use of a stilling basin as a terminal structure is common. The sizing of the stilling basin is based on the determination of the conjugate depth. For smooth spillways the velocity and the depth at the chute toe are used for calculating the conjugate depth. For stepped spillways the equivalent clear water depth and equivalent terminal velocity should be determined at the chute toe upon the base of the residual energy (Boes and Hager, 2003b).

Precaution has to be taken when using standard stilling basins, as these were developed for smooth chutes. Particular care has to be taken for those which used baffle blocks and sills to reduce the pool length. Work by Frizell *et al*. (2009) regarding the performance of USBR-Type III stilling basins (Peterka, 1978) shows that the change in the velocity distribution in a stepped chute reduce the effect of the baffles, with the result being that similar or even larger pool length would be required.

In the case of overflowing spillways for embankment dams constructed with RCC, both the unit discharges (average of 3 $m^3/s/m$) and the dam heights (average of 15 m) are smaller than those of the RCC gravity dams, hence the required terminal structures are simpler. In many cases a downstream apron is sufficient to prevent erosion and stilling basins are only used for the larger protections.

4.5. NOTATION

The following symbols are used in this chapter:

A	Cross-sectional area of flow
C_e	Equilibrium air concentration for uniform flow
$\overline{C}_i$	Averaged air concentration at the inception point
D_h	Hydraulic diameter ($D_H = 4A / P$)
D_{hw}	Hydraulic diameter computed using the equivalent clear water depth
E	Parameter used to determine the residual energy when uniform flow is attained
F_*	Roughness Froude number ($F_* = q / (g \cdot \sin\Phi \cdot K_s^3)^{1/2}$)
F_{*1}	Roughness Froude number ($F_{*\Phi} = q / (g \cdot \cos\Phi \cdot s^3)^{1/2}$)
$F_{*\Phi}$	Roughness Froude number corrected for the slope effect ($F_{*\Phi} = q / (g \cdot \cos\Phi \cdot K_s^3)^{1/2}$)

f	Darcy friction factor for non-aerated flows
f_b	Friction factor of bottom roughness
f_e	Equivalent Darcy friction factor for the air-water mixture
g	Gravitational acceleration
H_{chute}	Chute height
H_{max}	Maximum head –potential energy of water at the reservoir–
H_{res}	Residual energy
h_c	Critical depth
h_i	Flow depth at the inception point
h_t	Sidewall height perpendicular to pseudo-bottom
h_w	Equivalent clear-water depth
h_{wu}	Equivalent clear-water depth on uniform flow
h_{90}	Characteristic mixture flow depth
h_{90u}	Characteristic mixture flow depth on uniform flow
K_s	Bottom roughness ($K_s = s \cdot \cos \Phi$)
L_i	Streamwise length from spillway crest to inception point
N_s	Number of steps
P	Wetted perimeter
q	Unit discharge
s	Step height
x	Streamwise distance along the spillway from the crest
α	Kinetic energy corrector coefficient
η	Freeboard coefficient
Φ	Chute slope

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5. LABYRINTH SPILLWAYS

5.1 INTRODUCTION

Labyrinth spillways can convey higher discharges at a given pool elevation than a linear overflow structure of comparable width. The spillway layout can be adapted to site-specific requirements to significantly increase spillway capacity, improve flood control, and offer additional reservoir storage. These '3-D' spillways also provide passive-control operation, reliability, energy dissipation, flow aeration, and desirable aesthetics. The hydraulic optimization of these spillways is commonly moderated by project economics and constructability. They are particularly well suited for applications where it is advantageous to minimize the width of the spillway or overall size of the spillway footprint.

An example of a labyrinth spillway is shown in Figure 5.1. The spillway has a discharge capacity of 15 000 m^3/s and features aeration vents and bridge piers.

Figure 5.1 - View of the Labyrinth Spillway of Maguga Dam in Swaziland

(Photo courtesy of Aurecon)

One of the first studies of labyrinth weirs may perhaps be credited to Gentilini (1941). The precursor to the first published design method by Hay and Taylor in 1970 was a systematic study performed by Kozák and Sváb (1961). Numerous physical model studies and ensuing design methods have been published since that time. Labyrinth weir design publications of note have been produced at the U.S Bureau of Reclamation (Denver, Colorado, USA), the Laboratório Nacional de Engenharia Civil (Lisbon, Portugal), and the Utah Water Research Laboratory (Logan, Utah, USA). Falvey (2003) summarized many of the contributions to labyrinth weir hydraulics up to the turn of the century. Crookston (2010) expanded the labyrinth weir research knowledge base with a more comprehensive design method and by investigating the influences of crest shape, nappe behaviour, configuration (in-channel vs. reservoir application), arched weir geometries, and size-scale effects on labyrinth weir hydraulics.

5.2 GENERAL DESCRIPTION

Labyrinth weirs have been placed in assorted channel and reservoir applications and have featured a wide variety of weir geometries and crest shapes. Labyrinth spillways commonly feature multiple trapezoidal-shaped cycles placed in a linear fashion at a single crest elevation, as shown in Figure 5.3. However, arced spillway configurations have been constructed in reservoir applications, such as Avon Dam (Australia, Darvas 1971), Kizilcapinar Dam (Turkey, Yildiz and Uzecek 1996), Maguga (Swaziland, Van Wyk et al 2006), María Cristina Dam (Spain, Page et al. 2007) and Weatherford Dam (USA, Tullis 1992). Furthermore, the U.S. Army Corp of Engineers is designing a large arced labyrinth spillway at Isabella Dam in California, USA. In general, labyrinth spillways are symmetrical with some type of chute or energy dissipation basin located downstream. In addition, some labyrinth weir spillways have incorporated multiple crest elevations and various appurtenant structures such as foot and vehicle bridges, piers, base-flow notches, cool-water releases, and low-level outlets.

Crookston (2010) provides a comprehensive overview of the hydraulic design of labyrinth spillways. It presents a thorough history of labyrinth weir research and design, including prior design methods, numerous case studies, and a comprehensive bibliography of labyrinth weir research contributions. The results of Crookston (2010) have been summarized and published in Crookston and Tullis (2013a,b) and Crookston and Tullis (2012a,b,c). These technical articles comprise the principal references for the following discussion regarding the hydraulic design of labyrinth spillways and shall be generally referred to as the Crookston and Tullis Design Method.

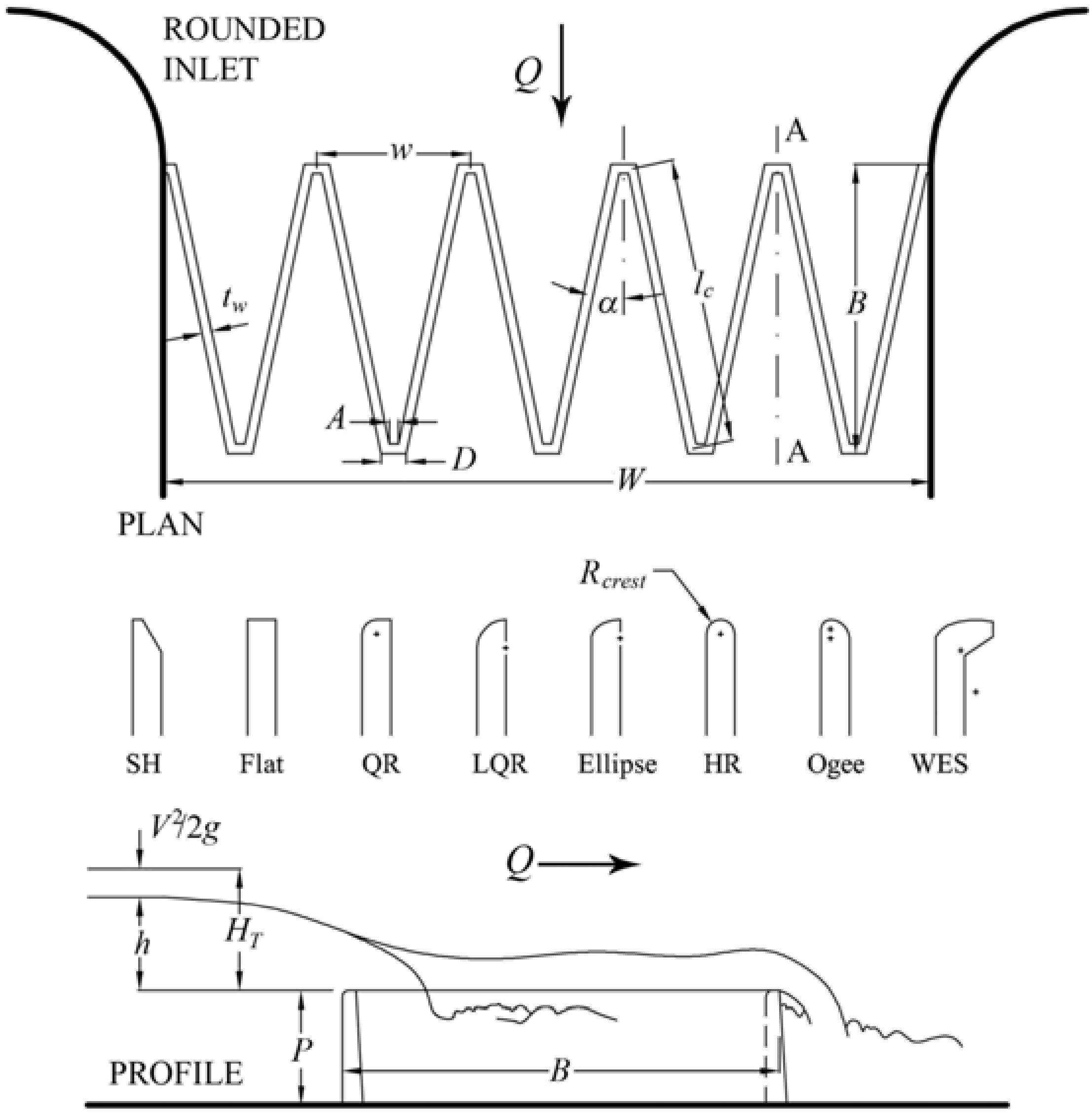

Figure 5.2 - Definition sketch for Labyrinth Spillway geometry (Crookston, 2010)

5.3 DISCHARGE CAPACITY

Total discharge over a labyrinth spillway is a function of the flow conditions and weir geometry, expressed as:

$$Q = f\left(g, \nu, H_T, H_d, L_c, \alpha, A_c, l_c, P, B, w, t_w, \text{crest, approach, nappe}\right) \tag{5.1}$$

where Q = total discharge; g = acceleration constant of gravity; ν = kinematic viscosity of water; H_T = total upstream head measured relative to the weir crest, where $H_T = V^2/2g + h$; V = average cross-sectional velocity of flow; h = piezometric head upstream of the weir (relative to the weir crest elevation). H_d = downstream tailwater total head; L_c = total centreline length of the weir crest, where $L_c = 2N(A_c+l_c)$; N = number of labyrinth cycles; A_c = centreline length of the apex; l_c = the centreline length of the sidewall; α = sidewall angle; P = weir height; B = cycle depth (longitudinal length); w = cycle width; t_w = wall thickness at the crest. (Crookston and Tullis 2013a) document the influence of crest shape on discharge efficiency. The upstream topography and approaching flow field, as well as the abutment geometries and location relative to the weir placement (projecting, flush, etc.), also influence discharge efficiency (Crookston and Tullis 2012b). The influence of weir height and cycle width on labyrinth hydraulics are also discussed in Crookston et al. (2012) with recent physical modelling based research by Seamons (2014). The collision of adjacent nappes (near the upstream apex) influences labyrinth weir hydraulics; the occurrence and effects of local submergence wakes, and standing waves are discussed in Crookston and Tullis (2012c). The influence of apex width is presented in Seamons (2014). The nappe aeration condition (clinging, aerated, partially aerated, drowned) and behaviour (nappe vibration, nappe instability) should be considered in labyrinth weir design (Crookston and Tullis 2013b).

The head-discharge relationship for labyrinth weirs has been characterized using Equation (5.2), a common form of the weir equation (Henderson 1966):

$$Q = \frac{2}{3} C_{d(\alpha\circ)} L_c \sqrt{2g} H_T^{\;3/2} \tag{5.2}$$

where $C_{d(\alpha°)}$ = experimentally determined dimensionless discharge coefficient that is sidewall-angle-specific, and accounts for parameter influences noted in Equation (5.1) that are not expressly accounted for in Equation (5.2). Figures 5.4 and 5.5 present Crookston (2010) graphical $C_{d(\alpha°)}$ data as a function of α and dimensionless headwater ratio (H_T/P) for quarter-round and half-round crest shapes, respectively. Crookston and Tullis (2013a) provide empirical curve-fit equations for quarter- and half-round $C_{d(\alpha°)}$ data. Linear interpolation may be appropriate for estimating $C_{d(\alpha°)}$ for other values of α not tested by Crookston (2010).

The $C_{d(\alpha°)}$ curve-fit equations presented by Crookston and Tullis (2013a) are based on experimental data limited $H_T/P \leq 1.0$. The specific form of the curve-fit equations, however, was selected to facilitate $C_{d(\alpha°)}$ extrapolation beyond H_T/P=1.0 (see Figure 5.5). The α=15° equation was validated for $H_T/P \leq 2.1$ by Crookston et al. (2012) through physical and numerical modelling, increasing the design flexibility of the Crookston and Tullis Method. In contrast, the polynomial curve fits used by Tullis *et al*. (1995) (quarter-round crest shape) are limited to the range of experimental data from which they were derived (H_T/P<0.9) and cannot be used for extrapolation (i.e., $C_{d(\alpha°)}$ values become negative with increasing H_T/P due to the nature of the polynomial functions, see Figure 5.5).

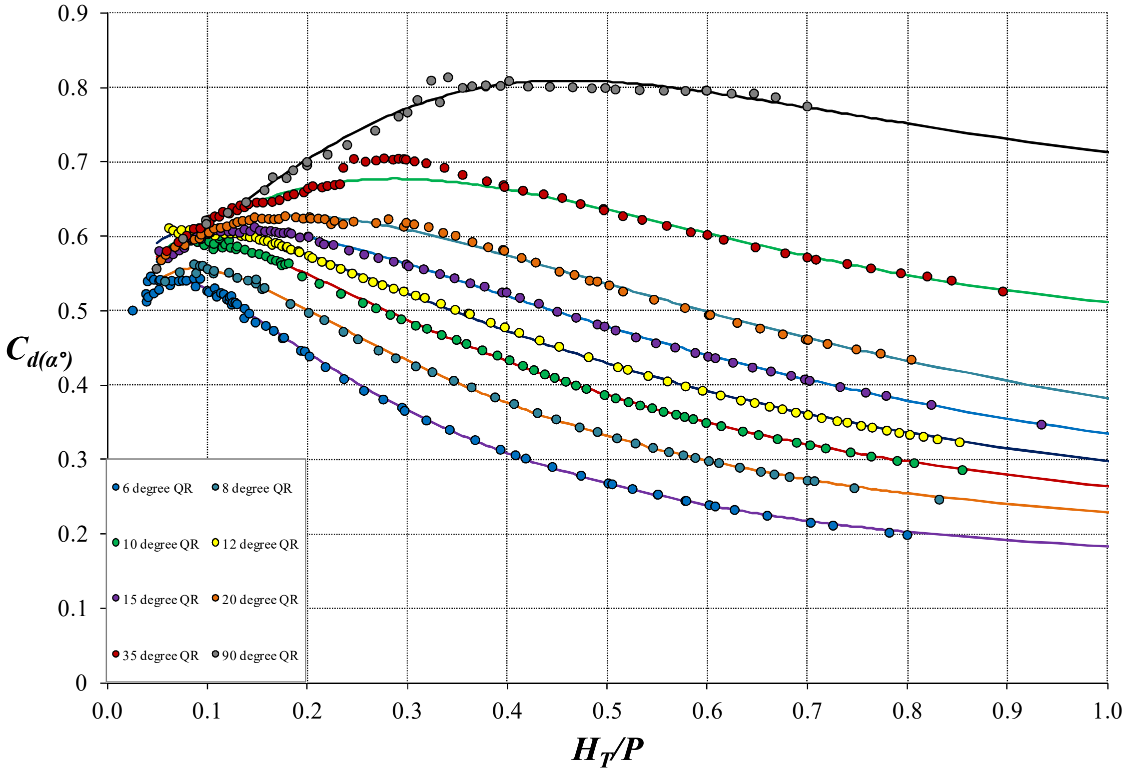

Figure 5.3 - $C_{d(\alpha°)}$ *versus* H_T/P for quarter-round trapezoidal labyrinth spillways

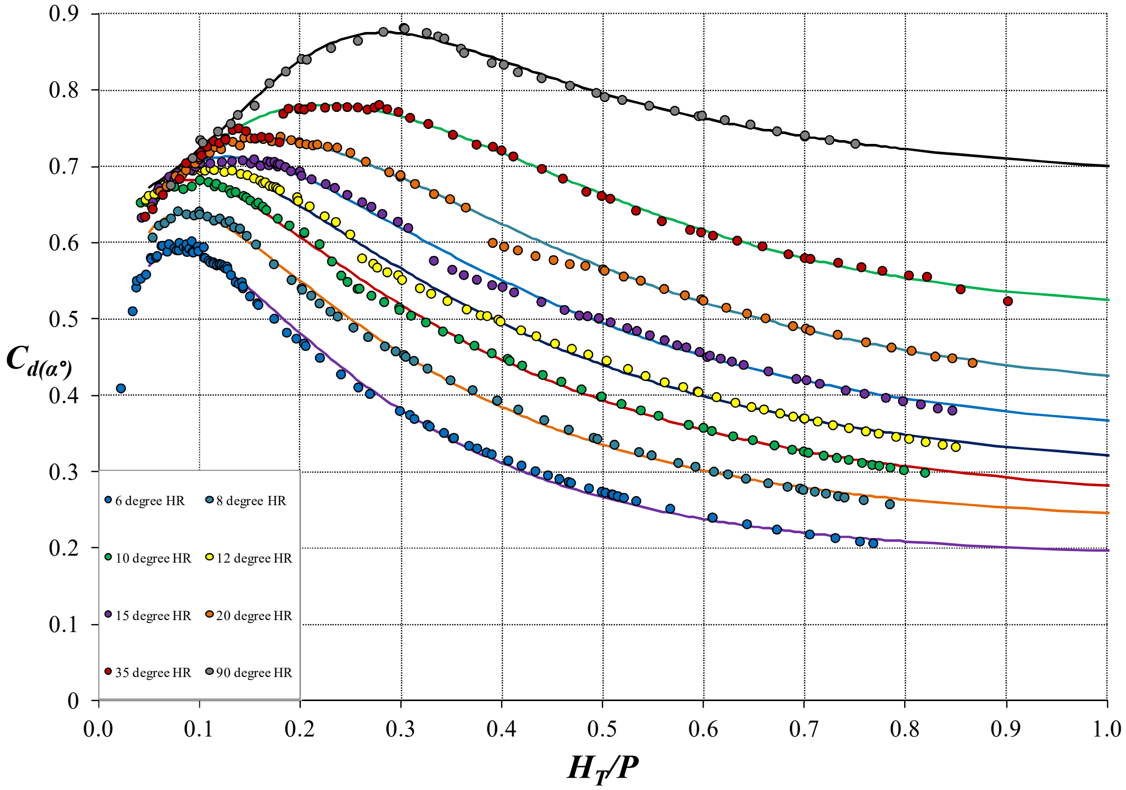

Figure 5.4 - $C_{d(\alpha°)}$ *versus* H_T/P for half-round trapezoidal labyrinth spillways

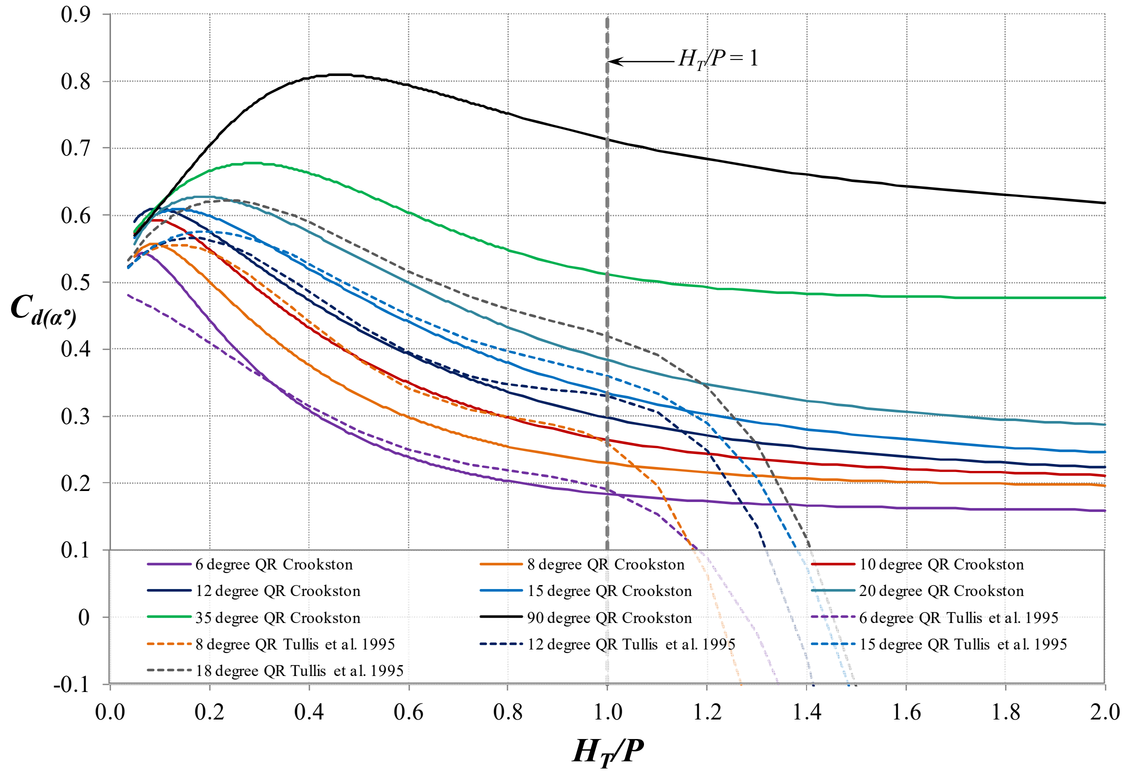

Figure 5.5 - Comparison of Crookston and Tullis (2013a) and Tullis *et al*. (1995) quarter-round $C_{d(\alpha°)}$ curve-fit equations

The difference in hydraulic efficiency between a quarter- and half-round crest types are discussed by Crookston and Tullis (2013a). The half-round crest is hydraulically more efficient than the quarter-round crest at low H_T/P values due to the ability of the half-round crest nappe to remain attached to the downstream crest profile. At higher H_T/P values, the nappe will eventually separate from the downstream crest profile and be hydraulically similar to that of the quarter-round crest. Other labyrinth weir crest shapes that have been implemented in practice are presented in Figure 5.2.

Rounded crest shapes have been efficiently constructed via formwork and hand finishing or fabricated forms. Cost-effectiveness generally increases with the number of cycles. An excellent example of a cost-effective labyrinth weir that utilized fabricated forms is Lake Brazos Dam (Texas, USA) where the designers specified rounded apexes and an ogee-type crest.

5.4 NAPPE BEHAVIOUR AND ARTIFICIAL AERATION

In addition to discharge, labyrinth spillway nappe behaviour should also be considered in design. Figure 5.6 illustrates the various nappe aeration behaviours conditions, and nappe instability (also termed flow surging) for labyrinth spillways with quarter-round crest shape; similar information for half-round crests is also presented in Crookston and Tullis (2013b). Not included in Figure 5.6 is nappe vibration, which may also occur (Crookston et al. 2014, Anderson 2014). Four nappe aeration conditions exist for labyrinth weirs: clinging, aerated, partially aerated, and drowned. The influence of artificial aeration, vent pipes, and nappe breakers or flow splitters on discharge and nappe behaviour is also discussed, including information regarding device placement and general nappe breaker shape.

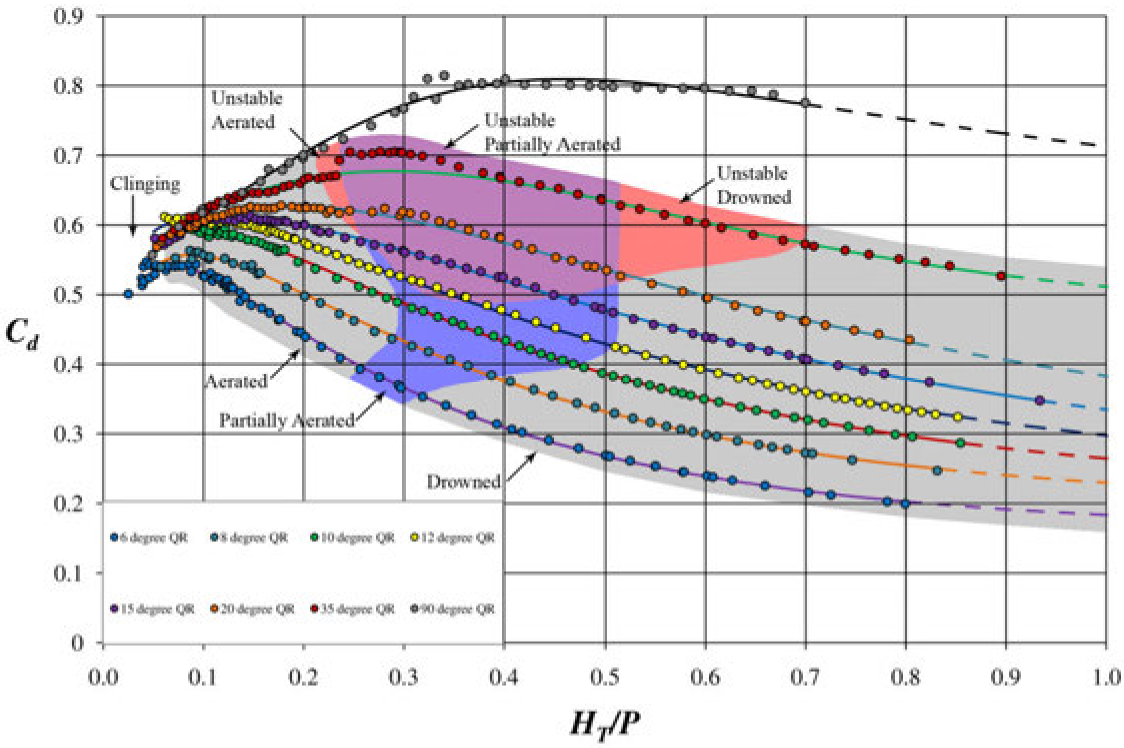

Figure 5.6 - Nappe behaviour for labyrinth spillways with a quarter-round crest

For some hydraulic conditions, the size of the aeration cavity behind the nappe can fluctuate temporally, resulting in nappe instability. It is a low-frequency nappe trajectory fluctuation phenomena that occurs at moderate H_T/P ratios, produces noise, and is also a function of α and crest shape. Flow splitters may decrease the magnitude of nappe instability but were observed to be insufficient to prevent occurrence in the laboratory (Crookston 2010). Additional information regarding nappe instability is discussed in Crookston and Tullis (2013b).

Separate from nappe instability, nappe vibration (see Figure 5.7) can occur at low-head flow conditions (observed in some prototypes with flow depths above the crest of approximately 15 cm or less) and can develop with linear and non-linear weirs (Crookston et al. 2014). Nappe vibration can result in intense acoustic pressure waves and noise (Casperson, 1995) and has been aptly described as sounding similar to a helicopter. In general, nappe vibration is visually observed as closely spaced horizontal bands that initiate where the nappe departs from the crest. Very thin nappes may undulate or flutter. Also, vibration can be amplified by wind.

The most widely accepted theory for this phenomenon attributes vibrations to shear forces at the interface between the falling water sheet and the surrounding air, known as the Helmholtz mechanism (Helmholtz, 1868). Instability due to said mechanism can be amplified by an enclosed air pocket behind the nappe (Naudascher and Rockwell, 2005); however, a fully vented nappe displaying intense vibrations has been observed by Falvey (1987), in prototype structures, and in large models at the Utah Water Research Laboratory (Anderson 2014).

Figure 5.7 - Nappe vibration (photo courtesy of Schnabel Engineering, USA)

The specific influence of crest shape, weir length, fall height, and flow characteristics on nappe vibration for labyrinth spillways is still unclear. Adding roughness elements to the crest surface (Metropolitan Water, Sewerage and Drainage Board 1980) is an example of one successful method reported in the literature for mitigating nappe vibration Vibrations have also been mitigated using numerous nappe breakers at some minimal interval determined by experimentation.

5.5 CYCLE EFFICIENCY

Per Equation (5.1), Q is proportional to $C_{d(\alpha°)}$, which decreases with decreasing α (see Figures 5.3 and 5.4), and L_c, which increases with decreasing α (for a given cycle width). Cycle efficiency ($\varepsilon' = C_{d(\alpha°)}L_{c\text{-}cycle}/w$) is representative of the discharge per cycle (at a given H_T/P value) for a given labyrinth weir geometry and can be used to illustrate the net effect of these two opposite-trending parameters on discharge efficiency. ε' data for quarter- and half-round crest shapes and various values of α and H_T/P are shown in Figure 5.8 (Crookston and Tullis 2013a). The ε' data show that the increase in weir length more than compensates for the declining $C_{d(\alpha°)}$ values as α decreases. This means that the largest discharge per cycle occurs at the smaller α values. The hydraulic benefit of increased ε' of smaller α angles decreases with increasing H_T/P. ε' only evaluates the discharge efficiency per cycle and should be yoked with a cost analysis to select the optimal labyrinth spillway geometry as a less efficient spillway that meets design requirements may be a more cost-effective option. ε' comparisons between structures are particularly useful when the structures share common H_T/P ratio. The actual discharge per cycle width requires multiplying ε' by $H_T^{3/2}$.

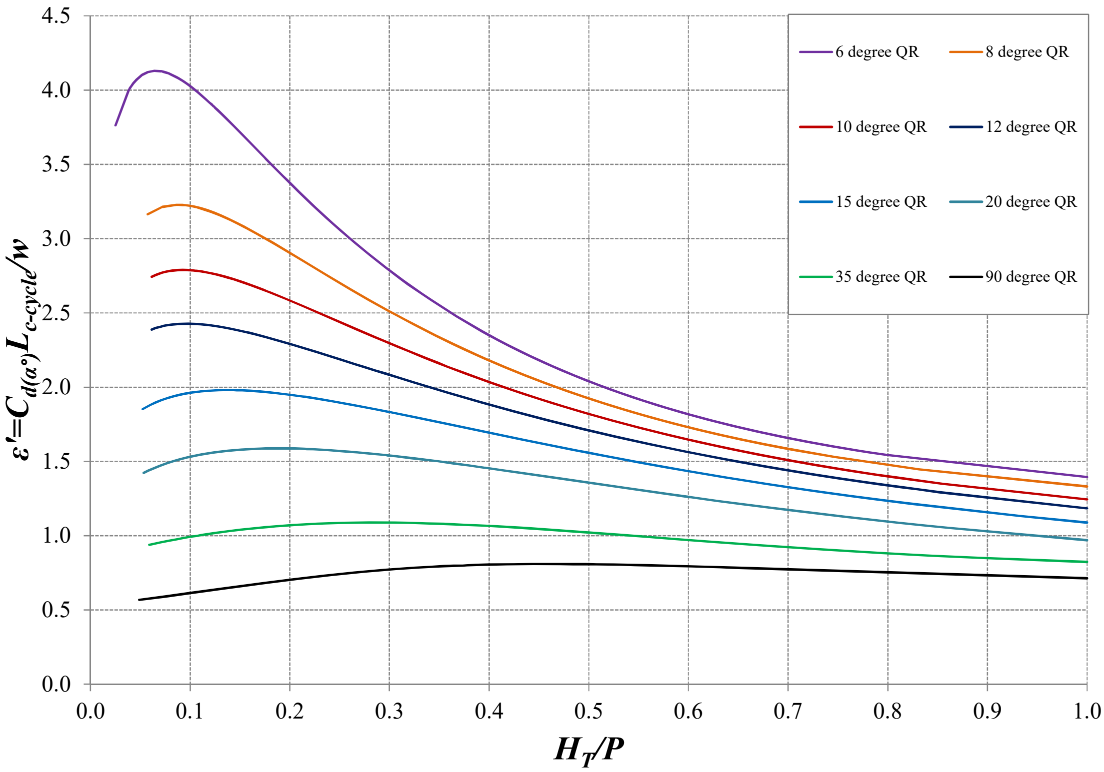

Figure 5.8 - Cycle efficiency for labyrinth spillways with a quarter-round crest

5.6 TAILWATER SUBMERGENCE

Weir tailwater submergence occurs when the elevation of the downstream water surface exceeds the crest elevation. A tailwater submergence condition that does not increase the headwater elevation is referred to as modular submergence. Submergence levels higher than the modular submergence limit will increase the upstream water elevation for a give discharge. Local submergence differs from tailwater submergence in that it is independent of the downstream tailwater condition. A labyrinth weir dimensionless submerged head-discharge relationship is presented in Figure 5.9 (Tullis et al. 2007). In Figure 5.9, H^* and H_d are respectively the total upstream and downstream heads under submerged labyrinth weir flow conditions. H_T is the free-flow upstream total head for the same discharge (Q represents the independent variable in this analysis). The modular submergence range corresponds to H^*/H_T=1.0. The data in Figure 5.9 can be used to determine H^* knowing H_T and H_d. Similar head-discharge data for submerged piano key weirs is presented by Dabling and Tullis (2012).

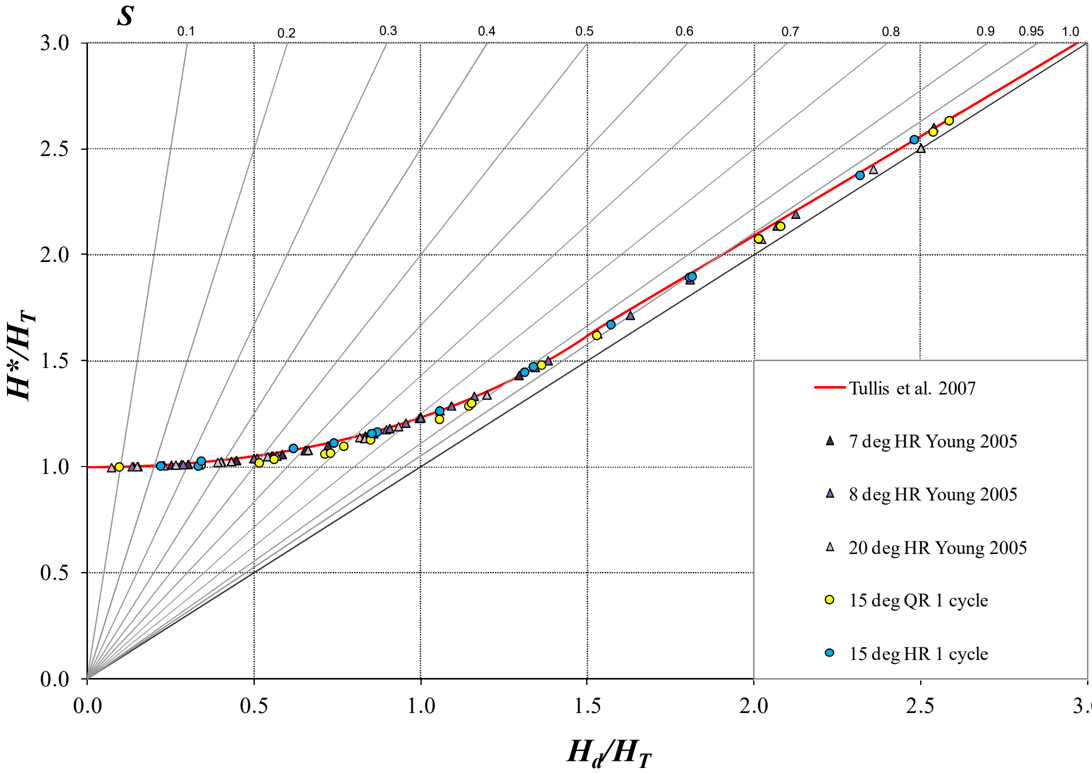

Figure 5.9 - Tailwater submergence dimensionless head relationship for labyrinth spillways (adapted from Tullis et al. 2007)

5.7 CYCLE WIDTH RATIO

A minimum cycle width ratio (*w/P*) greater than 2 has been suggested in previously proposed design methods; this recommendation has been based upon supporting data and discharge efficiency. However, increasing *P* may improve the approach flow conditions and therefore improve discharge capacity. In addition, reductions in *w* and hence, *w/P*, typically result in reduced construction costs for a given design flow and therefore increase spillway effectiveness, despite reduced flow efficiency (Crookston et al. 2013, Paxson et al. 2013). Furthermore, the Tullis et al. (1995) Design Method included $C_{d(\alpha°)}$ data for *w/P* ratios less than 2 despite suggesting ratios from 3 to 4. In light of this, research performed by Crookston (2010) explored the influence of *w* and *P* on discharge, as identical *w/P* ratios may exist for different value combinations of *w* and *P*. General guidance regarding the influence of *w* and *P* on discharge of labyrinth spillways is presented in Crookston et al. (2013) and Seamons (2014).

5.8 WEIR PLACEMENT AND ABUTMENT EFFECTS ON HYDRAULIC EFFICIENCY

Research performed at the USBR for labyrinth prototypes provided some insight regarding the effect of labyrinth weir placement in reservoir applications (Houston 1983). Additional applied research regarding the effects of spillway abutments for labyrinth spillways with a half-round crest is presented in Crookston & Tullis (2012b). The various in-channel and reservoir application specific geometric configurations tested are presented in Figure 5.10.

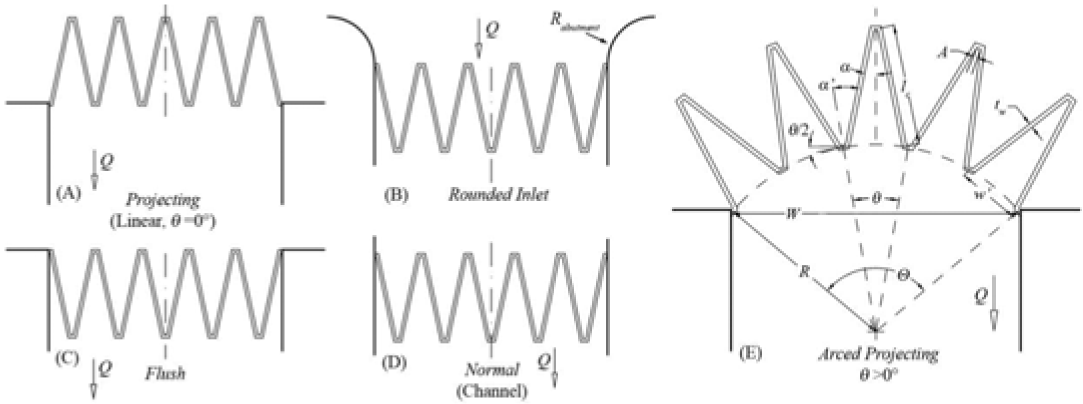

Figure 5.10 - Crookston and Tullis (2012b) reservoir geometries: (A) *Projecting*, (B) *Rounded Inlet*, (C) *Flush*, (D) *Normal*, and (E) *Arched*.

A comparison of $C_{(\alpha°)}$ for four reservoir geometries [Figure 5.10(A), (B), (C), and (E)] relative to an In-Channel [Figure 5.10(D)] application (α=12°) is presented in Figure 5.11 (Crookston and Tullis 2012b). Based on the specific geometries tested, the *Flush* configuration produced the largest hydraulic efficiency reduction, relative to the in-channel application, due to flow separation at the abutments. Adding rounded abutments [Figure 5.10(B)] improved the hydraulic efficiency; however, both the *Rounded Inlet* and *Projecting* configurations were approximately 3% to 7% less efficient than the in-channel configuration. Additional information regarding the hydraulic performance of these configurations is presented in Crookston and Tullis (2012b).

A projecting labyrinth spillway with a cycle angle θ = 10° [see Figure 5.10(E)] exceeded the discharge efficiency of the *In-channel* configuration by approximately 5% to 11% and was the most efficient geometry tested. An *Arched* labyrinth weir configuration increases the discharge capacity for reservoir applications by orienting the cycles toward the approach flow. The *Arched* configuration also facilitates a narrower downstream channel or chute width, potentially reducing construction costs. Crookston and Tullis (2012a) present standardized nomenclature for arched labyrinth spillways and discuss geometrically comparable layout designs. Several *Arched* configurations were tested and found to be approximately 5 to 30% more efficient than in-channel linear configurations (H_T/P dependent). The hydraulic advantage of the *Arched* labyrinth weir diminishes with increasing H_T/P (increased influence of local submergence increase).

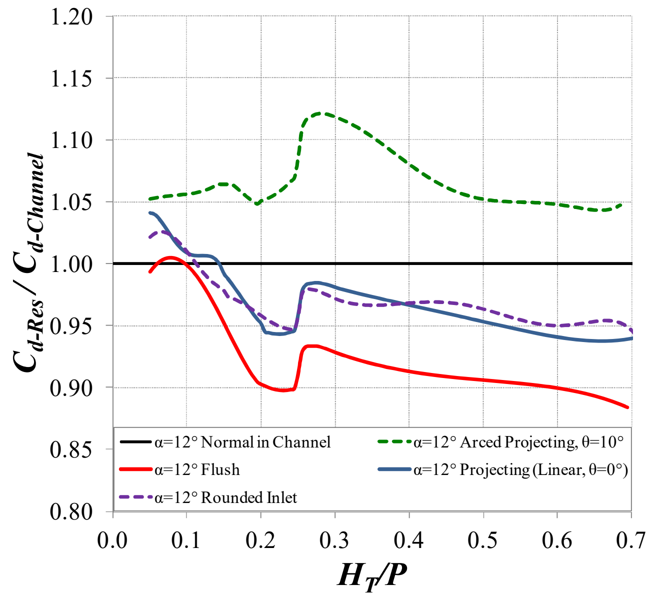

Figure 5.11 - Discharge efficiency for reservoir-specific labyrinth spillways, relative to in-channel application (α=12°)

5.9 ENERGY DISSIPATION AND RESIDUAL ENERGY

Labyrinth spillways are highly effective at energy dissipation. Lopes et al. (2006, 2008) present design information regarding relative residual energy (H_1) downstream of labyrinth weirs. Relative residual energy increases as H_T/P increases (i.e., less relative energy dissipation); labyrinth weirs produce a smaller residual energy than vertical drops (regardless of α). Lopes et al (2006, 2008) are recommended for estimating residual energy downstream of labyrinth weirs.

5.10 DEBRIS

Debris is often transported to hydraulic structures during flood events (Pfister et al. 2013). A survey regarding the debris handling of 75 labyrinth spillways in the USA and Portugal was recently completed (Crookston et al. 2015). The results indicate that in general, the debris handling performance of labyrinth spillways is acceptable with few reports regarding debris maintenance. For example, storm events in forested catchments may result in flows laden with woody debris. Depending upon the spillway hydraulics and site conditions, this type of debris may pass over the labyrinth weir during floods and require little maintenance. Conversely, conditions may result in large accumulations of woody debris that raise safety concerns. Therefore, the accumulation of debris on labyrinth weirs should not be overlooked for situations where there is the potential for debris to reduce spillway capacity. Research on driftwood for PKW spillways provides unique insights that are generally applicable to labyrinth weirs (Pfister et al 2013).

5.11 HYDRAULIC DESIGN AND ANALYSIS

The recommended procedure for designing a labyrinth weir is presented as Table 5.1 (Crookston and Tullis 2013a). This format was first introduced by Tullis et al. (1995), continued by Falvey (2003), and has been refined and expanded by Crookston and Tullis. The top section of the design table features user-defined hydraulic conditions or requirements for the labyrinth weir. The design flow rate (Q_{design}) typically represents a flood discharge (e.g., 100-yr storm, PMF, etc.) estimated from a hydrologic analysis. In practice, Q_{design} is commonly estimated using computer programs such as HEC-HMS, its predecessor HEC-1, or other suitable methodologies. H represents the maximum allowable reservoir pool elevation. H_T may be assumed to be the difference in the reservoir pool elevation and the crest elevation, H_{crest} (negligible velocity head). However, it may be appropriate to consider the upstream flow conditions and any minor losses when estimating H_T. H_d is the total downstream head measured relative to the spillway crest and may be determined by a backwater curve. User-defined labyrinth weir geometric parameters are presented in the second section; the use a nappe aeration device can be specified if desired. The third section computes the labyrinth weir hydraulic performance data and determines additional weir geometric parameters. Although the Crookston and Tullis Design Method only provides $C_{d(\alpha°)}$ for quarter- and half-round crest shapes, the method easily accommodates experimentally determined $C_{d(\alpha°)}$ for site-specific conditions or other crest shapes. $C_{d(90°)}$ and the corresponding linear weir crest length (same crest shape) required to match the Q_{design} and H_T requirements are also reported for comparison. The last section of the design method includes the submerged head-discharge relationships developed by Tullis et al. (2007). Table 5.1 can be expanded to include a complete head-discharge relationship for the specified labyrinth weir geometry. This design method can also be used to estimate the head-discharge relationship for existing labyrinth spillways. Such a procedure, which also adapts easily to a spreadsheet program format, is outlined in Crookston and Tullis (2013a); the labyrinth weir geometric parameters are specified rather than calculated. The effects of tailwater submergence may be determined by solving for Q or H^* as discussed by Tullis et al. (2007).

Table 5.1 - Labyrinth weir geometry calculation template

Parameter	Symbol	Value	Units	Notes
		Hydraulic Conditions – Input Data		
Design Flow	Q_{design}	1,500.00	(m^3/s)	g = 9.81 m/s^2
Design Flow Water Surface Elevation	H	1,680.00	(m)	
Approach Channel Elevation	H_{apron}	1,674.00	(m)	
Crest Elevation	H_{crest}	1,678.00	(m)	
Unsubmerged Total Upstream Head	H_T	2.00	(m)	Piezometric Head + Velocity Head - Losses
Downstream Total Head	H_d	0.50	(m)	
		Labyrinth Weir Geometry – Input Data		
Angle of Side Legs	α	12	(°)	α ~ 6° - 35°
Number of Cycles	N	9	-	whole or half cycles
Crest Height	P	4.00	(m)	P ~1.0H_T
Thickness of Weir Wall at the Crest	t_w	0.50	(m)	t_w ~ P/8
Inside Apex Width	A	0.50	(m)	A ~ t_w
Crest Shape	*Crest Shape*	Quarter	-	Quarter- or Half-Round
Aeration Device (Nappe Breakers, Vents)	-	Breakers	-	Breakers, Vents, or None
		Calculated Data		
Headwater Ratio	H_T/P	0.50	-	Data for H_T/P<1.0, extrapolation for H_T/P≤2.0
Labyrinth Weir Discharge Crest Coefficient	$C_{d(\alpha°)}$	0.429	-	$C_{d(\alpha°)} = f(H_T/P, \alpha, \mathit{Crest\ Shape})$
Total Centerline Length of Weir	L_c	418.28	(m)	$L_c = 3/2Q_{design}/[(C_{d(\alpha°)}H_T^{3/2})(2g)^{1/2}]$
Centerline Length of Sidewall	l_c	2.33	(m)	$l_c = (B-t_w)/\cos(\alpha)$
Outside Apex Width	D	1.30	(m)	$D = A+2t_w\tan(45-\alpha/2)$
Cycle Width	w	11.10	(m)	$w= 2l_c\sin(\alpha)+A+D$
Width of Labyrinth (Normal to Flow)	W	99.87	(m)	$W = Nw$
Length of Apron (Parallel to Flow)	B	22.35	(m)	$B = [L_c/(2N)-(A+D)/2]\cos(\alpha)+t_w$
Magnification Ratio	M	4.19	-	$M = L_c/(wN)$
Cycle Width Ratio	w/P	2.77	-	Normally 2 ≤ w/P ≤ 4
Relative Thickness Ratio	P/t_w	8.0	-	~8
Apex Ratio	A/w	0.05	-	<0.08
Cycle Efficiency	ε'	1.80	-	$\varepsilon' = C_{d(\alpha°)}M$
Efficacy	ε	2.23	-	$\varepsilon = C_{d(\alpha°)}M/C_{d(90°)}$
# of Nappe Breakers or Vents	-	9	-	Breaker on ds Apex, 1 Vent per Sidewall
Linear Weir Discharge Coefficient	$C_{d(90°)}$	0.808	-	$C_{d(90°)} = f(H_T/P, \alpha, \mathit{Crest\ Shape})$
Length of Linear Weir for same Flow	$L_{c(90°)}$	222.33	(m)	$L_{c(90°)} = 3/2Q_{design}/[(C_{d(90°)}H_T^{3/2})(2g)^{1/2}]$
		Submergence (Tullis et al. 2007)		
Downstream/Upstream Ratio of Unsubmerged Head	H_d/H_T	0.25	(m)	
Submerged Head Discharge Ratio	H^*/H_T	1.013	-	Piecewise function Tullis et al. (2007)
Submerged Upstream Total Head	H^*	2.025	(m)	
Submergence Level	S	0.247	-	$S = H_d/H^*$
Submerged Weir Discharge Coefficient	$C_{d\text{-}sub}$	0.421	-	$C_{d(\alpha°)}(H_T/H^*)^{3/2}$

†Design limited to extent of experimental data; designs that exceed these limits may warrant a physical model study

The design method and support data are limited to the geometries (Crookston 2010) and hydraulic conditions tested (e.g., 0.05 ≤ H_T/P ≤ 0.9). These results may be conservatively applied (with sound engineering judgment) to other labyrinth weir geometries and flow conditions (design verification with a hydraulic model study is recommended). Linear interpolation is recommended to determine $C_{d(\alpha°)}$ for α values other than those presented. Based on the available support data from the current study, the design method (Table 5.1) may be used for H_T/P ≤ 2.0, as the experimental results are well behaved and the α=15° was tested up to H_T/P = 2.1; the resulting $C_{d(\alpha°)}$ data agreed well with the curve-fit equation proposed by Crookston and Tullis (2013a).

5.12 PHYSICAL AND NUMERICAL (CFD) MODELLING

Further discussions on the use of Physical and CFD modelling in spillway design are provided in Chapter 9.

5.13 SELECTED PROTOTYPE LABYRINTH WEIR SPILLWAY EXAMPLES

The bibliography included herein contains references for many labyrinth spillway case studies; a summary of these structures is presented in Table 5.2.

Table 5.2 - Labyrinth Spillway Prototypes

Name	Location	Q_{design}	H_T	N	Source
		(m^3/s)	(m)	( )	
Accord Pond Dam	USA	-	0.46	2	Crookston et al. (2015)
Agua Branca	Portugal	124	1.65	2	Quintela et al. (2000)
Alfaiates	Portugal	99	1.60	1	Quintela et al. (2000)
Alijó	Portugal	52	1.23	1	Magalhães & Lorena (1989)
Alloway Lake Dam	USA	-	-	5	Crookston et al. (2015)
Antelope Creek Channel	USA	-	-	3	Crookston et al. (2015)
Arcossó	Portugal	85	1.25	1	Quintela et al. (2000)
Avon	Australia	1790	2.80	10	Darvas (1971)
Bartletts Ferry	USA	5920	2.19	20.5	Mayer (1980)
Belia	Zaire	400	2.00	2	Magalhães & Lorena (1989)
Beni Bahdel	Algeria	1000	0.50	20	Afshar (1988)
Berg	South Africa	270	2.50	2	Fourie (1999)
Boardman	USA	387	1.77	2	Babb (1976), Lux (1985)
Bospoort 1	South Africa	2465	4.20	7.5	ARQ (2008)
Bospoort 2	South Africa	1613	4.20	2.5	ARQ (2008)
Boyde Lake	USA	1209	0.59	59	Brinker (2005)
Calde	Portugal	21	0.60	1	Quintela et al. (2000)
Capital City Country Club	USA	-	2.44	1	Crookston et al. (2015)
Carty	USA	387	1.80	2	Afshar (1988)
Cloe Dam	USA	-	-	7	Crookston et al. (2015)
Castelletto-Nerv. Canal	Italy	25	0.12	24	Magalhães & Lorena (1989)
Cimia	Italy	1100	1.5	4	Lux & Hinchliff (1985)
Concourse Lake Dam	USA	-	1.07	2	Crookston et al. (2015)
Crystal Bridges	USA	-	-	2	Crookston et al. (2015)
Dog River	USA	1572	2.74	8	Savage et al. (2004)
DRA Detention Structure	USA	-	1.22	4	Schnabel (2013)
Dungo	Angola	576	2.4	4	Magalhães & Lorena (1989)
Eikenhof	South Africa	190	3.5	2	ARQ (1998)
Elmendorf Lake	USA	-	-	6	Crookston et al. (2015)
Estancia	Venezuela	661	3.01	1	Magalhães & Lorena (1989)
Forestport	USA	76	1.02	2	Lux (1989)
Fort Miller	USA	-	-	45	Crookston et al. (2015)
Garland Canal	USA	25.5	0.37	3	Lux & Hinchliff (1985)
Gema	Portugal	115	1.12	2	Magalhães & Lorena (1989)
Glen Park	USA	-	-	25	Crookston et al. (2015)
Grahamstown	Australia	628	1.4	8	Barker et al (2001)
Greystone	USA	-	1.45	1	Schnabel (2013)
Hard Labor Creek	USA	-	3.66	4	Schnabel (2013)
Harrezza	Algeria	350	1.9	3	Lux (1989)
Hollis G Lathem Reservoir	USA	-	2.44	3	Crookston et al. (2015)
Huntington Hills	USA	-	0.58	3	Crookston et al. (2015)

Indian Run	USA	-	2.44	2.5	Crookston et al. (2015)
Infulene Canal	Mozambique	60	1.00	3	Magalhães & Lorena (1989)
Juturnaiba	Brazil	862	0.70	4	Afshar (1988)
Kauffman	USA	128	5.0	5	Crookston et al. (2015)
Keddara	Algeria	250	2.46	2	Magalhães & Lorena (1989)
King Falls	USA	-	-	6	Crookston et al. (2015)
Kizilcapinar	Turkey	2270	4.6	5	Yildiz (1996)
Lake Brazos	USA	24609	-	24	Tullis and Young (2005)
Lake Natalie	USA	-	2.44	4	Crookston et al. (2015)
Lake Paupacken	USA	-	-	2	Crookston et al. (2015)
Lake Sovereign	USA	-	1.22	2	Crookston et al. (2015)
Lake Townsend	USA	3483	4.57	7	Tullis & Crookston (2008)
Lake Upchurch	USA	-	1.52	6	Crookston et al. (2015)
Leaser Lake	USA	289	3.26	2	Crookston et al. (2015)
Linville Land Harbor	USA	693	2.61	4	Schnabel (2013)
Little Blue Run	USA	-	-	10	Crookston et al. (2015)
Lyman Run	USA	-	-	8	Crookston et al. (2015)
Maguga	Swaziland	15000	8.25	9	Van Wyk et al (2006)
Midmar	South Africa	3052	4.15	10	ARQ (2002)
María Cristina Dam	Spain	5444	-	7	Page et al. (2007)
Meacham Grove	USA	-	-	4	Crookston et al. (2015)
Mercer	USA	239	1.83	4	CH2M-Hill (1976)
Navet Pumped Storage	Trinidad	481	1.68	10	Phelps (1974)
New London	USA	-	-	4	Crookston et al. (2015)
Ohau C Canal	New Zealand	540	1.08	12	Walsh (1980)
Opossum	USA	-	-	4	Crookston et al. (2015)
Pacoti	Brazil	3400	2.72	15	Magalhães & Lorena (1989)
Pine Run	USA	-	1.40	4	Crookston et al. (2015)
Pisão	Portugal	50	1.00	1	Quintela et al. (2000)
Pye Lake	USA	-	0.34	3	Schnabel (2013)
Quincy	USA	552	2.13	4	Magalhães & Lorena (1989)
Rapp Run Flood Control	USA	-	-	6	Crookston et al. (2015)
Rocklands-Berg	South Africa	-	-	2	Fourie (1999)
Roy F. Varner Reservoir	USA	-	2.13	8	Crookston et al. (2015)
São Domingos	Portugal	160	1.84	2	Magalhães & Lorena (1989)
Sam Rayburn Lake	USA	-	-	16	USACE (1991)
Santa Justa	Portugal	285	1.35	2	Magalhães & Lorena (1989)
Sarioglan	Turkey	490.7	1.06	7	Yildiz (1996)
Sarno	Algeria	360	1.5	8	Afshar (1988)
South River No 29	USA	-	3.05	3	Schnabel (2013)
Standley Lake	USA	1539	1.98	13	Tullis (1993)
Teja	Portugal	61	1.05	1	Quintela et al. (2000)
Upper Dam - Rangeley	USA	-	1.19	4	Schnabel (2013)
Ute	USA	15570	5.79	14	Houston (1982)
Weatherford	USA	-	-	4	Tullis (1992)
Woronora	Australia	1020	1.36	11	Darvas (1971)

The plan and profiles of a few of these installations are shown on the following pages to illustrate the varied configurations that have been used.

- **MAGUGA DAM, SWAZILAND** (Information courtesy of Aurecon, South Africa)

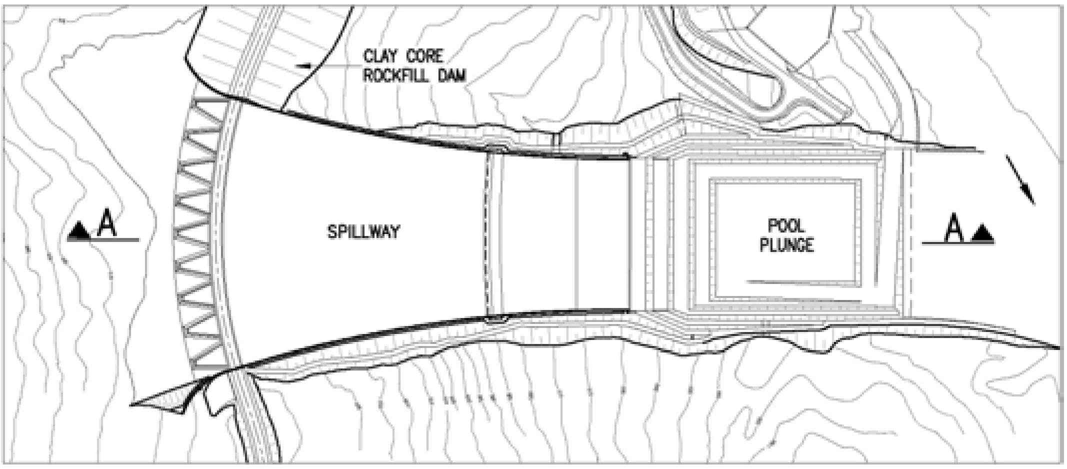

Figure 5.12: Maguga Dam Labyrinth Spillway Layout

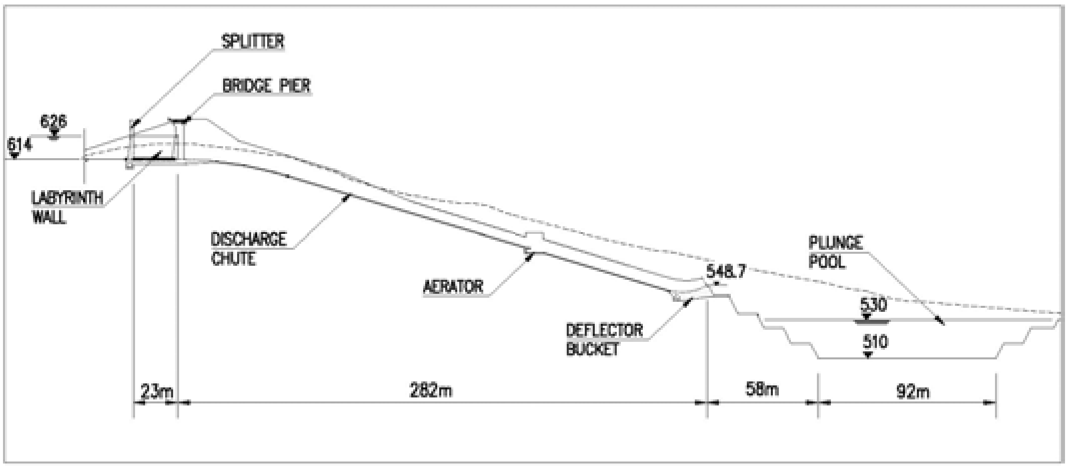

Figure 5.13: Maguga Dam Labyrinth Spillway Longitudinal Section

A bridge had to be constructed over the labyrinth spillway. In order to accommodate the road alignment over the spillway and the embankment, the labyrinth was curved to a radius of 300 m. The discharge chute tapers from 181 m wide at the labyrinth to 100 m wide at the deflector bucket lip. In order to make the transition as gradual as possible, curved sidewalls with a radius of 876 m was fitted tangentially to both the labyrinth and the deflector bucket. This resulted in the individual flows from each cycle to be directed down the discharge chute in such a way that no cross waves developed.

- **MIDMAR DAM, SOUTH AFRICA** (Information courtesy of ARQ, South Africa)

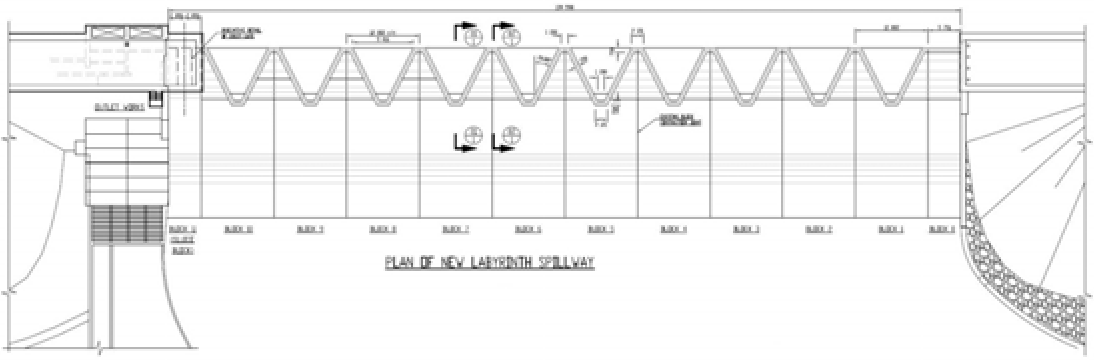

Figure 5.14: Midmar Dam Labyrinth Spillway Layout

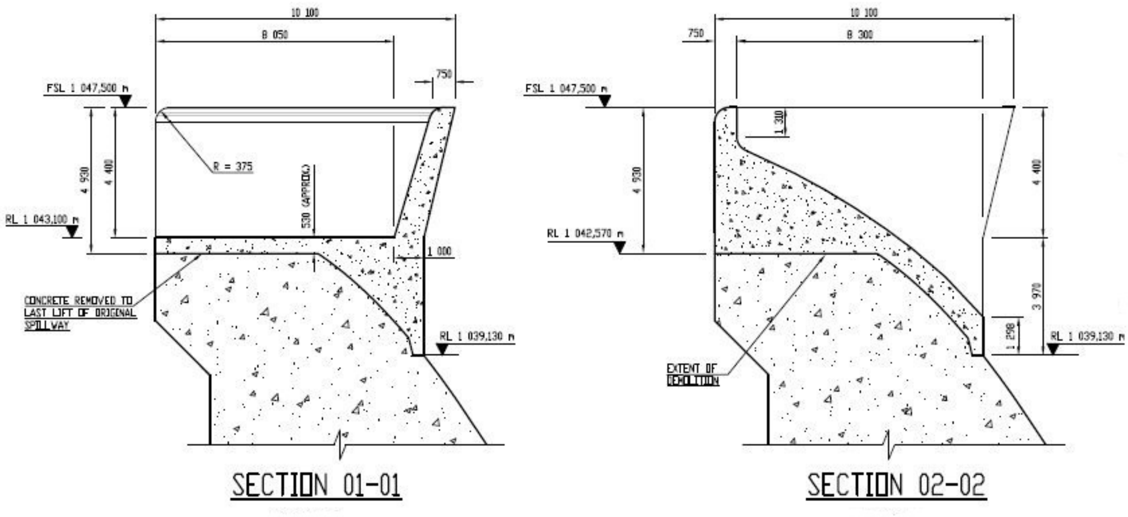

Figure 5.15: Midmar Dam Labyrinth Spillway Sections

This structure was incorporated on the crest of an existing mass concrete gravity dam originally intended to be raised through the addition of radial gates. It is believed to be the first implementation of a labyrinth in this configuration in the world. The bridge, piers and trunnion points, as well as a portion of the ogee crest were demolished to make way for the new labyrinth. Additional mass was provided on the downstream between the labyrinth bays to assist with overall stability.

The primary motivation for the deviation from the original design was the reliability of a fixed labyrinth and its capacity to safely attenuate the Design and Safety Evaluation Floods.

The labyrinth design was based on the bucket depth and the structure was model tested using three full cycles at 1:27 scale to confirm theoretical capacities.

- **LAKE TOWNSEND, UNITED STATES** (Information courtesy of Schnabel Engineering, USA)

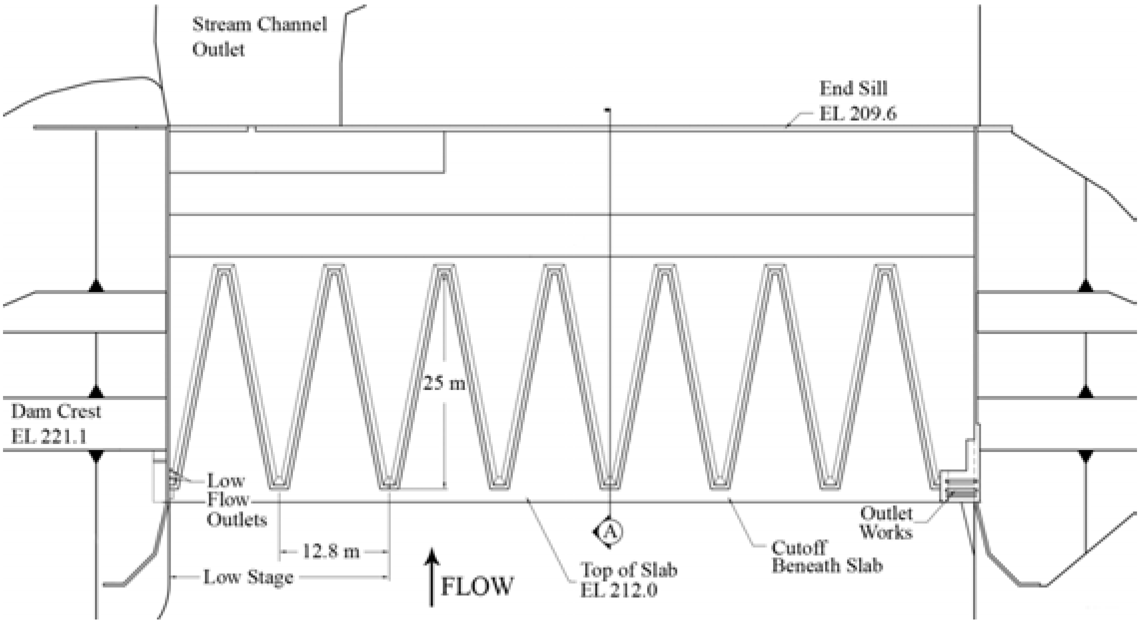

Figure 5.16: Lake Townsend Labyrinth Spillway Layout

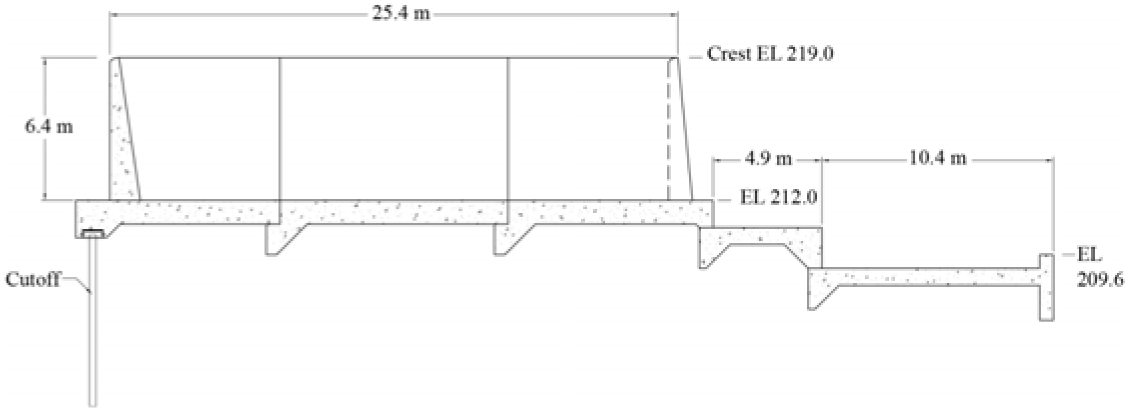

Figure 5.17: Lake Townsend Labyrinth Spillway Section

The labyrinth spillway (6.1 m high) at Lake Townsend Dam replaced an 84 m wide gated concrete spillway. The 7-cycle labyrinth features a staged crest with 2-cycles at normal pool and 5 cycles comprising the high stage. It is also subject to tailwater submergence for large flows. The design was tested with physical modelling performed at the Utah Water Research Laboratory and CFD modelling performed at Idaho State University (Paxson et al. 2007). The spillway design flow is about 3,483 m^3/s; the embankment dam includes overtopping protection to allow passage of the spillway design flood (SDF) plus flows resulting from the failures of three upstream dams.

Figure 5.17: Lake Townsend Labyrinth Spillway

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6. PKW SPILLWAYS

6. 1 GENERAL DESCRIPTION

The recently developed Piano Key weir (PKW) spillway, which is an innovative structure that can convey very high specific discharges, is a variation of the traditional labyrinth weirs, originally devised to circumvent some drawbacks of the latter (Barcouda *et al*., 2006). Using a rectangular layout and ramped floors which create overhanging or cantilevered apexes, the PKW is structurally simple and efficient, can be placed on existing or new gravity dam crest sections and multiplies significantly the discharge capacity compared to a standard weir of similar width (Ouamane and Lempérière, 2003).

Figure 6.1 - View of the PKW spillway of Gloriettes Dam in France during construction

(Photo courtesy of EDF)

Figure 6.2 - View of the PKW spillway of Malarce Dam in France during spillage
(Photo courtesy of EDF)

The succession of inclined apexes turned alternatively in upstream and in downstream direction gives the name Piano Key weir. Compared to a rectangular labyrinth weir with a common crest layout (in plan form), the PKW has the main advantage that it can be more easily installed at sites featuring limited foundation space (e.g., crest of a gravity dam). In addition, the ramped floors reduce the vertical walls height and thus the volume of reinforcing steel required in concrete. These are the reasons why PKW spillways are an efficient and economical solution for the increase of the flood releasing capacity at existing dams.

PKW has been first proposed by Hydrocoop in collaboration with the Hydraulic Laboratory of Electricité de France (France), the Roorkee University (India) and the Biskra University (Algeria) (Ouamane and Lempérière, 2003). Since its invention, several works have been carried out all over the world to understand its hydraulic behaviour, optimize its design and objectify its advantages and drawbacks (see for instance Blanc and Lempérière, 2001; Barcouda *et al*., 2006; Ouamane and Lempérière, 2006; Truong Chi *et al*., 2006; Machiels *et al*., 2011a & 2014; Leite Ribeiro *et al*., 2012a & b; Machiels, 2012; Anderson and Tullis, 2012 & 2013). Studies are still ongoing, for instance with a trapezoidal layout (Cicéro *et al*., 2013a).

The first PKW was installed by EDF in 2006 at Goulours dam in France (Laugier, 2007). Since then PKW have been used to increase the flood discharge capacity of five other EDF dams, namely St. Marc (2008), Etroit (2009), Gloriettes (2010), Malarce (2012) and Charmine (2014) or as new overflow structure (Escouloubre (2011). Lessons learned from the design of these first PK weir spillways can be found in Vermeulen *et al*. (2011) and Laugier *et al*. (2013) or Laugier *et al*. (2009). Other PKW are presently in operation in Vietnam (Ho Ta Khanh *et al*., 2011a & 2012), Sri Lanka (Jayatillake H. and Perera K., 2013), Switzerland (Eichenberger P., 2013) and Scotland (Ackers *et al*., 2013). New PKW are under study or construction in Vietnam (Ho Ta Khan *et al*., 2011a & 2012), France (Dugué *et al*., 2011; Erpicum et al., 2011b; Loisel et al., 2013; Bail et al., 2013), Algeria (Erpicum *et al*., 2012), South Africa (Botha A. *et al*., 2013) or India (Das Singhal & Sharma, 2011). These works are part of the rehabilitation of existing dams (to increase discharge capacity) or new projects, with PKW built in the river (diversion weir), on the top of a gravity dam, or on a reservoir bank.

Most of the information available so far on PKW has been published in two books (Erpicum *et al*, 2011a and 2013a), edited following two specialized workshop held in Belgium (2011) and France (2013).

6. 2 GEOMETRY AND TYPES

The PKW geometry may appear complex. It involves indeed a large set of parameters. In order to unify the notations, a nomenclature specific to the structure has been developed (Pralong *et al*., 2011a).

Following this nomenclature, the "PKW-unit" can be defined as the basic structure of the weir. It is made of two side walls, one inlet and two half-outlet keys (Figure 6.3). The main geometric parameters of the structure are the heights of the inlet and outlet keys P_i and P_o, their widths W_i and W_o, the unit width W_u, the number of PKW-units N_u, the longitudinal crest length B_h, the lengths B_o and B_i of the up- and downstream overhangs, the base length B_b and the wall thickness T_s. Subscripts *i, o* and *s* refer respectively to the inlet key, the outlet key and the side wall. W_u is equal to $W_i + W_o + 2T_s$ and the total width W of the weir is equal to N_u times W_u. The PKW-unit developed crest length L_u of a PKW-unit is equal to $W_u + 2B_h$ and the total developed crest length L of the weir is equal to N_u times L_u. Parapet walls (vertical extensions of the crest) may be added to the weir. Their height is referred to as P_p.

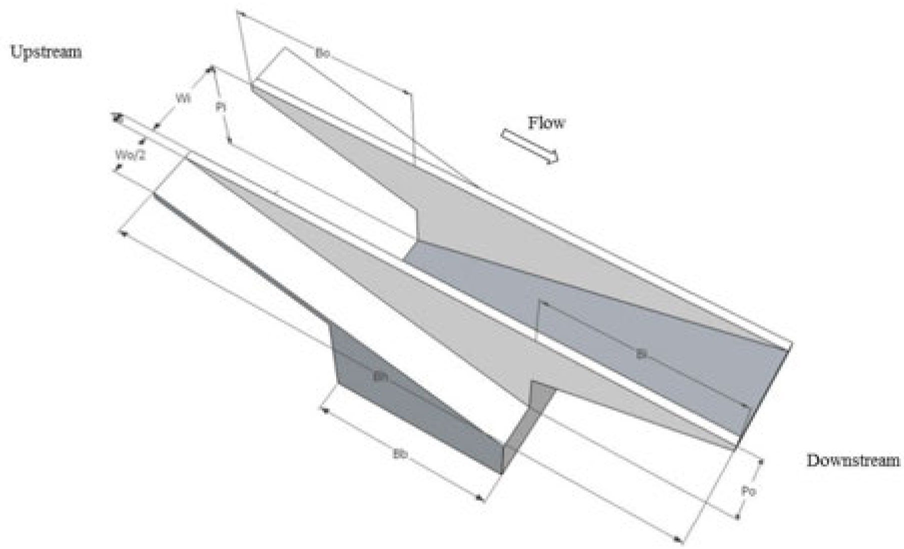

Figure 6.3 - Sketch of a PKW unit and main notations (Erpicum *et al*. 2013b)

Depending on which PKW apexes have overhangs (upstream and/or downstream), PKW have been classified in 4 types (Truong *et al*., 2006): type-A with symmetric overhangs, type-B with a single upstream overhang, type-C with a single downstream overhang and type-D without overhang (Figure 5.16).

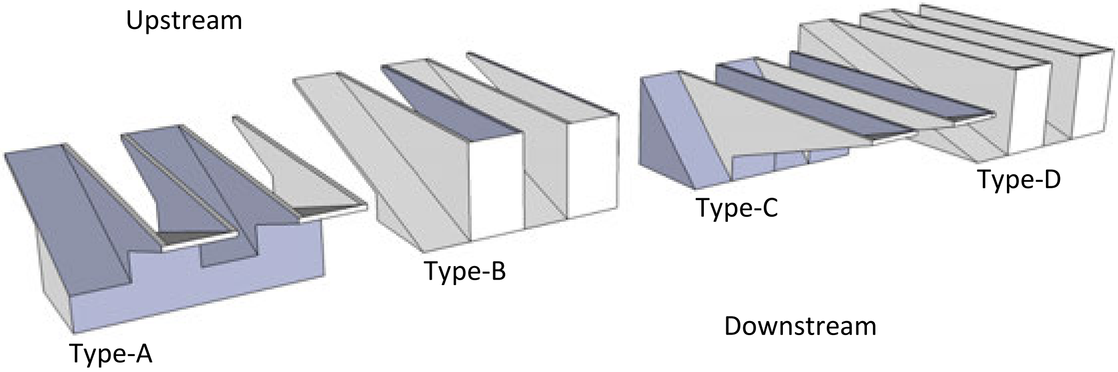

Figure 6.4 - Types of PK weir (adapted from Erpicum *et al*. 2013b)

6. 3 DISCHARGE CAPACITY

The PKW is a free surface weir and its discharge Q_P is thus proportional to the upstream head H as

$$Q_P = \propto \sqrt{2gH^3} \tag{6.1}$$

As summarized by Leite Ribeiro *et al* (2012a), two approaches may be chosen to derive the proportion factor, which represents the effect of the crest length and shape.

Referring to the developed crest length L, the discharge coefficient $C_{P,L}$ is closely related to the crest shape. Eq. 6.1 writes as (Leite Ribeiro *et al*., 2012b)

$$Q_P = C_{P,L} L\sqrt{2gH^3} \tag{6.2}$$

In this approach, L varies with the head as the effective crest length decreases with increasing heads because of local submergence on the upstream apex, for instance. $C_{P,L}$ also varies with the head as it includes both frontal and side weirs effects.

Referring to the width of the weir W, Eq. 6.1 writes (Ouamane & Lempérière, 2006; Machiels *et al*., 2011a)

$$Q_P = C_{P,W} W\sqrt{2gH^3} \tag{6.3}$$

with the discharge coefficient $C_{P,W}$ accounting for both crest shape and developed length effects.

Whatever the approach used to model the PKW discharge, it is of common use to look at its discharge capacity by comparison with the one of a standard linear weir of same width (Q_s). The discharge increase ratio r is defined by Leite Ribeiro *et al* (2012a & b) as

$$r = \frac{Q_P}{Q_S} \tag{6.4}$$

Considering Eq. 6.3, Eq. 6.4 writes

$$r = \frac{C_{P,W}}{C_S} \tag{6.5}$$

where C_S is the discharge coefficient of the standard linear weir.

Whatever its geometry, a PKW is much more efficient than an ogee crested weir of same width, especially for low heads (Figure 6.5). This explains why most of the existing PKW have been designed for a maximum *H/P* ratio lower than 1 (Pfister *et al.*, 2012). This high increase in efficiency is due to the developed length of the crest which equals several times the weir width while the discharge coefficient is close to the one of a sharp crested weir. Regarding a traditional labyrinth weir with the same cycle shape in plan view (same crest footprint), a PKW is around 10% more efficient for a head H equal to its height P_i, as shown in a study performed by Anderson and Tullis (2012).

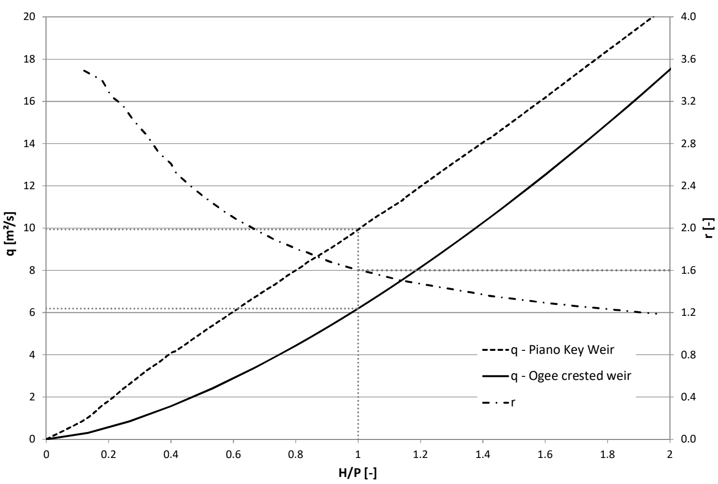

Figure 6.5 - Discharge per unit width W of a PKW compared to an ogee crested weir – PKW: P=P_i=P_o=2 m and L/W =5; Ogee crested weir: design head=2 m, C_S=.494 (Erpicum *et al*. 2013b)

6. 4 MAIN GEOMETRICAL PARAMETERS

The PKW discharge efficiency results from the cumulative effects of (i) three differents types of overflow (linear weir flow over the inlet key apex, linear weir flow over the outlet key apex, and lateral weir flow over the sidewalls), (ii) the developed length of the crest and (iii) the upstream head. Clinging, leaping or springing nappes may happen, depending upon the geometrical parameters of the structure and the head (Machiels *et al*., 2011a). The greater the head, the smaller the PKW's efficiency increase compared to a linear weir, according to the progressive reduction of the effective developed length and the saturation of the outlet.

Ouamane & Lempérière (2006) and Leite Ribeiro *et al* (2012a) showed that the crest length magnification ratio L/W is the main parameter controlling the discharge capacity. A value of 5 seems to be a reasonable compromise between weir efficiency and structure complexity (Lempérière, 2009; Lempérière *et al*, 2011), while L/W ratio of existing PKW ranges from 4 to 8 (Pfister et al, 2012). In a further step, Machiels *et al* (2011a and 2012b) identify the key height P, the key width ratio W_i/W_o and the overhangs positions ratio B_o/B_i as the main geometric parameters influencing the PKW hydraulic efficiency for a given L/W ratio.

These studies showed that it is of major importance to increase all the geometrical ratios which increase the inlet cross section, as this last one can be seen as the "engine" of the PKW. Increasing the inlet cross section decreases the flow velocity along the lateral crest, and thus increases its efficiency. Maximum value of the parameters is reached when the release capacity of the outlet key is affected. Indeed, the outlet key can be seen as the "brake" of the PKW. Too small of an outlet key cross section and slope increase the free surface level over the lateral crest elevation (local submergence) and thus significantly limit the weir efficiency.

A PKW design with a height ratio P/W_u equal to 1.3, a key widths ratio W_i/W_o equal to 1.25 and an overhangs lengths ratio B_o/B_i equal to 3 was found by Machiels (2012) to provide the highest discharge capacity when the L/W ratio is equal to 5. This finding is consistent with the findings of Leite Ribeiro *et al* (2012a), Anderson & Tullis (2013) or Lempérière *et al* (2011).

However, the Machiels' study also highlights the importance of the technical and economic criteria in the definition of an optimal PKW design. A high PKW (P/W_u = 1.3) is more effective from a hydraulic point of view and should thus be considered for new dam projects, shorter PKW designs ($P/W_u \approx 0.5$), though less hydraulically efficient, would be more practical for dam rehabilitation projects. For the later, W_i/W_o and B_i/B_o ratios equal to 1 are relevant (Erpicum *et al*, 2104).

Each PKW's design will be a compromise of the aforementioned parameters in order to find an appropriate answer to the specific project's features. In any case, for a given available width, the PKW has (i) a great efficiency at low relative heads (H/P_i), and (ii) can provide a discharge capacity 2 to 5 times greater than an ogee crest using the same width.

6. 5 TAILWATER SUBMERGENCE

Weir tailwater submergence occurs when the elevation of the downstream water surface exceeds the crest elevation and increases the upstream water elevation for a give discharge. For most applications, PKW submergence would only be a consideration for in-channel or in–river structures and not for top-of-dam applications.

Similarly to labyrinth weirs, PKW tailwater submergence has been studied mainly using physical modeling in channel configurations (Belaabed and Ouamane, 2011; Cicéro and Delisle, 2013b; Dabling and Tullis, 2012). It appears that the PKW behaviour regarding submergence is very sensitive to the geometry and type (Figure 6.6), and deserves particular care to be predicted.

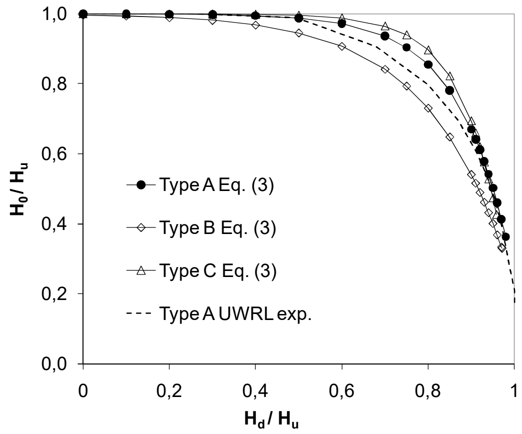

Figure 6.6 - Comparison of the sensitivity to submergence of three PKW different types (Cicéro and Delisle, 2013b)

Dabling and Tullis (2012) concluded that for relatively low levels of submergence, the PKW requires less upstream head relative to the labyrinth weir to pass a given discharge. This increase in efficiency was smaller than 6%, and this trend reversed at higher submergence levels.

6. 6 FLOATING DEBRIS

Debris is often transported to hydraulic structures during flood events (Pfister *et al*. 2013b). The accumulation of debris on PKW should be considered in situations where there is the potential for debris to reduce spillway capacity. For example, storm events in forested catchments may result in flows laden with woody debris. Depending upon the spillway hydraulics and site conditions, this type of debris may pass over the weir during floods and require little maintenance. Conversely, conditions may result in large accumulations of woody debris that reduce spillway capacity (-25% may be observed) and raise safety concerns.

A systematic laboratory study conducted by Pfister et al. (2013b) to evaluate the interaction between various PKW geometries and woody debris types and sizes indicated that floating debris blockage probability is highly influenced by trunk diameter and upstream head. The effects of debris accumulation on the upstream head varied with the value of the debris-free reference upstream head condition. At lower upstream reference head values, the cumulative debris tests indicated a relative increase of the debris-associated upstream head of approximately 70%; higher upstream reference head values produced upstream head increases limited to approximately 20%.

6. 7 AERATION AND ENERGY DISSIPATION

The observed downstream nappes on existing PKW prototypes, mostly equipped with aeration pipes below the inlets apex, are usually well-aerated and not subjected to clinging or vibration, even for very low heads (Figures 6.7 and 6.8). However, no measurement of air entrainment in aeration pipes has ever been done on prototypes or adequately sized hydraulic models. On Malarce spillway (France), in 2014 EDF equipped the prototype aeration pipes with air entrainment measurement devices. Data will be published in the forthcoming years (Pinchard *et al*., 2013).

Figure 6.7 - Flow over the Malarce dam PKW (France) with an upstream head of a few cm
(Photo courtesy of EDF)

Figure 6.8 - Flow over the Escouloubre PKW (France) with an upstream head of a few cm
(Photo courtesy of ULg-HECE)

The possible high specific discharge downstream of a PKW implies that great care has to be provided to the downstream energy dissipation structure design, in particular when the weir is located on the crest of a high dam.

The energy dissipation solutions already designed and build downstream of PKW prototypes cannot be generalized and have all been found in an innovative way using physical modelling (Figures 6.9 and 6.10) (Bieri et al., 2011; Erpicum et al., 2011b; Leite Ribeiro et al., 2011).

Figure 6.9 - Low slope stepped spillway channel downstream of the Gloriette dam PKW

(Photo courtesy of EDF)

Figure 6.10 - Smoth spillway and inclined flip bucket downstream of the Saint Marc dam PKW
(Photo courtesy of EDF)

Laboratory tests have been performed with PKW upstream of stepped spillways (Ho Ta Khanh *et al*., 2011b; Silvestri *et al*., 2013a & b). They showed that the flow downstream of a PKW is always well aerated, i.e., that the inception point is located immediately at the weir toe (Figure 6.11).

In the scope of the Gloriette dam project, Bieri et al. (2011) showed that PKW combined with stepped chutes may lead to pronounced downstream energy dissipation.

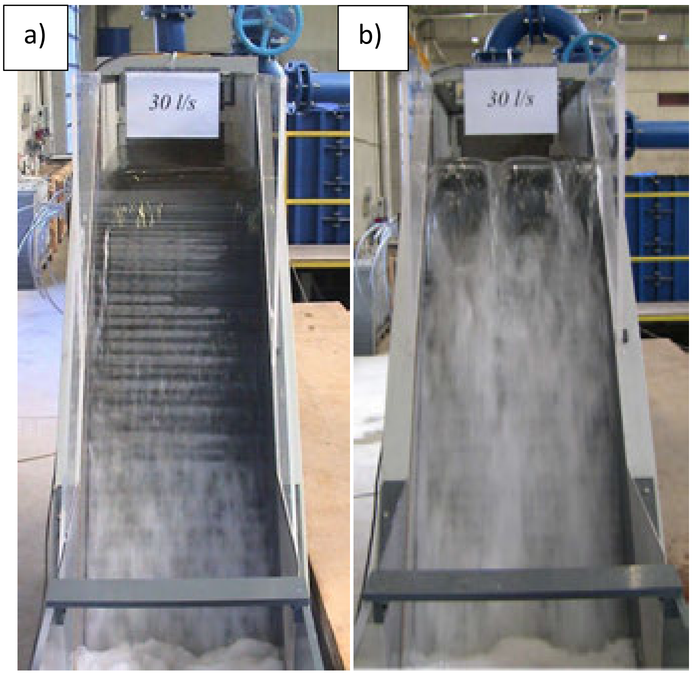

Figure 6.11 - Modification in the inception point location along a stepped spillway with (a) a standard ogee-crested weir, and (b) a PKW (q = 0,06 m²/s) (Adpated from Silvestri *et al*., 2013a)

6. 8 HYDRAULIC DESIGN EQUATIONS FOR A-TYPE PIANO KEY WEIRS AND DESIGN METHOD

Hydrocoop proposed a very simple preliminary method to specify the discharge per unit width q under an upstream head H for an A-type PKW with a maximum height of the labyrinth walls equal to P_m (Lempérière, 2009). This formula (Eqn. 6.6) applies only in the range of parameters specified by the author.

$$q = 4.3\, H\, P_m^{0.5} \qquad (6.6)$$

In the meantime comprehensive and systematic model test series have been conducted in several laboratories (Schleiss, 2011). Based on such tests series, three general hydraulic capacity equations for Type-A PKWs have been proposed and validated (Kabiri-Samani and Javaheri, 2012; Leite Ribeiro *et al*., 2012a; Machiel *et al*., 2014). It is important to notice that these experimentally obtained equations must be used only within the limits of the parameter range specified by their authors, as clearly demonstrated by Pfister *et al*. (2012). It is worth noticing that these limitations should strictly be respected as small discrepancies may result in significant changes in the PKW discharge coefficient.

Pfister & Schleiss (2013a) compared these three hydraulic design formula for a hypothetical symmetrical Type-A PKW, placed on a Roller Compacted Concrete (RCC) gravity dam with a $W = 100$ m wide chute spillway on its downstream face, a dam height of $P_d = 30$ m below the PKW foundation. A design discharge of $Q_D = 2\ 500$ m^3/s was considered, resulting in a specific discharge of 25 m^2/s as commonly used on stepped spillways.

The PKW geometry had a total streamwise length B = 8,00 m, $P = P_i = P_o = 5.00$ m as vertical height, $T_S = 0.35$ m as wall thickness, $R = 0$ m (without parapet walls), $W_i = 1.80$ m as inlet key width, $W_o = 1.50$ m as outlet key width, and $B_i = B_o = 2.00$ m as overhang lengths. The following characteristics result from this PKW geometry: cycle width $W_u = W_i + W_o + 2T_S = 4.00$ m, number of cycles $N = W/W_u = 25$, developed crest length $L = W + (2NB) = 500$ m, $L/W = 5.00$, $B/P = 1.60$, $W_i/W_o = 1.20$, $B_i/B = B_o/B = 0.25$, and $S_i = S_o = 0.83$.

Since the tests of the three hydraulic design formulas mentioned before, were performed with different crest shapes, the results were normalized to a broad-crested weir. The resulting discharge capacity curves are shown in Figure 6.12 within their application limits of *H/P*. As it can be seen, the rating curves of the three PKW studies are similar. In general, the empiric equation of Kabiri-Samani and Javaheri (2012) predicts the highest discharge capacity. Additionally, the rating curve of a linear standard crest profile (ogee) is also shown in Fig. 6.12, derived from Vischer and Hager (1999) considering a design head of $H_D = 5.00$ m for Q_D.

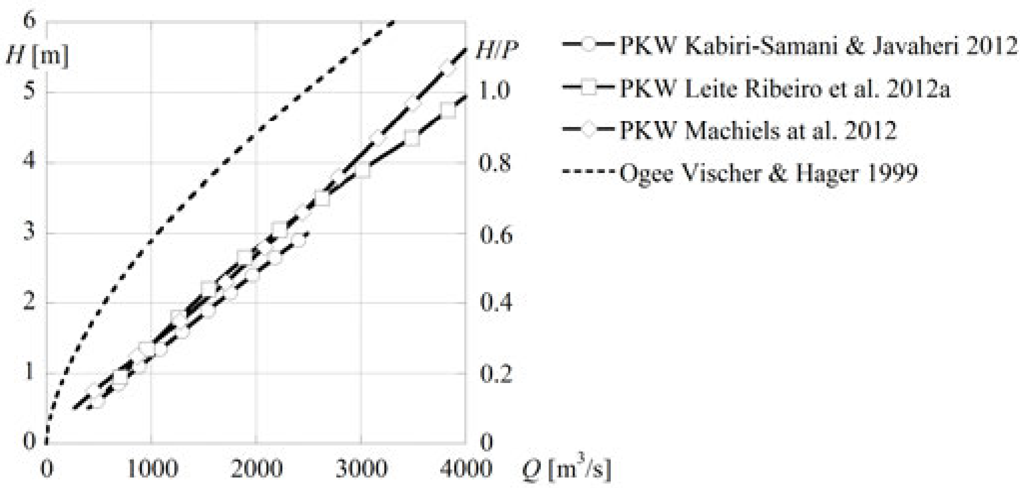

Figure 6.12 - Comparisons of the three PKW Type-A discharge capacity equations and the rating curve of an ogee crest weir (Pfister and Schleiss, 2013a)

For practical application it may be concluded that the most appropriate PKW hydraulic design formula is dependent upon the application range and the prototype characteristics. The crest shape can serve as a second criterion for the formula selection. If several formulas are within the application range of the prototype, the most conservative result may be considered, in order to be on the safe side.

In addition, Machiels *et al*. (2011b & 2012a) proposed a simple preliminary design method to select a PKW geometry from existing rating curves and specific project constraints (maximum discharge, reservoir level, available width, etc).

6. 9 PHYSICAL AND NUMERICAL (CFD) MODELLING

The recent development of PKW leads to intensive comparison between 3D numerical modeling and laboratory hydraulic tests on numerous geometries. These works clearly show that the use of (commercial) 3D software is very promising to predict the discharge capacity of a PKW geometry with a good accuracy (+/-10 %) (Pralong *et al*., 2011b; Lefebvre *et al*., 2013).

Besides, the freeware WOLF1D PKW (available on http://www.pk-weirs.ulg.ac.be) has been developed by the HECE research group of the University of Liege, on the basis of the large scale physical tests of Machiels (2012). From a PKW geometry, this freeware is able to compute very quickly the flow over half a unit of the weir for a given range of discharge upstream of the structure. The results of the computation are the weir head / discharge curve (+/- 10% accuracy), the water level upstream of the weir depending on the discharge and the distribution of water depth and discharge along both the inlet and the outlet.

Nevertheless, CFD modelling cannot reproduce all the flow characteristics such as instabilities for instance, the effects of complex approach flow conditions or specific abutment geometry as well as the consequences of blocking by floating debris. Furthermore, downstream flow and energy dissipation are critical issues for the overall spillway efficiency which cannot be assessed accurately with numerical models.

Therefore the final geometry of a PKW together with the energy dissipation system should be tested on a physical model, which is the only way to reproduce properly the complete flow features with the required level of precision. It is worthwhile mentioning that all large PKW prototypes have been validated and optimized by physical modelling.

6. 10 STRUCTURAL AND CONSTRUCTION ISSUES

Structural issues highly depend upon the PKW geometry (size and number of overhangs), load cases (ice influence or site seismicity for example), respective unit rates and project features (location at a gravity dam crest or isolated on a bank...).

Existing references have highlighted some specific ratios, such as:

- 50 to 100 kg reinforcement /m^3 concrete
- 60 to 130 m^2 formwork / m of width

It is common practice to add concrete or anchors in order to reach the required stability factors of safety. Thermal load cases (one face is under the sun, the other one protected by the water) or ice load may easily become controlling sizing parametres.

Particular attention should also be given to construction features, such as pre-fabrication (Figure 6.13) or use of steel instead of concrete, as on many prototypes built up to now.

Fig 6.13 - Back Esk Reservoir spillway – PKW made of prefabricated concrete elements (Courtesy of Black and Veatch Ltd)

6. 11 ENGINEERING

So far, PKW have been primarily been designed and constructed as flood-routing spillways in the following three configurations:

1. narrow valley where only a small place is available for installing a new structure;

2. gravity dam, where the PKW - easily implemented on the crest of the dam thanks to its small footprint - allows maximizing the active storage capacity of the reservoir for a given Maximum Water Level. Generally, PKW is combined with surface or bottom outlet gates for the fine tuning of the upstream water level regulation and the flushing of the reservoir;

3. long barrage (gated structure dam) in flat areas. In such a case, the PKW (that may be associated with gates, whose number is then smaller than in a traditional alternative), minimizes the inundated areas during floods by lowering the Maximum Water Level compared to a solution with a linear weir.

According to the actual hydraulic knowledge, future design practice may be cost orientated. For example, research regarding (i) standard hydraulic profiles (allowing savings on the design and the construction), or (ii) best compromise between PKW, gates and fuse devices (in order to delay the tilting of the first fuse element).

The Van Phong barrage in Vietnam, 475-m long and 7- m high on the river bed, with a 15 400 m^3/s design flood, is a good example of the third configuration (Figure 6.14) mentioned above. The initial design included 28 radial gates with 15-m width and 7.5-m height. The final design, for the same Maximum Water Level, included only 10 radial gates (and 8 would have been probably enough) in the central part and 60 PKW units with a total length of 302 m on each side. Compared to a labyrinth weir solution, the PKW offered a smaller footprint on the rather deep bedrock, which required less excavation and concrete volume. The main advantages of the final alternative were: (i) the investment and maintenance cost savings, (ii) a higher safety level in operation, and (iii) a better integration with the environment. Taking into account the recent non-functioning of several gates during floods, many Vietnamese engineers think that a safe solution is now to combine, as much as possible, gates and free overflow crest spillway. In this scope, the PKW is often a good alternative, particularly when the available crest length is limited.

Figure 6.14 - The Van Phong barrage in Vietnam
(Courtesy of M. Ho Ta Khanh)

A new project with similar configuration is now under design for the Xuân Minh barrage.

6. 12 ONGOING RESEARCH

Ongoing research and design progress are orientated towards new developments of the PKW geometry (side wall angle narrowing the inlet key and widening the outlet one; new hybrid configurations using rectangular labyrinth and PKW) and the analysis of downstream flow feature (aeration and energy dissipation).

Research also concerns structural and construction aspects such as the use of steel or combined steel/concrete structures as well as pre-fabrication.

6. 13 PROTOTYPE PIANO KEY WEIR EXAMPLES

The bibliography included in the end of the chapter contains references for many PKW prototypes.

A summary of these structures is presented in following table.

Table 1: Piano Key Weir Spillways Prototypes

Name	Country	Configuration	Q_{design} (m^3/s)	P (m)	H_{design} (m)	Source
Black Esk	Scotland	Bellmouth crest	183	2.10	0.97	Ackers et al. 2013
Campauleil*	France	Dam crest	120	5.35	0.90	Laugier et al. 2013
Charmines	France	Dam crest	300	4.38	1.00	Laugier et al. 2013
Dak Mi 4B	Vietnam	Dam crest	500	3.75	2.00	Ho Ta Khanh et al. 2014
Dak Mi 4C	Vietnam	Dam crest	200	2.50	1.00	Ho Ta Khanh et al. 2014
Dak Rong 3	Vietnam	Dam crest	6550	5.00	3.50	Ho Ta Khanh et al. 2014
Emmenau	Switzerland	River	4.35	1.20	0.45	Eichenberger, 2013
Escouloubre	France	Bank	13	1.80	0.65	Laugier et al. 2013
Etroit	France	Dam crest	82	5.30	0.95	Laugier et al. 2013
Gage*	France	Right bank	455	6.00	1.75	Laugier et al. 2013
Giritale	Sri Lanka	River		2.40	0.45	Jayatillake and Perera. 2013
Gloriettes	France	Right bank	90	3.00	0.80	Laugier et al. 2013
Goulours	France	Right bank	68	3.10	0.95	Laugier et al. 2013
Hazelmere*	South Africa	Dam crest	4300	9.00	3.23	Botha et al. 2031
Loombah	Australia	Bank	528	2,5	3,09	GHD Australia
Malarce	France	Dam crest	570	4.40	1.50	Laugier et al. 2013
Raviège*	France	Dam crest	300	4.67	1.40	Laugier et al. 2013
Saint Marc	France	Dam crest	138	4.20	1.35	Laugier et al. 2013
Van Phong	Vietnam	Barrage (River)	8750	5.00	5.20	Ho Ta Khanh et al. 2014

* not yet constructed

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7. TUNNEL, SHAFT AND VORTEX SPILLWAYS

7.1. INTRODUCTION

This chapter considers different kinds of tunnel spillways, such as high-level, low-level (or low outlet), vortex, shaft and orifice tunnels. The general arrangement strategies, the control structure, the conveyance structure and the terminal structure, as well as operation issues, risk analysis and considerations, are discussed. Some new developments, innovations and applications on tunnel spillway design for high dams are specially given, with emphasis in what have been achieved in the past 20 to 30 years. Risk analyses both on cavitation risk control and emergency operation are discussed. Some good and valuable cases are given as references.

7.2. GENERAL ARRANGEMENT STRATEGIES FOR HIGH HEAD SPILLWAYS

7.2.1. Location, type and size

A tunnel spillway can be arranged on one side or on both sides of dam abutments. This last case will happen when a dam is built in a narrow valley with large flood flows and where surface discharge facilities are not sufficient to discharge the floods. The selection and design of tunnel spillways will depend on the type of dam, the total discharge capacity, the river diversion scheme and on operation and management requirements. Table 7-1, at the end of this chapter, gives the characteristics of some typical large tunnel spillways in the world which are under design, construction and/or operation.

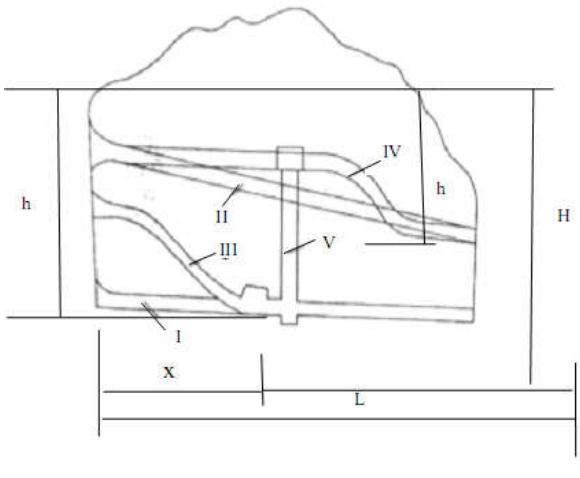


Fig. 7.1 – Classification of tunnel spillway layout by longitudinal sectional arrangement

Figure 7.1 depicts a classification of five typical tunnel layouts, based on longitudinal section arrangements. Tunnels can also be classified by types of energy dissipation, as shown in Fig. 7.2.

The flow energy dissipation in Type I to Type IV tunnels happens outside the tunnels. High speed flow carries the huge amount of energy to the river reach downstream. On the other hand the flow energy in Type V tunnel is dissipated inside the tunnel and the flow reaches the river with small velocities.

The layouts of Type I and II tunnel spillways are relatively simple. The tunnel is almost straight and the elevation difference between inlet and outlet, in most cases, is not too large, allowing its application in both high and low dams, depending on the operation purpose of flow discharge; see Fig. 7.3.

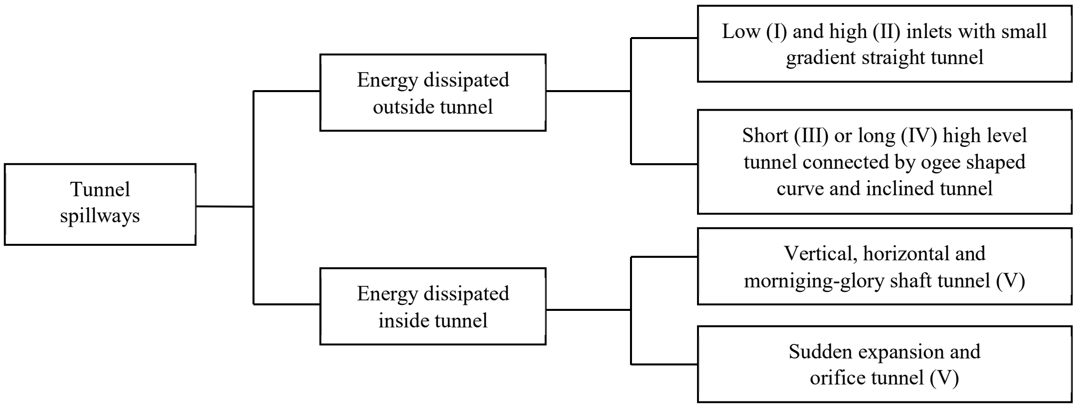
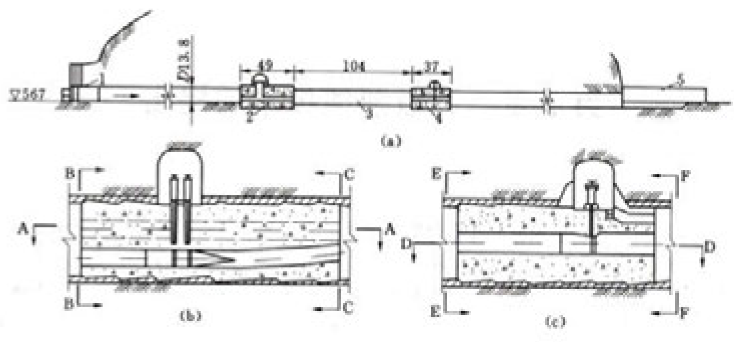
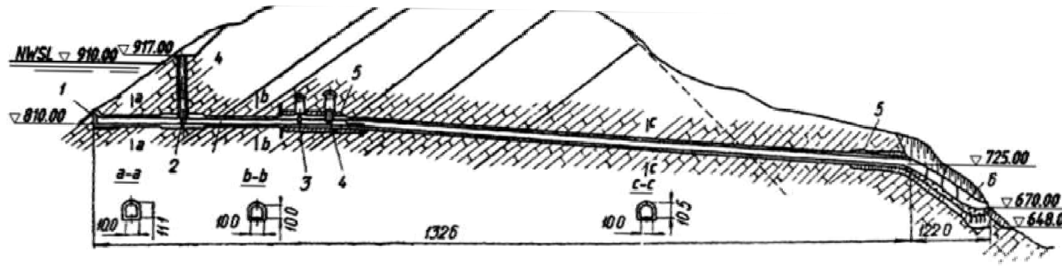


Fig. 7.2 Classification of tunnel spillway layout by type of energy dissipation

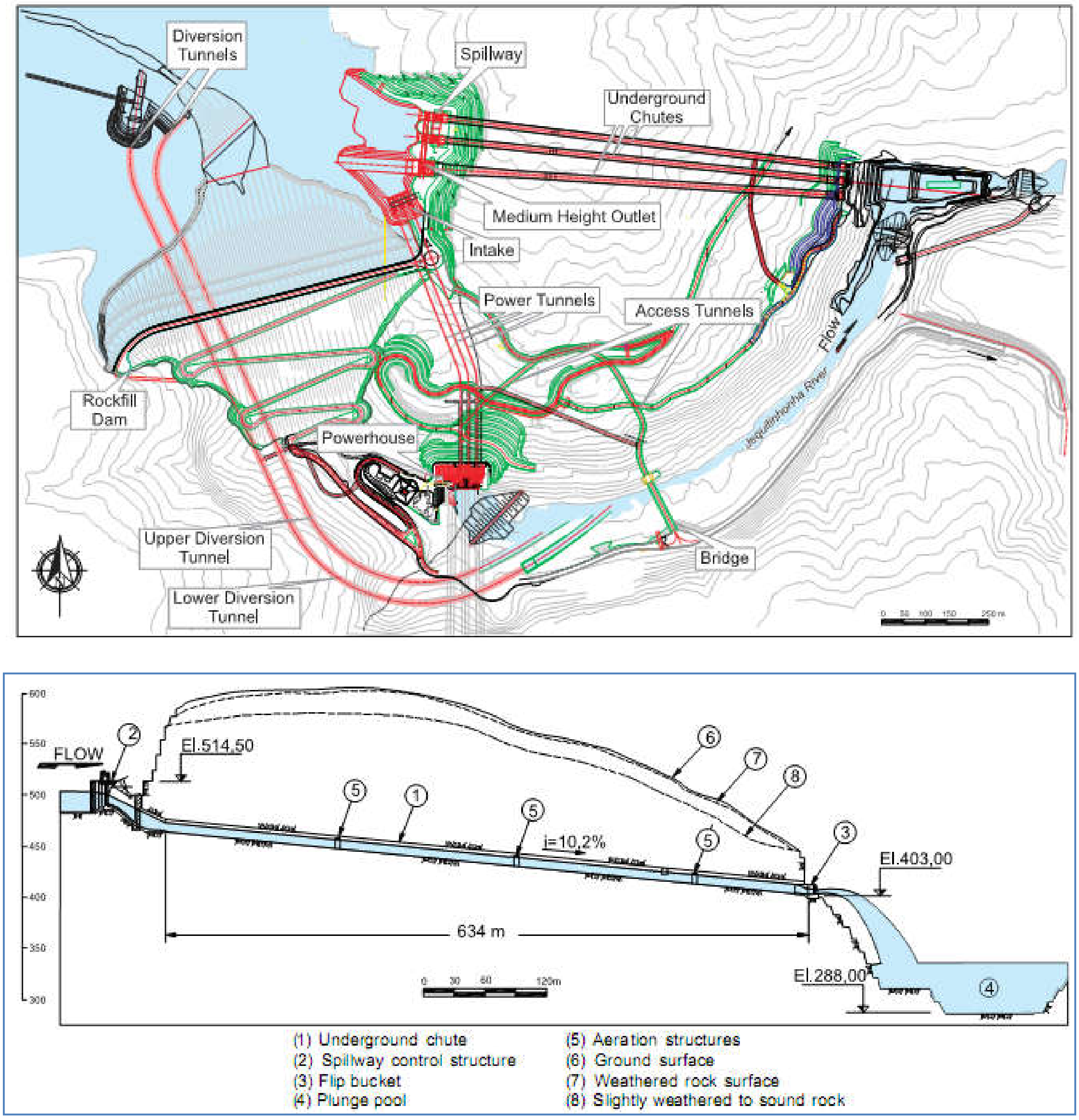
The low-level outlet tunnel in Fig 6.3 (a) (Moore, 1989) and the mid-level outlet in Fig. 7.3 (b) (Shakirov, 2013) are built to meet the requirement of starting reservoir impounding and creating a discharge which provided dam safety and environmental flow. In the Brazilian Irape Dam, shown in Fig 6.3 (c) and (d) (CBDB, 2009) there are two surface tunnel spillways and one intermediate outlet. A large part of energy is dissipated in a plunge pool by flow free fall between spillway bucket and the river level, as the dam height is over 200 m.



(a) Low-level outlet tunnel spillway in Mica Dam, Canada, with sudden expansion



(b) Mid-lever outlet tunnel spillways in Nurek Dam in Tajikstan



(c) Tunnel spillway in Irape dam in Brazil

Fig. 7.3 – Layouts for Types I and II tunnels

Layout of Type III tunnel spillway is most widely applied in high dams in the world since it was first adopted in the Hoover Dam in the 1930's; see Fig. 7.4. A short high level tunnel, less than 100 m long, is built and connected to an ogee shaped section, followed by an inclined tunnel and then to an inversed curve tunnel and finally to a small gradient low level tunnel. As the height between high and low level tunnels can be as large as 80 m to 100 m and the cross section area of tunnel can be of the order of 150 m^2, this type of tunnel spillway can be applied in dams with heights over 200 m and for discharge capacities over 3 000 m^3/s. The typical applications are the tunnel spillways in Hoover Dam (Falvey, 1990) and Glen Canyon Dam, (Burgi, P.H. and Eckley, M.S., 1988) in the United States and Ertan Dam in China (Gao, J., 2014).

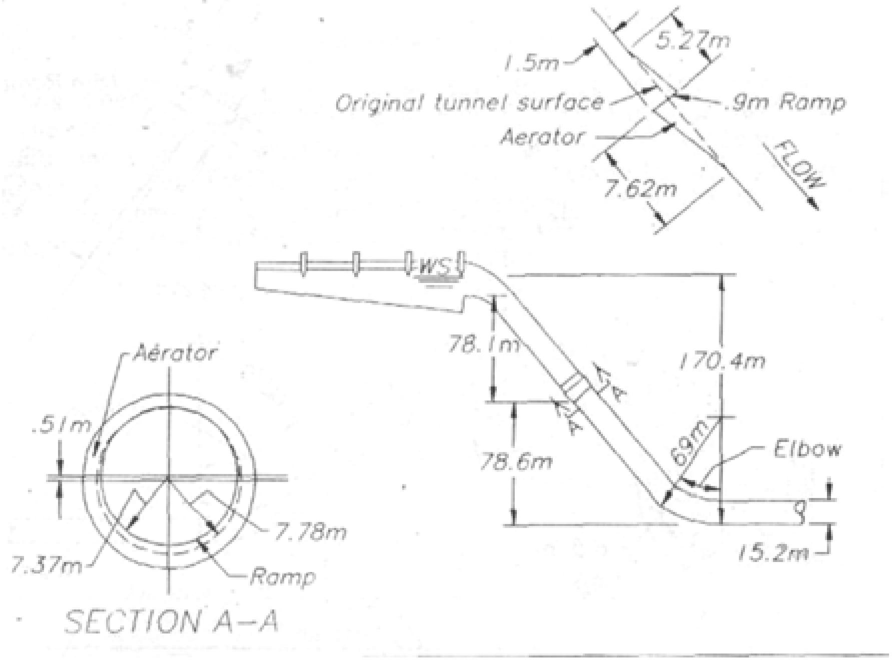


Fig. 7.4 – Type III spillway layout at Hoover Dam, United States

Large diversion tunnels are often necessary when a dam is built in narrow valleys where large diversion discharge capacity is required. Cost considerations may lead to build a high-level and an inclined tunnel to connect with the original diversion tunnel forming a permanent spillway tunnel. This arrangement is the same type as a Type III layout. It has been used in Mica Dam high-level tunnel spillway, Liujiaxia Dam low-level outlet tunnel spillway and in many other large projects.

As the cavitation risk increases in Type III tunnels when the dam height is over 250 m to 300 m, the Type IV tunnel spillway has been devised. The concept of this layout is to shorten the length of the tunnel with high speed flow by building a long pressurized high-level tunnel and moving the control structure of gate chamber as further downstream as economically possible. The alignment of the pressurised tunnel can be changed both in vertical and horizontal directions as convenient for the design.

This kind of tunnel spillway has been applied recently in several Chinese large projects, as for example, in the four large tunnel spillways designed for the high arch Xiluodu Dam. The dam height is 285 m and the discharge capacity of each tunnel is 4 000 m^3/s, (Fig. 7.5). The high-level pressurised tunnel is 550-m long with a diameter of 17 m, and there are two 90° vertical turnings before the service gate chamber. The length of last part of the straight pressurised tunnel has to be long enough to readjust the pressure until the pressure distribution is close to uniformity before the flow goes into the transition section. The length with high speed flow in the last part of tunnel is only about 20% to 30% of total length. The cross section in open flow tunnel part is 14×12.5m^2.

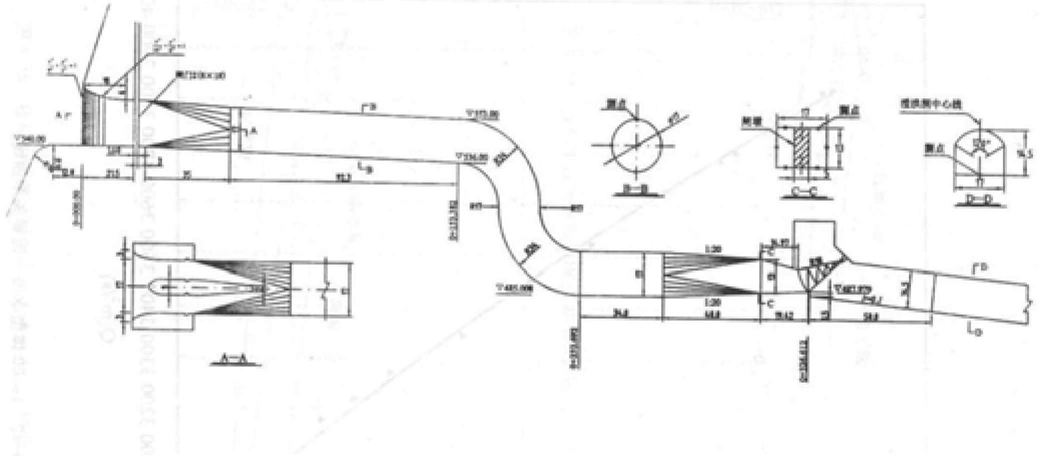


Fig. 7.5 - Layout of Type IV tunnel spillway in Xiluodu Arch Dam

The other way of transforming a diversion tunnel into a spillway tunnel is the Type V configuration. It can be designed as a vortex shaft tunnel spillway or an orifice tunnel spillway. The distinct feature of this type of tunnel spillway is to dissipate energy inside the tunnel. The vortex shaft tunnel can easily change flow direction by vortex action and shorten the length of the connecting tunnel.

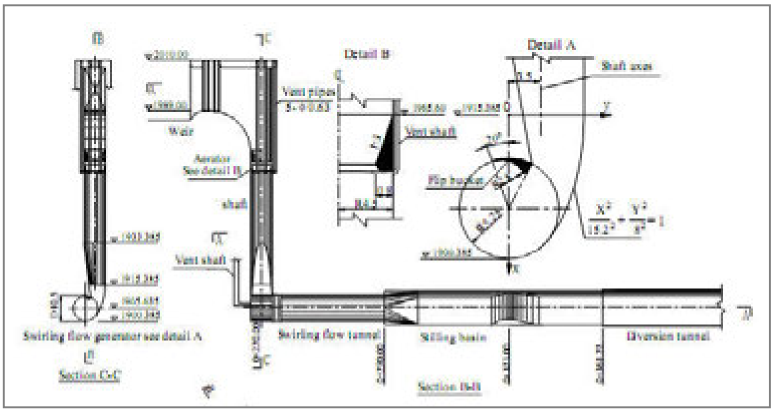
The hydraulic study on large scale of vortex shaft spillway was started in the former Soviet Union, then in Russia and followed by China, Swizerland and some other countries. Shapai Dam vertical vortex shaft tunnel spillway was the first application in China with a working head about 110 m and discharge capacity of 250 m^3/s, which was build in 1990s. Gongboxia Dam horizontal vortex shaft spillway tunnel was the second application in China with the working head about 110 m, but the discharge capacity up to 1 000 m^3/s, which was operated in 2006 at the design condition, as shown on Fig. 7.6 (a) (Chen, W.X. *et al*. 2007).

Tehri Dam high dam in India, adopted four large scale vortex shaft tunnel spillways, two on each bank, Fig. 6.6 (b) (Sharma, R.K. *et al*. 2006) They all used previously built diversion tunnels. The discharge capacity for each tunnel is about 1 800 m^3/s to 1 900 m^3/s and total discharging head is over 200 m (Fink, 2012 and Shakirov, R. 2013)

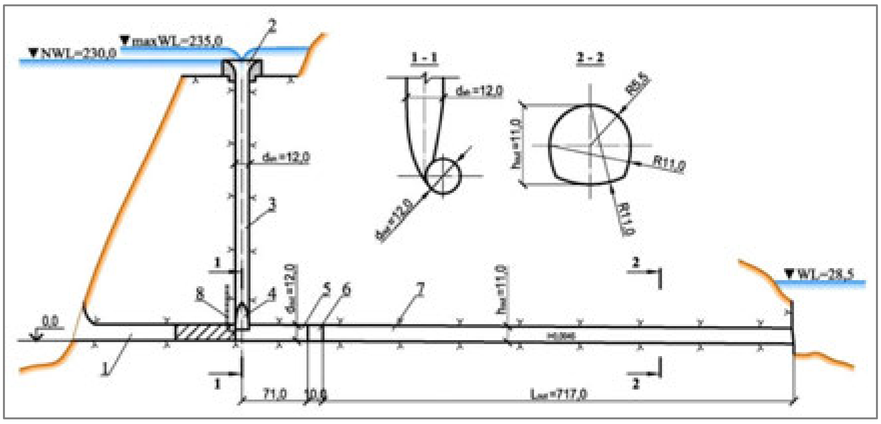
Fig. 7.6 (c) and (d) shows the vortex vertical and inclined shaft spillways designed for Rogun Dam in Tajikistan, with surface and bottom intakes (Shakirov, R, 2013). These vortex shaft spillways with surface and bottom intakes have a total discharge capacity 3 800 m^3/s (2 000+1 800 m^3/s). The vortex inclined shaft spillway of this project with high level intake has a discharge capacity 4 040 m^3/s. This kind of design has an advantage of wide range of operational water head especially for the requirement of water release in the initial impounding of the reservoir.

In vortex spillways about 80% of flow energy can be dissipated inside the outlet tunnel without significant dynamic effect to tunnel lining (Galant, M.A, *et al*. 1995 and Riquois, M. *et al*. 1967). Water from the tunnel into the river arrives practically with natural river velocities.

Xiaolangdi Dam orifice tunnel spillway was also built by reconstruction of diversion tunnel and energy was dissipated by three orifices, as seen Fig. 7.6 (d). At Mica Dam low outlet tunnel spillway takes such a kind of energy dissipation by sudden expansion chamber, as shown Fig. 6.3 (a).

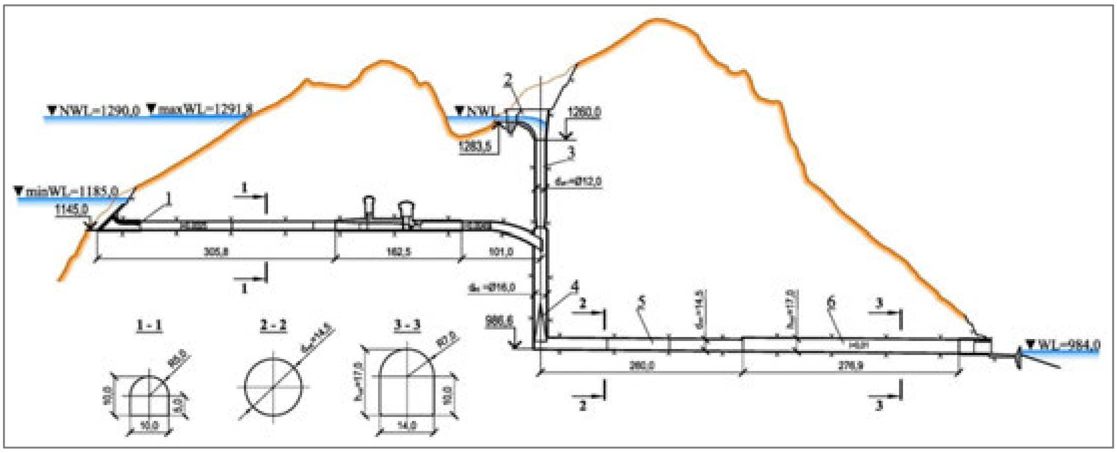


(a) Horizontal vortex tunnel spillway in Gongboxia Dam in China



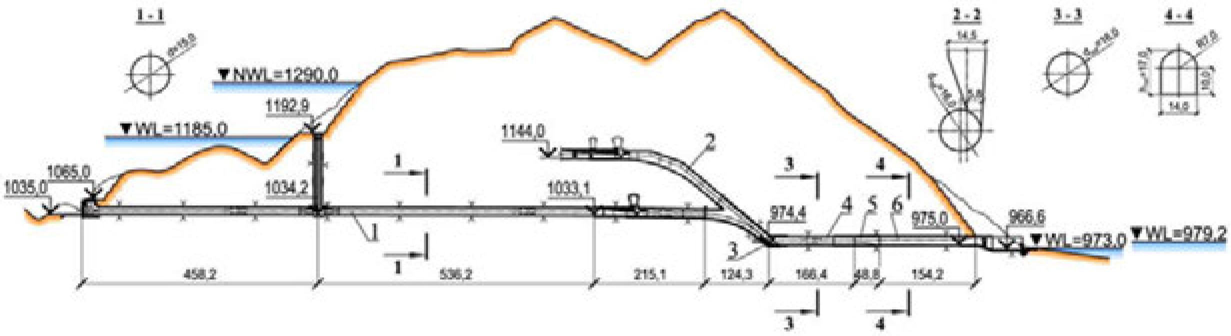
(b) Vertical shaft tunnel spillway in Tehri Dam, India

1 – Diversion tunnel; 2 – Surface intake; 3 – Vertical shaft; 4 – Flow swirling unit;
5 – Circle-shaped outlet tunnel section; 6 – Transition section;
7 – Horseshoe-shaped outlet tunnel section; 8 – Deaerator



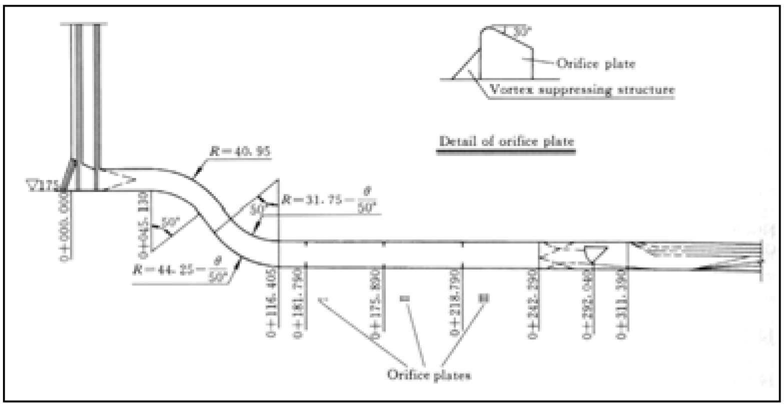
(c) Vortex shaft spillway with surface and bottom intakes, Rogun Dam, Tajikistan

1 – Bottom intake; 2 – Surface intake; 3 – Vertical shaft; 4 – Flow swirling unit;
5 – Circle-shaped outlet tunnel section; 6 – Channel-shaped outlet tunnel section



(d) Vortex inclined shaft spillway of Rogun Dam with low and high level intakes

1 – Low level tunnel DT-3; 2 – Upper level tunnel; 3 – Vortex device; 4 – Round section tunnel;
5 – Transition section; 6 – Outlet tunnel



(e) Orifice tunnel spillway in Xiaolangdi Dam, China

Fig. 7.6 - Type V tunnel spillway cases

Morning-glory shaft spillway has seldom been used. It has a circular inlet connecting to a shaft and low-level horizontal tunnel. The circular inlet can be divided by piers and controlled by gates. The working head and discharge capacity is usually small and is not fit to be used in large dams. This arrangement has not been used recently.

7.2.2. Single structure vs. different functional components

Different types of tunnel spillway are arranged for the different purposes and requirements from the construction to operation stages, especially for a high dam. High-level tunnels (Type II to Type IV) usually take most of the operation tasks of flood control since they have large discharge capacities. Low-level outlet tunnels (Type I) take primarily the tasks of discharging both for flood control and water supply during reservoir impounding, or lowering the reservoir water level for dam maintenance and also, sometimes, for emergency operation.

Some mid-level or temporary outlet tunnel spillway is sometimes necessary for high dam construction, especially in the case that the dam will is built by stages. A typical example is and Rogun dam. In this case the mid-level outlet tunnel spillway is connected to the permanent shaft spillway, as shown in Fig. 7.6 (c).

7.3. CONTROL STRUCTURE

7.3.1. Surface spillways

For high-level tunnel spillway (Type II to Type IV), the gate chamber is arranged just downstream of the inlet. Two gates are usually adopted: a flat gate for maintenance and a radial gate for the service operation. However provision of three gates is a more reliable and recommended solution: the first – a flat gate or stop-log (for low head) for maintenance; the second – the emergency gate, and the third – a radial gate for operation service. The working head in this design should be about 40 m to 50 m. There is an open flow after the service gate and throughout the tunnel. The service gate can also be a flat-type gate when the working head is smaller.

The gate chamber is divided into two sections to reduce the load acting on each gate when the working head is high or discharge capacity is large. This arrangement has been applied in the tunnel spillways of the Hoover Dam and of Xiluodu Dam as the discharge capacities for these two single tunnels are 5 000 m^3/s and 4 000 m^3/s respectively.

7.3.2. Bottom outlet spillway

The service gate in bottom or low-level outlet tunnel spillway is usually a radial type as the working head can be larger than 60 m to 80 m, reaching even 100 m to 120 m, and the discharge capacity is around several hundred or a thousand cubic meters per second. A slide gate can be used when the working head is larger and the discharge is smaller. For example, there are three slide gates built in the concrete plug in the Mica Dam low-level outlet tunnel spillway, as shown in Fig. 7.3 (a).

7.4. CONVEYANCE STRUCTURE

7.4.1. Chute channels

The longitudinal layout of a chute channel can be designed in any type of tunnel spillways depicted in Fig. 7.1. The gradient of bottom floor is usually less than 10% which is mainly controlled by the transportation capability during the construction. It has been increased to about 12% to 13% as the large and heavy trucks become available. The gradient of tunnel chute in Irape Dam (Fig. 7.3 (c) is 10.2% and the average length of the three tunnels is 634 m. A large gradient of the tunnel can reduce the length of the chute channel.

The cross section of chute channel can either be a circle, a D-shape or horse-shoe shape. The circle type was used for the tunnel spillways of Hoover Dam, Glen Canyon Dam and some other dams. But the D-shape is more convenient for the excavation and transportation during the construction. The concrete lining has to be designed to withstand a high speed flow. Lining work in D-shape tunnel is much simpler to be built than that in a circular shaped one.

High speed flow in chute channels in Type I to Type III tunnels will necessarily happen. Therefore the proper handling of such high speed flow requires cavitation control and is a most important work in the hydraulic design of the tunnel lining.

7.4.2. Aeration facilities

There are some important instructive cases of cavitation damages in tunnel spillways as well as in surface chute spillways. The mitigation of cavitation by aeration was recognized after the cavitation damage in Hoover Dam tunnel, and many studies had been carried out since then. Fig. 7.7 shows damage in Glen Canyon Dam left tunnel spillway in 1983 (Burgi, P.H. and Eckley, M.S., 1988 and Falvey, H. 1990). The 11-m deep "big hole" was found by site investigation.

A group of standard aeration facilities was developed in the 1980s and further on by different countries, based on their particular design of tunnels and operation requirements (ICOLD, 1987), (see Fig. 7.8). The aeration facilities were widely applied worldwide and cavitation damage in high speed flow tunnels were greatly reduced, although the mechanism of cavitation mitigation is still under investigation.

In the standard aeration device, an abrupt recess is created in the tunnel floor and an air vent is provided sometimes combining with a small ramp where negative pressure develops. Air is forced to enter the low pressure zone and aerate the water flow. It is very important to keep the air conduits clear and to ensure the aeration is continuous and sufficient during the tunnel operation. Interval distance between two aerators should be about 120 m to 150 m. Several aerators along the bottom floor of a tunnel may be necessary in a long tunnel spillway.

Fig. 7.7 - Damage to Glen Canyon Dam left Spillway in 1983.

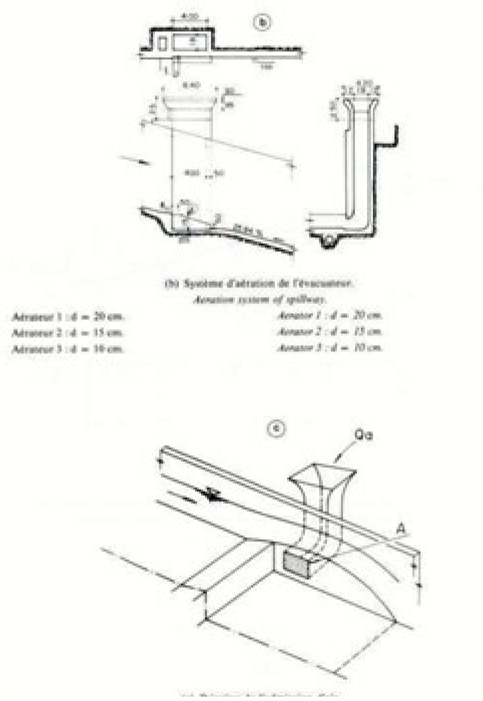


Fig. 7.8 - Typical design of an aerator

A special type of aerator was developed for cases where it is difficult to force air to enter the low pressure zone in a small gradient floor of a tunnel or where the tunnel cross section has a circular shape. The aerator built along the wetted perimeter is effectively to provide air entrainment for this special case. A half circular aerator has been applied in the tunnel spillways of Hoover Dam, Yellowtail Dam and Glen Canyon Dam. A circular aerator can also be applied in shaft tunnel, as for example in Gongboxia Dam shaft tunnel where the physical model study showed that the pressure at the lower part of shaft is quite low. The length of the low pressure zone measured in the prototype, downstream of the circular aerator device was longer than that measured in the laboratory.

An intermediate aeration tunnel or shaft may be necessary if a tunnel spillway is too long.

7.5. TERMINAL STRUCTURE (ENERGY DISSIPATION FEATURES)

7.5.1. Stilling basin

A stilling basin energy dissipater can be applied in a low head tunnel spillway scheme if geological conditions are not good - weak rock or soft ground – to allow the use of a flip bucket dissipater. Presence of high velocities and pulsating loads at energy dissipation in stilling basins requires special attention to their structural strength. A good design of stilling basin is usually based on the physical model study.

Experience in the operation of hydraulic facilities gives many examples of serious damages of the stilling basins (Malpasso in Mexico, Tarbella in Pakistan). The most common damage is characterized by the upward dislodgement of entire bottom slabs, which can be a consequence of the emergence of full hydrodynamic pressure under the slabs or high seepage uplift.

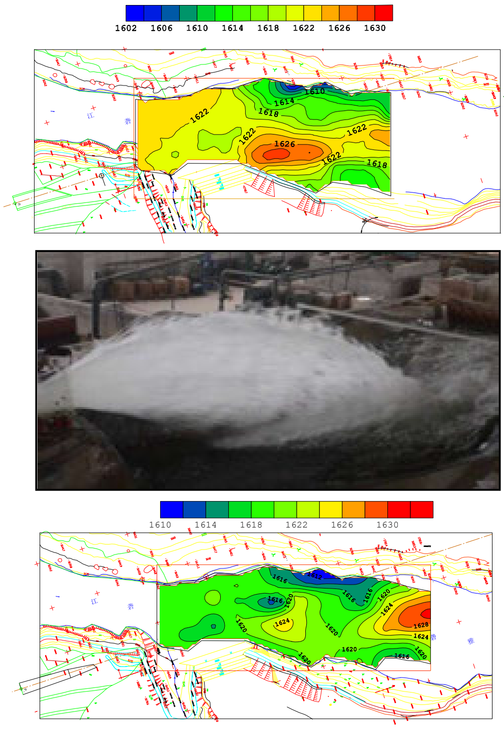
7.5.2. Flip bucket and plunge pool

A flip bucket dissipater is more often used in a high head tunnel spillway scheme if geological conditions are good, that is, are constituted by good solid rocks. Jet flow with large erosive power can be sent away from the structures, further downstream in the river channel.

As discussed in Chapter 3 of this bulletin, a plunge pool may be necessary, and in some situations it can be built by pre-excavation. The discharging heads in the tunnels of Fig. 7.3 (b) and Fig. 7.3 (c) are all over 200 m and the working head between the flip buckets and the water surface downstream are also over 100 m; therefore, a huge flow power must be dissipated in a large water body.

Normally, in plan view, there is a sharp angle between tunnel spillway and river channel, and the bucket design may involve considerable attempts. The bucket can be designed with different shapes and distortions including oblique and slit bucket. The aim of bucket design is to direct a jet flow into the river channel in a proper manner, achieving high efficiency of energy dissipation, reducing the depth of river bed scour and effects on the river banks. A good design of a bucket is usually combined with physical model study. A slit bucket is very convenient to be applied in a narrow valley and to put the jet flow in longitudinal distribution along the river channel (Guo, J., 2014), see Fig 7.9.

The depth of a plunge pool can be so important that it may threaten to reach the dam toe and in narrow sites cause stability loss of the valley abutment slopes. As an example of such problems, at Kariba dam, on the Zambese River in Africa, after five years of flood discharges reaching 8 500 to 9 000 m^3/s through six bottom spillways, the scour pit reached a depth 45-50 m. (Riquois, M., 1967).



(a) Scour by oblique bucket

(b) Flow pattern of slit bucket

(c) Scour by slit bucket

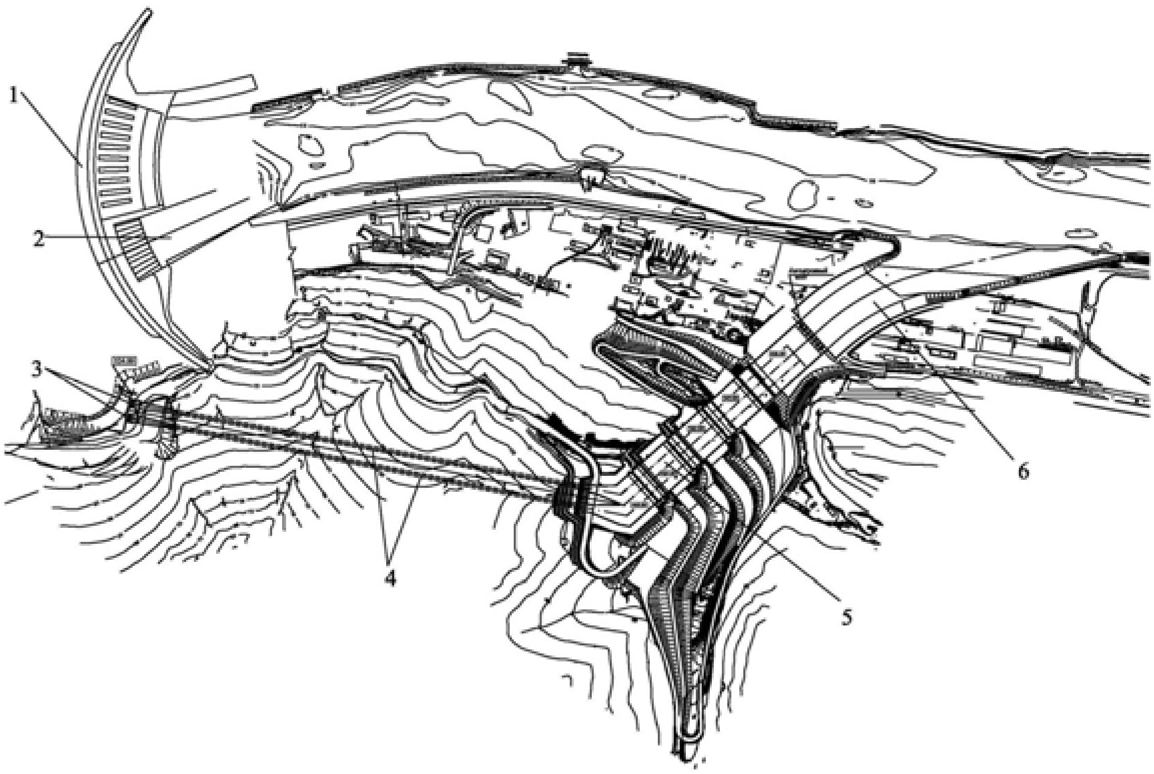
Fig. 7.9 - Comparison of energy dissipation between two types of flip bucket

7.5.3. Energy dissipated inside tunnel

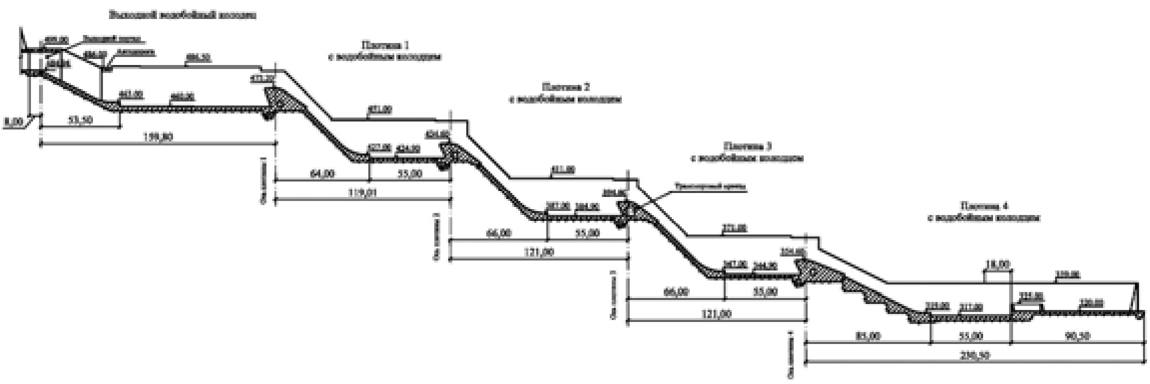
The flow energy can also be dissipated inside the tunnel for shaft tunnel spillway or orifice tunnel spillway, such as in Xiaolangdi Dam shown in Fig. 7.6 (e). In vortex shaft spillway tunnels more than 70% to 80% of energy can be dissipated inside the tunnel and the velocity in the last horizontal reach of the tunnel can be reduced to less than 20 m/s.

The orifice dissipater is built inside the tunnel by creating a sudden contraction and then an expansion through a chamber or orifice. This was the solution used for Mica Dam low-level outlet tunnel to dissipate energy and it achieved a good operational result. At Xiaolangdi Dam the tunnel spillways were built by reconstructing diversion tunnels with a diameter of 14.5 m each, and three orifices in each pressurised tunnel were installed with an interval of 3D and orifice ratios of 0.690, 0.724 and 0.724 respectively. More than 40% of energy is dissipated by the three orifices.

Another example of an energy dissipation arrangement inside the spillway tunnel is the case of the Sayano – Shushenskaya Power Project in Russia as depicted in Fig. 7.10. This rather unusual configuration dissipates the energy of the auxiliary 240-m head spillway in a sequence of 5 underground stilling basins in a stepped chute following the leveled spillway tunnels.



1 – arch-gravity dam; 2 – service spillway; 3 – intake structure of reserve spillway;
4 - tunnels; 5 – stepped chute; 6 – tail race channel



Longitudinal profile of stepped chute

Fig. 7.10 **-** The reserve spillway of Sayano – Shushenskaya HPP

7.6. OPERATIONAL ISSUES

7.6.1. Gate operation

The safety of tunnel spillways is mainly affected by abnormalities in gate operation, ventilation, aeration and energy dissipation during the flood discharge.

Radial gates are usually used for flow control both in high-head and low outlet tunnel spillways to allow flexibility by partially opening to control discharge. The operation of gates must follow strict regulation orders. Any improper or faulty operation may cause serious accidents and damage the gate and/or the tunnel. Small openings, for instance 5% to 10%, should be avoided as vibration may appear at this range of openings according to operation experiences and field observation.

Flat gate is commonly designed mainly for low-level tunnel spillway. Partial openings are not usually allowed as the working head is high. For flat gates it may be necessary to increase the working load on the gate by stages if the head is large.

The electrical and mechanical control system must have the same degree of safety reliability of the gate as it was demonstrate in the emergency operation experience of Zipingpu Dam low outlet tunnel spillway in "Wenchuan Earthquake" in 2008. This is necessary to ensure that the gate system is able work in an emergency case.

Ventilation behind gate is also important. The air flow speed must be controlled by selecting proper cross section of air vent because vibration and noise can be induced by a high air flow speed.

7.6.2. Inspection and maintenance

Regular inspections and maintenance of gates, operating equipment, the tunnels proper and aerators must be implemented after each flood season. The importance of this work has been addressed in the General Report of ICOLD Question 79 (Cassidy, J., 2000) as a vital part of assuring spillways operating reliably and safely. Minor and severe damages observed during the inspections must be recorded and repaired.

The inspection by the operator after the initial operation of the tunnel spillway is of utmost importance. The behaviour and performance of all structures have to be checked. These should also be complemented by the analysis of recorded monitoring results.

An underwater survey of the energy dissipation zone may be necessary in the case of a large flood discharge or a long duration of flood discharge operation. This survey will seek to determine the location and shape of the scour hole, and will be the basis of an analysis intending to define the need of alterations or provision of additional features in the existing energy dissipation scheme.

7.7. RISK ANALYSIS AND CONSIDERATIONS

7.7.1. Cavitation risk control

Fig. 6.10 gives the dimensionless geometrical characteristics of Type III and Type IV large scale tunnel spillways (Guo, J. *et al*. 2006). The dam heights listed are all over 200 m. The three indexes, which are, the relative height (h/H), the relative length (x/L) and the flow velocity, are taken into account to analyse cavitation risks of different kinds of tunnel spillway layouts.

Cavitation damages in Type III tunnels referred to in Fig. 7.10 happen in a horizontal reach of the tunnel just downstream of an inverse curve, where 40% to 80% of the water heads prevail. In about 70% to 80% of the length the flow velocity is over 40 m/s. This scheme presents a great risk of cavitation damage. In contrast, the length with high velocity flow in the layout of a Type IV tunnel corresponds to about 20% to 30% of the total tunnel length; therefore, the risk of cavitation damage in these tunnels will be greatly reduced. This kind of arrangement has been applied in several large dams projects in China with dam heights around 300 m and discharge capacities of individual tunnel over 3 000 m^3/s.

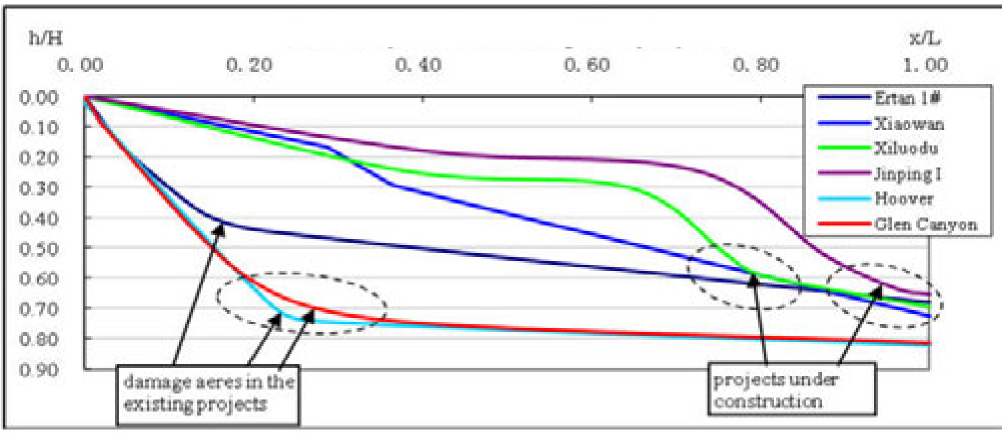


Fig. 7.10 – Risk analyses of tunnel spillway layouts

7.7.2. Sediment abrasive action in tunnel linings

Sediment abrasive action of the concrete lining surface is observed when there are tunnel intakes located near the reservoir bottom. Usually it happens for river diversion tunnels, in the beginning of construction period when the reservoir is too small to decrease river velocity and the flow transportation capacity, so that large quantities of solid river runoff sediment, with fractions up to 100 mm, are discharged by the tunnel during river diversion and construction periods. Such situation can happen again at the end of a reservoir drawdown period when even larger size particles may reach the spillway intake.

Damages at the flip bucket of the spillway of Grand Coulee dam is an example of significant damages of the concrete due to the abrasive action (Keener, K. B. 1944). Protection against abrasive action can also be provided by the use of special kinds of concrete with hardening the invert part or a steel lining.

7.7.3. Dynamic loads

Dynamic loads in spillway tunnels may be the result, among other causes, of constant change in the modes of operation of the tunnel, the presence of transient processes, or the formation of hydraulic jump in the tunnel in the absence of air supply behind the gate chamber piers leading to formation of vacuum zones, cavitation and separation of the metal liner (Ilyushin V.F., 2002)

Significant dynamic loads were the cause of a major accident at auxiliary spillway tunnel of San Esteban dam (Del Campo A., 1967). The tunnel was designed to operate in free-flow conditions, but at a high downstream water level period, it was flooded. As result a hydraulic jump was formed within the tunnel, which led to the damages of the lining in the area of a rock mass fault and the falling rock caused the obstruction of the spillway tunnel.

7.7.4. Emergency operation

Safety emergency operation requires that one or two low outlet tunnel conduits to empty tunnel spillway are provided, especially for earth and rockfill dams, in case of earthquake or a landslide happened. The case of Zipingpu Dam in China, which was subject to an intense earthquake (Case 2, below), is very instructive of the very important role on the dam safety management by releasing water to control reservoir water level in emergency conditions. It also gives a new consideration or new mission for reutilizing diversion tunnel.

7.8. CASES

7.8.1. Flood release by shaft tunnel spillway in Tehri Dam (India)

The water level in the Tehri Dam Reservoir rose to El. 882.0 m in September 2009; the right-bank shaft spillways automatically started passing a discharge of 480 m^3/s through each of them with maximum discharge capacity of 1850 m^3/s (with MWL at El. 835.0 m). Photo 1 shows the operation of the shaft spillways releasing the water-air mix from the deaeration chamber with a jet height exceeding 50 m demonstrating the effectiveness of operation of the deaeration facilities of the shaft spillway (Sharma, R.K. *et al*, 2006)

Photo 1 Deaeration from Tehri shaft spillway operation in 2009

7.8.2. Emergency operations of low outlet tunnel spillways in Zipingpu Dam after the Wenchuan Earthquake (China).

Zipingpu Dam is located on the upper reach of Minjiang River in China, 9 km upstream of Dujiangyan City and 60 km northwest to Chengdu City. It consists of a 156 m high CFRD, one chute spillway, power generation system, one silt sluice tunnel and two low outlet tunnel spillways. The two low outlet tunnel spillways are built by reconfiguring the diversion tunnels. The main purposes of this project are irrigation and municipal water supply, as well as power generation, flood control, and environment protection. The Project was completed in 2006.

A strong earthquake with a magnitude 8.0 happened on May 12 in 2008 (so called "Wenchuan Earthquake"). The Zipingpu CFRD is only 17 km away from epicentre of the earthquake (ICOLD Experts, 2009)

The dam had minor damages but power supply was stopped and water supply was terminated. Resuming water and power supply and controlling the reservoir water level were the most important tasks in the emergency management. The first unit was immediately put into operation just 7 minutes after the strong shock and another two units operated right afterwards. The total discharge capacity was 100 m^3/s which was supplied to the Dujiangyan City and Chengdu City downstream again where there is a population of 20 million people. The silt sluice tunnel operated 24 hours after the earthquake and the discharge rate was increased to 280 m^3/s, as shown in Photo 2 (a). One low outlet tunnel spillway opened 27 hours after the earthquake. The total discharge flow was up to 850 m^3/s that was close to the inflow. The power was resumed 128 hours after the earthquake. All tunnel gates were successfully opened 190 hours after the earthquake, therefore, the reservoir water level was effectively controlled during May 14 to 20 and the risk of dam was reduced greatly. One purpose of such operation is to ensure the dam safety by controlling the reservoir water level, inspecting damages of dam structure and reparing mechanical and electrical equipment; the other purpose is to provide a life rescue waterway for reservoir area, as shown in Photo 2 (b).

An international group of dam experts from ICOLD visited Zipingpu CFRD in April of 2009. Their observations concluded that emergency actions were taken immediately after the earthquake and the established emergency management works on dam safety in case of severe catastrophe, have been correctly applied in the "Wenchuan Earthquake" case (ICOLD Experts, 2009).

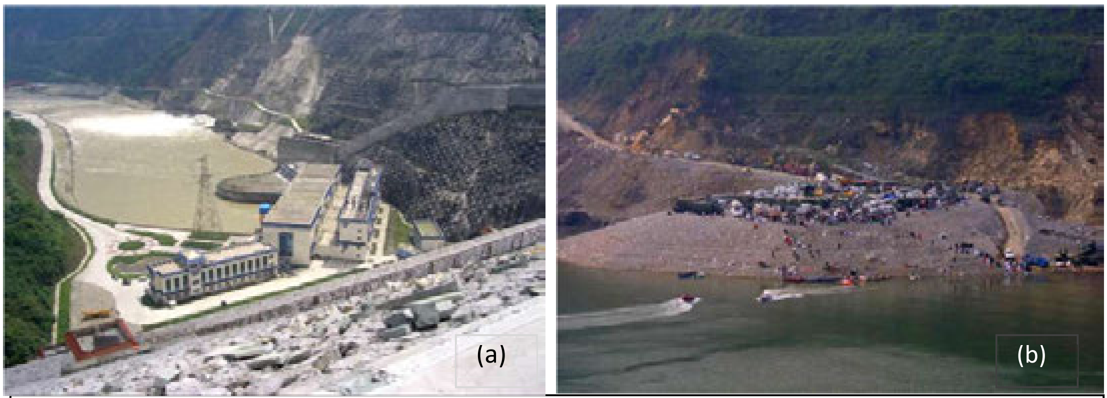


Photo 2 Emergency actions by Zipingpu CFRD after "Wenchuan Earthquake".
(a) silt sluice tunnel operated 24 hours after shock; (b) temparory jetty on reservoir shoreline to provide a life rescue waterway for reservoir area

Table 7-1 Characteristics of typical large scale tunnel spillways in the world (ordered by dam height)

Name	Country	Dam type	Dam height, H (m)	Type of tunnel	tunnel size (with × height)	Q_{max} (m^3/s)	Type of aeration	Completion*	Operation experience
Rogun	Tajikistan	ER	335	V(a)	D14×17	4040	aerator in vortex	u.d.	
Rogun	Tajikistan	ER	335	V	D14×17	1800	aerator in vortex	u.d.	
Rogun	Tajikistan	ER	335	V	D14×17	2000	aerator in vortex	u.d.	
Jinping I	China	AV	305	IV	14×12	1×3651	offset, ramp	u.c.	
Nurek	Tajikistan	ER	300	III	11×10	1×2000	Aerator ds radial gate	1980	
Nurek	Tajikistan	ER	300	II	10×10.5	1×2020	Aerator ds radial gate	1980	
Xiaowan	China	AV	292	IV	13×13.5	1×3811	2-steps offset	2011	
Xiluodu	China	AV	278	IV	14×12.5	4×3860	offset, ramp	u.c.	
Tehri	India	ER	261.5	IV	D=8.5	1×1100	-	2008	2010
Tehri	India	ER	260.5	V	4 horse-shoe, D=11	2×1850, 2×1800	Deaerator at the junction between vertical shaft and horizontal tunnel	2008	2010
Mica	Canada	ER	244	V	-	1×850	concrete plug	1977	good operation
Mica	Canada	ER	244	III	-	-	offset	1977	good operation
Ertan	China	AV	240	III	13×13.5	2×3700	offset, ramp, 3D later	1999	damaged in 2001
Sayano-Shushenshkaya	Russia	AV	240	II	2 - 10×12	2×1900	In stepped chute	2011	good operation
Chirkeyskaya	Russia	AV	232	III	11.2×12.6	2900	-	1978	good operation

Name	Country	Dam type	Dam height, H (m)	Type of tunnel	tunnel size (with × height)	Q_{max} (m^3/s)	Type of aeration	Completion*	Operation experience
Hoover	United States	PG	221	III	D15.56	2×5500	circular aerator	1936	damaged in 1941
Glen Canyon	United States	AV	216	III	D12.5	2×3900	circular aerator	1966	damaged in 1983
Irape	Brazil	TE	208	II	10×11.4	2×2000	offset	2005	good operation
Irape	Brazil	TE	208	II	12×11.4	1×2000	offset	2005	good operation
Longyangxia	China	PG	178	III	5×7	1×1340	offset at gate	1989	damaged in 1989
Charvakskaya	Uzbekistan	ER	168	IV, V	D=9.0 D=11.0	1100 1200	Downstream steel liner	1976	
Xiaolangdi	China	ER	160	II	10×12, 8×9, 8×9.5	1×2680, 1×1973 1×1796	offset	2000	good
Xiaolangdi	China	ER	160	V	D14.5	1×1727, 2×1549	offset at gate	2000	good
Yellowtail	United States	AV	160	III	2-D15.56	1×2600	circle aerator	1968	damaged in 1967
Zipingpu	China	CFRD	156	III	horse-shoe, D=10.7	1×1672	U-shape sidewall ramp	2006	good aeration, observed
Liujiaxia	China	PG	147	III	8×12.9	1×2105	aerator	1974	damaged in 1972
Cousar	Iran	PG	140	II	2 - 10×10	2×1900		2006	

Name	Country	Dam type	Dam height, H (m)	Type of tunnel	tunnel size (with × height)	Q_{max} (m^3/s)	Type of aeration	Completion*	Operation experience
Aldeadavila	Spain		139.5	III	D=10.4	2800	-	1962	
Gongboxia	China	CFRD	132.2	V	11×14	1×1090	circular aerator on lower part of shaft	2006	good operation
Shapai	China	RCC VA	132	V	4×5.5	1×242	air vent downstream shaft	2002	good operation in 2008

*: u.c. = under construction; u.d. = under design;
**:n.c. = not clear

Table 7-2 - Classification of Tunnel, Shaft and Vortex Spillways

Type	Level	Alignment	Inlet/Outlet Criteria	Hydraulic Condition	Control Structure	Control gate	Air Supply/Aeration/ Air Vent	Energy Dissipation
I(a)	Low	Straight	Low level inlet with small gradient straight tunnel. Free outlet without submergence.	Free gravity flow with very high velocity.	Upstream control at inlet	At upstream radial gate for high discharge. For small discharge it can be vertical slide gate	Must have sufficient air supply over the free surface. Due to very high velocity specific aeration groove/ arrangement has to be made for bottom surface.	Energy dissipation outside tunnel
I(b)	Low	After pressurised low reach, the rest is straight	Low level inlet with small gradient. Inlet for pressurised flow, outlet free flow without submergence.	Upstream flow pressurised by sudden expansion.	By sudden expansion	Upstream	Must have sufficient air supply over the free surface flow.	By sudden expansion
II	High	Straight	High level inlet with small gradient straight tunnel. Free outlet without submergence.	Free gravity flow with high velocity	Upstream control at inlet	At upstream radial gate for high discharge. For small discharge it can be vertical slide gate	Must have sufficient air supply over the free surface. Due to very high velocity specific aeration groove/ arrangement has to be made for tunnel floor.	Energy dissipation outside tunnel
III	High and Low	Straight	Short high level tunnel connected by ogee shaped inclined tunnel, inverse curve tunnel and finally to a small gradient outlet tunnel	Free gravity flow with high velocity	Upstream control at inlet	At upstream radial gate for high discharge. For small discharge it can be vertical slide gate	Must have sufficient air supply over the free surface. Due to very high velocity specific aeration groove/ arrangement has to be made for tunnel floor.	Energy dissipation outside tunnel

Type	Level	Alignment	Inlet/Outlet Criteria	Hydraulic Condition	Control Structure	Control gate	Air Supply/Aeration/ Air Vent	Energy Dissipation
IV(a)	High	Alignment of pressurised tunnel can be changed both horizontally and vertically.	High level long pressurised tunnel and short length free flow downstream outlet tunnel	Long length of tunnel under pressurised flow. Short length of tunnel under free flow.	Control at the end of pressurised flow by gate	Radial gate at the end of pressurised tunnel, Releasing flow to free flow tunnel	Because of short length of free tunnel, it may not have much problem in air supply and also cavitation	Energy dissipation outside tunnel
V(a)	High and Low	Vortex shaft changing flow direction to the shorter length of connecting tunnel.	High level intake entry to vortex shaft tunnel and low level outlet by free flow.	Flow by generating vortex in vertical shaft and then continuing in horizontal tunnel and subsequent free flow with energy dissipation arrangement in free flow area.	Control in upstream of vertical vortex shaft tunnel.	Radial/ Vertical Slide gate at upstream of vertical vortex shaft.	Sufficient and proper designed arrangement for vent pipe in vertical and horizontal vortex shaft for removal of air/air-water mix.	Energy dissipation inside the tunnel and stilling basin
V(b)	High and Low	Vortex shaft changing flow direction to the shorter length of connecting tunnel	High level long pressurised tunnel and low level free flow downstream outlet tunnel	Pressurised flow up to control gate. Inclined shaft tangentially joins the tunnel at low level.	Control in upstream of inclined vortex shaft tunnel	Radial/ Vertical Slide gate at upstream of inclined vortex shaft	Sufficient and proper designed arrangement for vent pipe in vertical and horizontal vortex shaft for removal of air/air-water mix	Energy dissipation inside the tunnel and stilling basin

Type	Level	Alignment	Inlet/Outlet Criteria	Hydraulic Condition	Control Structure	Control gate	Air Supply/Aeration/ Air Vent	Energy Dissipation
V(c)	High and Low	Inlet at high level. Upstream pressurised flow with change of horizontal and vertical alignment. Downstream free flow straight tunnel.	High level intake entry with pressurised flow. Low level out let with free flow.	Pressurised flow till control gate. Downstream tunnel with free flow.	Control downstream of pressurised tunnel.	Radial gate at the end of pressurized tunnel, Releasing flow to free flow tunnel.	Aeration and free air supply is not a critical issue but should be there.	Energy dissipation by orifice or number of orifices inside the horizontal pressurised tunnel.

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8. SPECIAL PROBLEMS OF SPILLWAYS IN VERY COLD CLIMATE

8.1 INTRODUCTION

It is known that snow and ice can partly or fully block spillways and thus reduce the spillway and water conveyance capacity in cold regions. These problems, but also other challenges concerning regulation of gates and special loads and impacts from ice on structures, are discussed in this chapter.

8.2 REDUCED CAPACITY OF SPILLWAY AND WATER CONVEYANCE

Stable operation of hydraulic structures particularly spillways, water intakes and water conveyance structures must be ensured not only in winter when the ice cover is stable but also during formation and destruction of the ice cover.

It is convenient to separate reduced capacity of spillways caused by cold climate into the following points:

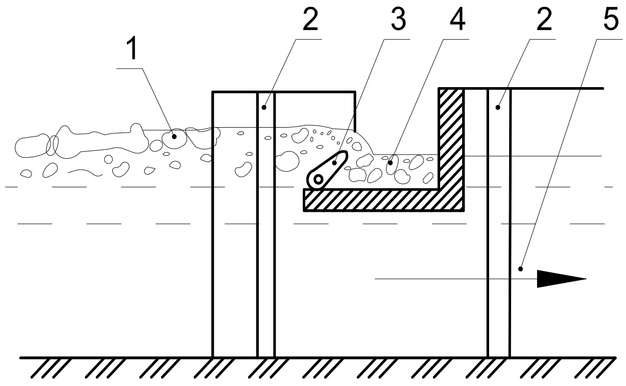
- frazil ice;
- aufeis blocking of the cross section;
- snow plug in the spillway;
- ice drift.

8.2.1 Frazil ice

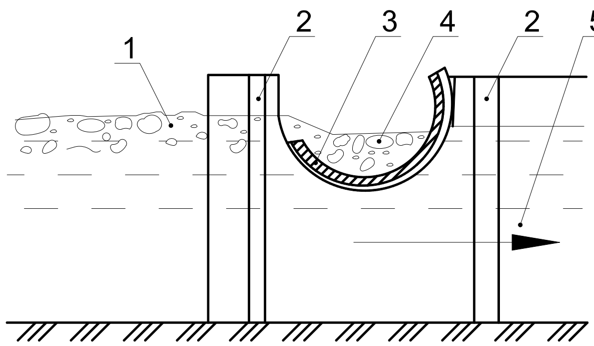
The period of formation of the ice cover is characterized by the drop of air temperature below 0˚C. During this period the surface layer of water is cooled to a temperature of about 0˚C which causes formation of such a phenomenon as "frazil ice" or "sludge" especially in the presence of precipitation in the form of wet snow. The frazil ice is water in transition to another solid state of aggregation i.e. ice, however, not yet having the physical properties of ice. It should be born in mind that the ice density is 917 kg/m^3 and the water density is 1000 kg/m^3 therefore the density of frazil ice is an average between these values. This fact explains a number of properties of frazil ice – viscous mass on the water surface resembling glue. When water is super-cooled flowing frazil ice will stick to structures such as booms, trash racks, baffle beams, slot structures, gate edges and stop logs. This will reduce the discharge capacity. The mass of the structures will also increase and make the operation of gates and hoist mechanisms difficult or even damaging them.

Frazil ice is particularly harmful for water intake structures of hydro power plants, pumping stations and hydropower equipment. Preventive measures to control such harmful actions can be taken by providing the following devices.

- Construction of ice intakes and ice passes (Fig. 8.1) diverting the ice laden flow downstream of the chute. The most efficient technology for the frazil ice pass is selected depending on the intensity of the frazil ice drift, water discharges and other specific conditions at a certain project. To reduce specific water discharges through ice passes to a minimum and to ensure a maximum entrainment of frazil ice the elevation of the overflow sill can be achieved by changing the inclination of a flap gate or the position of a swivel as shown in the figures.



a) Chute with flap gate



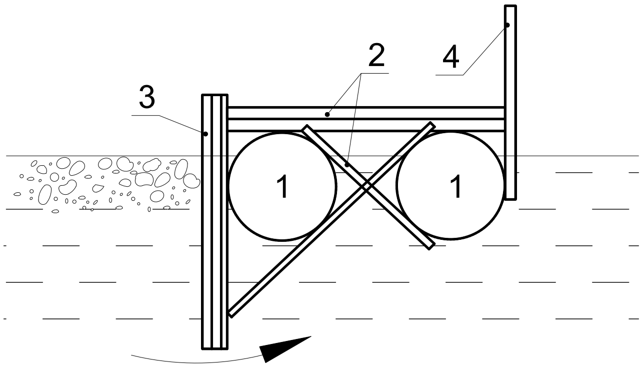
b) Swivel chute of A. Gostunsky

1 — floating frazil ice; 2— slot; 3— overflow sill with variable elevation through which frazil ice is entrained; 4 — flow with frazil ice in chute; 5— flow free from frazil ice.

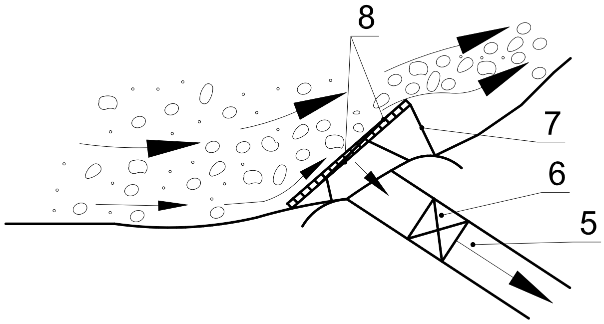
Figure 8.1 - Typical arrangement of ice and sludge passes

- Debris deflectors are structures deflecting surface flow streams and objects floating the water surface, away from the place being protected, such as a water intake (Fig. 8.2). The main element of debris deflectors is a deflector baffle or deflector shield. The depth of its submergence has to prevent diving frazil ice under its lower edge. The deflector plane is placed at an accurate angle to the flow so the deflector baffle does not restrict the flow but only deflects it. The adopted depth of the deflector baffle submersion under the water varies from 0.8 m to about 1.5 m, depending on its purpose.

  If the water intake structure is located near the transit portion of the river flow, it can be protected from frazil ice with the aid of a skimmer wall. The principle of its operation is similar to that of the deflector wall. To pass the frazil ice or other non-massive ice formations 1-2 gates of spillway openings are made as overflow ones (double-leaf gate). This is also called "skimmer walls"



a) Cross section



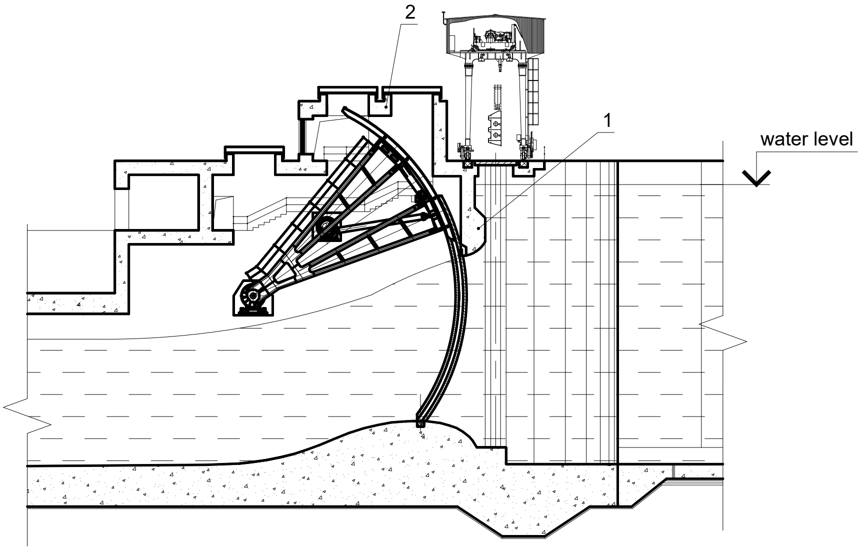
b) Arrangement for protection of river intake - plan

1 —floating elements (steel pipes portions, 600 mm in diameter); 2—bearing structures; 3— slot structure (for example, I-beam) with nested deflector shield; 4 —protection; ,5 — channel; 6 –water intake sluice; 7 — structures holding debris deflector; 8 — links of debris deflector.

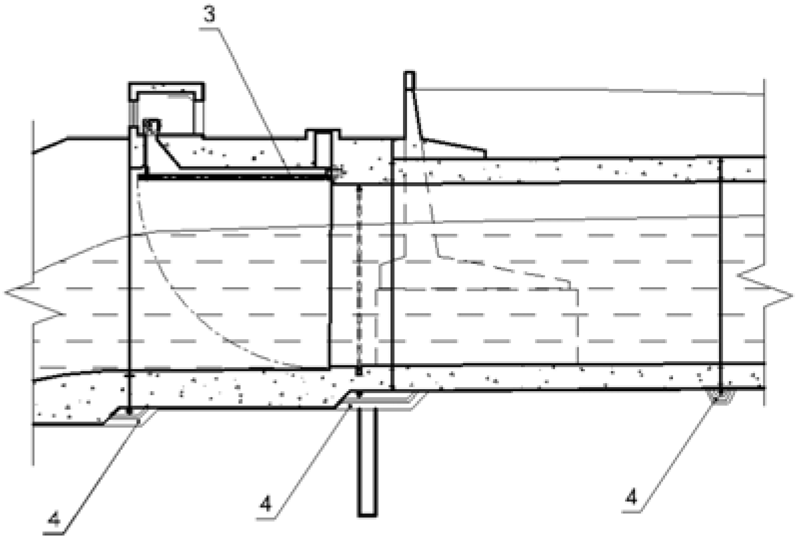
Fig. 8.2 - Debris deflector

- Arrangement of booms or baffle beams and walls (Fig. 8.3.a) designed to withstand the impact of frazil- and floating ice.

It is also known that high-flow bubbler systems are used to make a curtain that supress ice growth on upstream part of gates or locks (Tuthill, 2002).



a) Substructure above the gate and boom (bafflebeam);
1 – Boom installed in a proper way;
2 – Substructure providing >0°C temperature inside the structure



b) Design of thermal shield in the transit section
3 – Thermal shield protecting the structure against warm air coming from the D.S. side.
4 – Drainage behind the thermal shield (slab thickness exceeds the frost penetration thickness thus providing stability of the structure and uninterrupted operation of drainage).

Fig. 8.3 – Arrangement of booms and baffle beams

8.2.2 Aufeis blocking

Aufeis is a term used for ice formed by shallow flow of water which streams over an ice cover. Other terms as naled and icing are also used for the same phenomenon. Aufeis occurs under natural winter conditions.

The most common situation concerning aufeis in a spillway system is leakage through gated structures (Fig. 8.4). This can block the gate and reduce the capacity significantly.

Fig. 8.4 - Pont Arnaud: Gate leakage in winter creating aufeis (ice accumulation) and reducing significantly the spillway capacity (Hydro-Québec).

Another situation causing aufeis is through flow of cold air tunnel spillways. Leakages freeze in and reduce the open cross section and the capacity of the tunnel. Aufeis build-up with 3 m thickness has been observed in spillways tunnels. During the melting flood the water in the reservoir can rise to an unacceptable reservoir level caused by the reduced spillway capacity.

Description of the build-up and removal of aufeis is based on research by Schohl and Ettema (1986). The aufeis build-up depends on the time, surface area, heat flux, water discharge, slope and different water and ice properties.

The removal of aufeis can be separated into two processes: melting and mechanical break-up. It has been found that if water flows over the aufeis the melting rate depends on the water temperature. If no flowing water occurs, melting will depend on radiation, wind velocity and air temperature. A sudden break-up of the aufeis will result in an ice run, and is only possible for local deposits of aufeis. Sokolov (1973) found that a mechanical break-up never removed more than 30-35 % of the ice. According to Ashton (1986) aufeis melting is reported to last for several weeks.

Lia (1998) presented several solutions for reducing the problems with aufeis in spillway tunnels:

- Insulation. A non-structural wall in the outlet (and in some cases also in the inlet) prevents cold air from penetrating the tunnel or at least eliminating the through flow. In addition insulation of the tunnel walls can be installed.

- Grouting. In order to prevent water from leaking into the tunnel during cold periods, grouting can be used. For the same reason it is recommended to use controlled blasting close to the outlet, inlet and in the area where the spillway weir is to be constructed.

- Improved cross section. Aufeis in a spillway grows when water flows onto the whole tunnel bed and freezes to ice. If a narrow and deep ditch is constructed from the frost-free zone in the tunnel out to the tunnel outlet an ice cover will establish at the free water surface and protect the water flow beneath, as in a natural river, see figure 8.5.

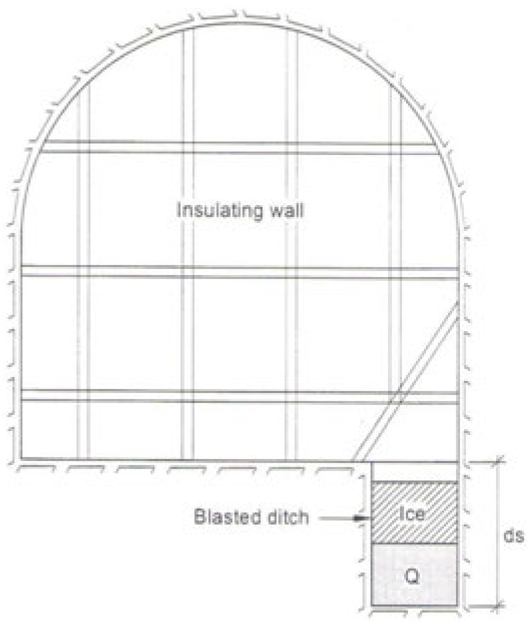


Fig. 8.5 - Cross section of the deep ditch with an established ice cover (Lia, 1998)

8.2.3 Snow

Snow is a material that changes behaviour and properties by time. Snow can block the inlet, tunnel/channel and outlet of the spillway, and may prevent the spillway from being inspected and maintained during winter time.

The average depth of snow cover is dependent on precipitation, temperature and snow drift. For local areas, the topography plays an important role for snow accumulation. The wind removes snow from one area and accumulates it in another. If the wind comes from one main direction (e.g. west) during the winter, the snow will be deposited in hills whose face is in the opposite direction (e.g. east). This is typical in high windy mountains. Although the wind direction changes in consecutive years, the main direction is very often the same. With this in mind, two or three years will be necessary to describe local patterns.

Wind transported snow change properties and become more compact when it accumulates in snowdrifts. After some hours a cohesive material develops. Drift snow accumulated in snowdrifts that can last one or several summers before they melt, named a multi-year snowdrifts. Such a snowdrift has blocked a tunnel outlet at a hydropower scheme in Norway for several years.

The snow will form a snow dam in the spillway that will reduce the capacity of the spillway or in the worst case, fully blocking the spillway.

If melt water is able to penetrate the cold snow, the water will seep through the snow dam. Research have been carried out to find the permeability of snow. The rain and melt water flow downwards in the snow, due to the gravity force, in the same way as in soil, and can be expressed by Darcy's law, where the permeability in the snow depends on the grain size of the snow crystals and the density, Sommerfeld (1989). The snow dam will eventually break by thermal processes, seepage, floating, overtopping, sliding or a combination of them.

Lia (1998) presented several solutions for reducing the problems with snow in the spillway:

- Snow fences. This is a convenient tool for manipulating the snow accumulation pattern. The fences do not stop the drifting snow, but they change the wind direction in such a way that the suspended snow accumulates behind the fences.

- Roofs. In Norway the spillway is typically a free overflow into a side channel. It is then efficient to reduce the amount of snow in the channel by building a roof over the channel. The roof will also reduce the compaction of the snow. Such roofs have been constructed at some side channels, see Figure 8.6. Inspections have proved them to be quite successful although more studies are recommended. The entire channel is not required to be covered since when the channel starts to fill up with water the snow will become saturated and erode easily.

- Heating cables. This method has been used successfully in culverts and the method has been transferred to different parts of the spillways (not reported in tunnel spillways), see Figure 8.7. The method requires electricity supply, but it only has to be switched on in periods when the spillway capacity is required.

- Location of spillway. During the design of new spillways, the topography and typical wind pattern should be studied before the location of the spillway is set in order to reduce problems with the accumulation of snow.

- In the steppe regions of Russia snow fences made with rows of planted trees or local bushes are common. This is an environmental friendly solution to protect structures from being covered and overtopped with snow.

Fig. 8.6 - Concrete roof at the side channel at Sønstevatn spillway
in Norway, view from upstream (Lia, 1998).

Figure 8.7 - Chute-Allard, Hydro – Quebec
a) and b): Summer 2010: Installation of heating plates for 3 inflatable dams
c) Spillway in Winter - Snow accumulation downstream
d) Heating plates experimented during winter

8.2.4 Ice drift

The ice coverage on top of rivers or lakes will break up when the water flow increase. Ice chunks will then be transported by the flow and can cause severe clogging of spillways. The spillway structures must be designed to handle the load from the drifting ice chunks.

8.3 ICE LOAD ON SPILLWAY STRUCTURES

In order to ensure stable operation of spillways, water conveyance structures and water intakes in severe climatic conditions, the impact of ice on the structure, particularly on gates and mechanical equipment should be reduced to a minimum or taken into account. The impacts and loads of ice on structures are discussed by Gosstroy (1989).

Spillway design allowing flow under the gate below the downstream water level should be avoided because it is practically impossible to exclude the impact of ice on the gate from the downstream side.

To minimise the impact of ice on the mechanical equipment (gates) from the upstream side, booms and baffle walls are designed, as depicted in figure 8.2 and 8.3. Installation of the booms below the ice penetration level should ensure the absence of the direct impact of ice on the gate, i.e. the boom is installed below the depth of frost penetration and below the depth of ice cover. The effect of the boom structure on the discharge capacity has to be considered during design. Booms take up the pressure of ice on the structure and therefore have to be carefully designed to withstand this load.

Booms or baffle beams can also serve as protective structures from floating debris.

8.4 OTHER CHALLENGES CONCERNING DIFFERENT PARTS OF THE SPILLWAY SYSTEM

8.4.1 Gated structures

Ice formation in slot structures or behind the gate caused by frozen leakage water should be avoided. This can cause the gate mass to become much larger and may keep the gates completely stuck in the ice or put the hoisting equipment out of service. The following measures can be undertaken to reduce this problem:

- Heating slot structures and heat insulation can be provided to prevent the gates to get frozen and ice to be formed at the gate sheathing on the upstream side. Heat insulation layer thickness shall be calculated with the use of thermal physics methods proceeding from thermal conductivity of the material. Heating of slots and the gate with built-in electrodes helps to remove ice from the structure. For provision of reliable and uninterrupted electric heating the electric circuitry must be doubled.
- When choosing the lifting equipment the additional weight of heat insulation and possible ice sticking onto the gate structure must be taken into account.
- Design of a thermal shield can maintain a microclimate with positive temperatures inside the structure that has a positive effect on the operation of the gates, on the hoisting equipment and on the structure as a whole. Thermal shields can be used at tunnels, discharge pipes, open discharge chutes or behind the spillway structure. Possible leakages from under the gate seals should be collected and diverted to the downstream pool by a drainage system where the outlet must be under the level of frost penetration.

8.4.2 Tunnels with linings

Frost penetration to the tunnel linings and liners must not be allowed. In regions with a wide range of temperature variation (from "–" to "+") steel linings of tunnels and shafts experience considerable deformations. The presence of the shaft and non-gated weir on a control structure complicates the structure protection. Taking into consideration the difference between the atmospheric pressure at entry and exit of the tunnel with shaft and natural temperature of the soil below the frost penetration depth, a thermal shield at the control structure makes it possible to maintain positive temperatures inside the tunnel and the shaft.

8.4.3 Spillway chute

In case of an open spillway chute, the thickness of the bottom slab and chute walls must be designed concerning thermal insulation properties of concrete not allowing frost penetration under the slab and behind the walls. If the concrete thickness is not sufficient, heat insulating materials can be added in the structure. Frost penetration into the drainage under the bottom slab or behind the chute walls should normally not be allowed. Drainage should preferably be located below the frost penetration depth of soil (See Figure 8.3b).

8.4.4 Water mist and icing

In cold climatic regions, floods can start in the midst of the cold season as a result of sharp short-term warming or when average daily temperatures are below 0º C. If the flood reach the structure at the moment when the thaw is over and the average daily temperatures have become below zero flood passing can cause misting and icing.

If the temperature in the middle of the day is above 0º C while the night temperatures are below 0º C a structure with high aeration (e.g. in the spillway transition section or in an energy dissipater/flip bucket) may cause mist and icing.

Water mist can be transported for quite long distances and may cause icing not only on the spillway structures but also on other nearby structures. The design of structures in heavy climatic regions should avoid design solutions connected with formation of water mist in the spillway system. A striking example of ice formation over the spillway is the Sayano-Shushenskoye hydropower plant, where rapid water discharge through the structures during winter resulted in failure of the hydropower plant and putting it out of service.

8.4.5 Energy dissipation

Designing the energy dissipation structures for severe climatic conditions requires special approaches. The design shall take into account possible freezing – thawing of the structures and ice effect on them. In this connection the energy dissipation arrangements (splitters, toothing, blockwoods etc.) shall be located below natural water level and below depth of frost penetration. This will help the energy dissipaters to perform their functions and protect them against various kinds of defects (crumbling, chips, cracks).

8.5 SUMMARY

In the design and construction of spillway structures in severe climatic conditions, it should always be kept in mind that the period characterized by negative average daily temperatures (in Celcius), i.e. winter period, can last 6-9 months. The design and construction in permafrost regions are not considered in this paper as it is a separate subject requiring an individual approach. Negative temperatures during construction and operation of spillway structures impose specific requirements to the design of structures and construction material – materials must withstand low temperatures without loosening their properties and, more important, withstand alternating cycles of freezing and thawing without destruction, i.e. to be "frost-resistant".

First of all this applies to concrete used in structures, elements of rockfill or support, and drainage. Particular attention should be drawn to elements of structures located in the variable area – being alternately in water and in the open air. These areas are the most critical.

When designing spillway structures in cold climatic conditions, thin-walled structures, fully frozen and not ensuring proper thermal insulation of concrete blocks, soil and structures behind them, should be avoided.

In harsh environments steel structures should preserve the geometry at a large temperature range of +40˚C to -40˚C or more.

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9. COMPOSITE MODELLING OF HYDRAULIC STRUCTURES

9.1. INTRODUCTION

For many years physical hydraulic modelling has been a standard design tool for hydraulic engineering of dams and their appurtenant structures. An engineer's ability to gather and analyse data from physical models continues to improve with the development of more sophisticated electronic instrumentation, faster computers and advanced software. These same advancements in computer technology have enabled the development of more advanced mathematical modelling techniques and several powerful 2-dimensional and 3-dimensional algorithms are commercially available as options to hydraulic engineers.

Physical modelling often utilises numerical modelling as part of the modelling package. For example, it is not uncommon for the design engineer to utilise a numerical model to help develop a proposed design for a hydraulic structure. Numerical modelling can be cost effective and accurate when based upon quantifiable field or physical modelling data. When a numerical model is based upon sound prototype or physical modelling information, the numerical model can be used for years on end as a valuable design and operational tool. Yet numerical modelling is limited when it comes to multi-phase flows (air/water mixtures), highly turbulent flows with vortices and flows where scour and/or sedimentation are a concern.

The current trend for modelling hydraulic structures is to utilise both a numerical and physical model in parallel during a study (also known as composite modelling). Composite modelling is the effective utilisation of both physical and numerical modelling, used either in series or in parallel with one another to solve difficult hydraulic problems.

Although physical modelling is the proven standard for modelling hydraulic structures and has successfully been used for decades, it comes with limitations and constraints. Similarly, numerical modelling, which is relatively new, also carries with it a number of limitations and constraints as well as benefits not found in physical modelling. However, when the two modelling techniques are used together during a hydraulic study, many of the limitations of one technique can be complemented by the other and vice versa. Accordingly, when researchers understand the benefits and limitations of each modelling technique, they can develop a research plan that utilises composite modelling approaches where both modelling techniques are used together to increase the efficiency and effectiveness of the modelling process.

For example, a numerical model may first be utilised to determine a proposed design for the structure. A physical model is then commonly constructed of the proposed design. Data is collected from the physical model and that data is used to calibrate the numerical model to ensure its accuracy. The physical model is then used to optimise hydraulic efficiency and minimise construction costs of the original design. With a calibrated numerical model, it (the numerical model) can then be utilised for many years to check flow conditions or operational procedures even after the physical model has been dismantled. The important thing to remember with this approach is that numerical modellers need real data either from a flood condition through the prototype or from a physical model study. The physical model study is critical to the success of the numerical model. Currently, the trend in modelling is to utilise the benefits of both for design.

9.2 PHYSICAL MODELLING OVERVIEW

Physical models of hydraulic structures are typically modelled using Froude- scaling, which is based on the ratio of inertia to gravity forces. The similarity parameter used to operate the model is the Froude number, defined as:

$$Fr = \frac{\mathrm{V}}{\sqrt{\mathrm{gy}}} \qquad (1)$$

where y is a characteristic linear dimension, g is the gravitational acceleration constant, and V is a characteristic velocity. Similarity is achieved by operating the physical model so that the Froude number in the model is the same as the Froude number in the prototype. This ensures that the inertial and gravity forces are scaled properly. The proper length, discharge, velocity, pressures, time and rotational speed for the model in relationship to the prototype can be calculated using Froude relationships.

For a model to properly simulate hydraulic conditions in the prototype it is necessary to maintain geometric, kinematic, and dynamic similitude. Geometric similitude is achieved by carefully building the model to a selected scale ratio with all geometric components scaled appropriately.

The selection of the model scale has a direct impact on its ability to maintain kinematic and dynamic similitude. Physical models are always built as large as possible to minimize the scaling effects attributed to other forces, such as viscous and surface tension forces. When a large physical model is constructed (small scaling ratio), then flow phenomena such as super elevation, flow surging, wave action, hydraulic jumps, vortices, flow separation, local scour and erosion, air entrainment, standing waves, complex and composite roughness, etc., can be accurately modelled. In addition to being able to physically simulate complex flow conditions, the physical model provides a hands-on working simulation that most engineering teams find essential to the design process. Some of the more significant benefits and limitations of physical modelling are listed and summarised below.

9.2.1 Benefits of Physical Modelling

- As long as the scale ratio is properly selected and the model is constructed as large as possible to avoid potential size scale effects, physical modelling is a trusted and reliable method that will produce accurate results.

- Small changes can be made in the physical model very easily using temporary structures of wood, plastic, mortar, sandbags, etc., so that the resulting hydraulic changes can be seen immediately. It is time consuming to simulate a wall or a raised embankment into the grid of a numerical model, especially if the model is complex and the mesh resolution is fine, but only takes a matter of minutes to see the effect in the physical model.

- Multiple flow rates can be tested on a single model configuration in a very short time. For example, a flood routing or flood hydrograph can be simulated in a matter of minutes. Hydraulic design configurations can experience a hysteresis of flow conditions with changing flow rates. Physical modelling can easily determine the presence and magnitude of such hysteresis effects.

- Specific hydraulic problems are immediately apparent in an operating physical model, and many times, there are multiple problems occurring simultaneously. Problems like vortices, separation zones, scour potential, poor energy dissipation characteristics, wall overtopping, aeration effects or wave action are noted immediately in a physical model.

- Once constructed, a physical model can quickly determine the flow capacity of a flow control structure by increasing supply or inlet flow to the model until the given pool elevation is reached. Depending upon the complexity of the geometry of the control structure and the approach channel, the numerical modeling effort to determine flow capacity can be significantly more time consuming.

- The effects of tailwater or submergence on a hydraulic structure are easily and quickly observed in a physical model by simply varying the height of water in the downstream channel.

- Three dimensional effects, aeration effects, highly turbulent regions, standing waves and super-elevation conditions are easily modelled in a physical model. However, one must be wary of scale effects when evaluating aeration.

- Physical models facilitate "team" engineering. Geotechnical, structural and hydraulic engineers can get immediate feedback and make contributory suggestions to the design of the hydraulic structure as they witness flowing water through the physical model.

- Physical models will allow you to video–tape or photograph the moving water so that modelled flow conditions can be compared to prototype conditions providing a valuable tool in quality control.

9.2.2 Limitations of Physical Modelling

- The physical model can be limited by the required structure cost and size, and the time required to build and acquire data. The cost and time is usually proportional to the amount of information that is required and the time it takes to collect the data. It is often difficult to physically model the full extent of the flow region or channel and the approach conditions, due to limited laboratory floor space. Major changes to a physical model are often time/labour intensive and expensive.

- Pressure, flow rate, and velocity measurements taken in the physical model introduce some uncertainties as a function of the accuracy of the instrumentation being used for the measurements. It can be difficult to make detail pressure measurements in a physical model due to the installation of pressure taps, pressure transducers, and the presence of un-dissolved air in the flow.

- If a physical model is not scaled properly, then the data collected or the cost of the model is affected. If the model scale is too large (small-sized model), then size scaling effects can influence the accuracy of the results. If the model scale is too small (large-sized model), then the cost of the model will increase. It should be emphasized however, that it is very important to construct any physical model as large as possible to reduce scaling problems.

- It is difficult to vary the boundary roughness in a physical model and often a separate sensitivity study is needed to evaluate the effect of roughness.

- Utilising Froude similitude for the physical model study can be problematic in that full velocity profiles are not developed along the boundary because the model is not operating at the correct Reynolds number when operating at a Froude-scale velocity.

- Project schedule and limited budget usually minimise the number of possible design configurations or modifications that can be tested.

- The physical model has to be dismantled at some point in time, and may then not be available for other design changes resulting from future changes in construction or other constraints.

- There are real hydrodynamic processes which are either poorly or practically impossible to simulate on the physical models. Among these is, for instance, formation of water-air-ice cloud in the spillway tailrace with a high degree of flow aeration (for instance overhang spillways with jet ejection).

9.3 NUMERICAL MODEL OVERVIEW

Although there are multiple approaches to numerical modeling such as 1D and 2D methods, the full 3D method commonly referred to as computational fluid dynamics (CFD) is the type of numerical modeling referred to in this chapter. The most common CFD programs solve the Reynolds-averaged Navier-Stokes (RANS) equations using a variety of approaches. Most of the leading commercial CFD programs on the market have the ability to track free surface flows commonly associated with hydraulic structures with the Volume-of-Fluid (VOF) method or some variation thereof being the most common algorithm for tracking the free surface interface.

CFD codes solve the RANS equations using a finite volume method applied to a computational domain. The computational domain defines the region of interest as discretised (gridded) into smaller cells. Gridding techniques are usually classified as Boundary (Body) Fitted Coordinates (BFC) or a porosity technique. Solid objects are defined in BFC coordinates by wrapping the grids around the object whereas in porosity methods, the solid is imported into the grid and the solid blocks the fluid flow. Each method has its advantages and disadvantages; BFC tend to be more computationally efficient, especially in resolving boundary layer hydraulics, but grid construction can be time consuming whereas porosity methods are easier to grid but not as computationally efficient. Additionally, the porosity method lends itself for importing survey data (site geometry) into the model.

One of the difficulties in numerically solving flow in a hydraulic structure is the presence of a free surface, in which the location must be solved as part of the solution and may be transient in time. This is especially difficult when the water surface is rapidly changing with a high degree of curvature, such as when the flow changes from subcritical flow to supercritical flow as it flows over a spillway crest.

Another common technique used in CFD modelling is to approximate curved surfaces with a straight approximation. It is easy to see that the use of smaller grids and thereby more straight segments provides a better approximation to a curved surface. Therefore, in order to improve the approximation, variable grid spacing can be used with a tighter spacing or smaller grid size located near high curvature areas on the obstacles and in locations where a rapidly varying water surface was expected. The downside to smaller grids is the increase in computational time.

To numerically solve the RANS equations across the gridded domain, there are a variety of numerical approaches that can be used and each one may have multiple parameters. Primary inputs include defining the boundary conditions, the turbulence model and its parameters, model roughness and the numerical approach for solving the equations (implicit versus explicit, 1^{st}, 2^{nd} or 3^{rd} -order etc.). Some parameters such as the boundary conditions are critical for properly simulating prototype conditions. Other parameters provide more subtle changes. Understanding the effect of the various methods, approaches and parameters are crucial in making sure that the model represents the true physical flow.

Peer-reviewed papers have validated the use of CFD to model free surface flow for many different applications especially for subcritical flows. However, CFD modelling has not been validated for all types of flows and like any numerical method, the computed results can be no more accurate than the data and assumptions on which the computational techniques and methods are based – i.e., accept the values as a decent approximation, but not the absolute truth. A summary of some of the more significant benefits and limitations to numerical modelling are listed and summarised below.

9.3.1 Benefits of Numerical Modelling

- A numerical model is a good choice for large changes to physical topography or to the location of a control structure.

- Numerical modelling provides the option of testing a configuration without fabricating a physical structure; thus allowing multiple configurations to be investigated at the same time using multiple computers. Numerical modelling allows for the automated multivariate parametric optimisation of design elements.

- Data produced from numerical modelling results allows for velocities, pressures, turbulence parameters and water surface profiles to be determined at each time step and at any location in the flow domain.

- Numerical modelling helps the researcher see the "big picture", or in other words, where problem areas may be located. Flow patterns such as velocity profiles and vortices can be easily mapped using vector plots, streamline and flow ribbons.

- The numerical model can eliminate the need for physical sectional models [example: section of a spillway constructed at a smaller scale (larger model) so that spillway surface pressures can be evaluated]. The numerical model does a good job of evaluating fluctuating pressures as well as the magnitude of negative pressures along a fluid boundary.

- RANS CFD models do struggle to simulate pressure fluctuations associated with dynamic eddy structures that are small and therefore smoothed out (e.g. pressure fluctuations present in a hydraulic jump). RANS CFD models are good at surface pressures if the flow lines are parallel near the surface and relatively steady with time. Large Eddy Simulation (LES) and Detached Eddy Simulation (DES) CFD models largely eliminate this issue but are computationally expensive and are not (yet) widely used for free surface flows.

- Significant changes to a modelling configuration are simple to construct on the computer and only have labour and software licensing costs associated with them.

- Numerical modelling costs are often less than a physical model.

- Numerical models can be stored and maintained for future use/changes.

- Animations of the flow, including the free surface can be constructed using a variable of interest such as the velocity or pressure.

9.3.2 Limitations of Numerical Modelling

- A numerical model benefits from having prototype or physical modelling data for calibration but this is not always available or required.

- Numerical models are limited in the selection of input and operating parameters when there are no prototype data or physical modelling data to use as a basis. Additionally, numerical modellers who have not been properly trained or do not have vast experience in specific applications, introduce the possibility for data errors.

- Numerical models are limited in their application of geometric roughness and the development of the velocity profile at boundaries (law of the wall), especially in high turbulence supercritical flows. Depending on the grid size of the numerical model, the true near-the-wall velocities may not be accurate since the law-of-the-wall approximations are dependent on numerical grid size and other factors.

- Numerical modelling is often constrained by gridding/computational time for models with large spatial and/or time domains. To reduce computational time some CFD codes include hybrid solutions which allow 2D shallow water approximations for large areas where vertical gradients are not critical to be combined with full 3D hydraulic analysis in local regions.

- The number of runs required to evaluate the routing of a hydrograph will be very time consuming for a numerical study. Similarly, simulating the routing of a complete hydrograph can require simulation times that are unrealistic.

- Hysteresis effects between increasing flows of a hydrograph as compared to the decreasing flows of a hydrograph can happen and be significant. Numerical models are limited in their ability to analyse this type of flow relationship.

- It is not possible to video–tape or photograph the moving water of a numerical model, so that modelled flow conditions can be compared to prototype conditions as it can be done with a physical model. This type of analysis is extremely valuable as a quality control to ensure that the modelling is accurate. However it is possible to create visual simulations using numerical output in the form of short video clips that aid in the comparison with prototype conditions.

- A numerical model is limited in its ability to analyse the hydraulic stability and effect of unsteady flow downstream of a control structure.

- Flow conditions like a hydraulic jump can be very sensitive to changes in numerical boundary roughness and there is wide latitude in the application of the theory if prototype or physical model data is not available.

- A numerical model is limited by size of the computational domain both spatially and temporarily. A large physical domain may require a 2D solution rather than a 3D solution, or a hybrid solution as noted above, thereby potentially reducing the accuracy of the solution.

- Numerical models are limited due to the time it takes to construct the numerical grid and run the model when complex flow conditions are being simulated.

- Shallow flows can be computationally intensive, requiring a small grid spacing to accurately simulate a velocity profile. An example would be an extremely low flow over an ogee crest. Fortunately, most design concerns are based on larger flows.

- Complex flow conditions from turbulent flow, re-circulating flow, flow separation, and flow super-elevation around a flow bend may not be accurately calculated in a numerical model due to turbulent modelling limits. Although turbulence models work well for certain documented cases, they may not be appropriate for all flow conditions.

- Incorrect or improperly applied gridding techniques have the potential to introduce numerical errors. Grid convergence testing is necessary to determine the adequacy of the selected grid sizing.

- There is a non-proportional time requirement to simulate and analyse a very small change in a numerical model as compared to the time it takes in a physical model.

9.4 COMPOSITE MODELLING OVERVIEW

Composite modelling is the effective utilization of both physical and numerical modelling, used either in series or in parallel to solve difficult hydraulic problems. Despite weaknesses in the two modelling techniques when they are utilized independently, composite modelling has proven to be extremely effective and efficient. A properly applied composite modelling approach will improve the accuracy of the flow data and measurements by focusing on the benefits of both physical and numerical modelling as they are used together. It should be noted however, that composite modelling will not always reduce overall cost over using one modelling technique or the other. The value of composite modelling is normally in the quality of flow data and the confidence in the proposed hydraulic design and not necessarily in the cost. The composite approach allows for more design options to be evaluated and therefore the design to be optimised.

Composite modelling can be viewed in three components:

1) numerical modelling performed before the physical model is constructed and tested;

2) numerical modelling that is performed at the same time or in parallel with the physical model; and

3) numerical modelling that is performed after the physical model study has been completed.

9.4.1 Numerical Modelling Performed before the Physical Model is Constructed and Tested

- Prior to the construction of a physical model, numerical modelling results will provide information so that the approach geometry (i.e. the physical laboratory head box) of the physical model can be properly aligned with the approach streamlines in the flow, and can be minimized so that the head box that supplies water to the model is not constructed any larger than necessary. This reduces construction costs for the physical model and allows a larger sized model (smaller scale) to be constructed, thereby reducing the Reynolds scale effects especially in the region of interest.

- The numerical model can help reduce the number of physical modelling configurations necessary by performing pre-runs. The numerical model can test design conditions and alternative design concepts before the physical modelling to make decisions about the physical model and to obtain a general acceptance for design.

- The pre-runs from the numerical model will help engineers understand the general hydraulic problems; identifying problematic regions where flow separation, excessive velocities or non-uniform flow may occur.

9.4.2 Numerical Modelling that is Performed in Parallel with the Physical Model

- One of the most significant benefits of composite modelling is "modelling the model", which means that the exact geometry of the initial or baseline physical model is numerically modelled at a 1:1 scale so that numerical modelling errors can be evaluated and corrected if possible. This quality control effort is effective in reducing or eliminating the uncertainties of the numerical model.

- It is also possible for the calibrated numerical model to then be rescaled to model the prototype to evaluate potential physical modelling scale effects. Again, these scale effects should be minimal for the physical model if the physical model is constructed large enough to minimize the scale effects.

- Once the numeric model has been calibrated or adjusted to the results of the physical model, it can be used to evaluate a myriad of discharges, velocities, water surface profiles and detailed surface pressures and shear stresses that are labour intensive and expensive to perform in the physical model. Adjustments to the numerical model to match the physical model are normally done by adjusting surface roughness, turbulence model parameters and grid cell resolution until the correlation between the two modelling types seems reasonable.

- With a calibrated numerical model, any proposed structural or geometric change that requires major modifications to the physical model can be simulated in the numerical model to evaluate design acceptability as compared to the initial baseline calibrated numerical modelling results. The results of these numerical modelling runs are then carefully compared to determine the appropriate trial or final geometric configuration that should be physically modelled.

- The trial or final physical model configuration is then constructed in order that detailed hydrograph and rating curves can be tested. A physical model can quickly and accurately determine the rating curve of a hydraulic structure, while a numerical model will take a significantly longer time to generate the necessary runs for a rating curve. Again, however, once the noted physical model configuration is constructed, the numerical model can be used to provide an infinite number of data points, streamlines and/or flow ribbons within the flow domain that would take considerable effort to collect in the physical model.

- The physical model can be photographed and the flow conditions and problems associated with the structural design can be video-taped so as the data associated with both the physical and numerical models are reviewed, the recorded flow conditions can be reviewed. These recordings are also extremely valuable in publicizing the design to the general public and to funding agencies.

- During this phase of the composite model, regions in the flow domain in which the numerical model simulations are questionable are documented. This may include areas of highly turbulent flow, areas within the larger model in which extremely small depths and high velocity flows occur, flow conditions that are near critical depth, transient surges or wave oscillations, super-elevation as a result of supercritical flows rounding a tight curve, regions where scour is a potential problem and flow conditions in which the water is highly aerated.

9.4.3 Numerical Modelling that is performed after the Physical Model Study is Complete

After every physical model study, it is necessary to dismantle the model so that the space that was utilised for the physical model in the laboratory can be used for another physical model study. However, the numerical model does not take up any space and does not need to be discarded. The benefits of having maintaining a calibrated numerical model are:

- The numerical model can be used indefinitely after the physical model is dismantled.

- Additional information can be collected within the flow field (pressure and velocities recorded at every cell) if needed after the physical model has been dismantled.

- Minor modifications to the geometric design can be made and tested in the calibrated numerical model with confidence that the results will be accurate.

- The calibrated numerical model can be used for training and operational decisions after the physical model has been dismantled.

9.5 SUMMARY AND CONCLUSIONS

The use of both a physical and a numerical model together as a composite model has proven to enhance the hydraulic modelling effort, improve modelling accuracy and reduce modelling uncertainty. Composite modelling has been found to be an effective tool for hydraulic structure design.

Composite modelling provides a unique opportunity for researchers and engineers to understand the uncertainties and limitations of both the physical model and the numerical model, since their parallel operation allows for direct comparison and calibration. When complex three-dimensional flow conditions are being modelled, numerical models are often limited in their ability to simulate the flow field in all regions as compared with the physical model. This would include flow conditions with water surface profiles in which transverse super-elevation or non-symmetrical hydraulic jumps occur. This also includes flow conditions where standing waves in flows that are near critical depth are present, where upstream subcritical flows are being merged across a control structure to flows that are rapidly changing downstream of the control structure and where highly turbulent and/or mixed flow regimes occur.

In the past, numerical modelling has been proven to be very effective and accurate when applied to two-dimensional and some three-dimensional flow applications, but many of the hydraulic problems associated with hydraulic structures are complex, highly turbulent and three-dimensional, and numerical modelling can be questionable for these types of flow conditions. Therefore, composite modelling allows for the verification and validation of flow rates, water surface profiles and point velocities within the flow domain and consequently determines specific regions within the numerical model that are not being simulated accurately.

Composite modelling can produce a calibrated numerical model to collect very large and detailed amounts of data from the flow domain and produce a model that can be effectively used long after the physical model is dismantled. Composite modelling provides an opportunity for the numerical model to be calibrated to both the prototype structure at a 1:1 scale and to the physical model at a 1:1 scale. This is all very important to the quality control component of the study.

The use of numerical modelling as an integrated part of the modelling process has proven to be both cost effective and time beneficial for the design hydraulic engineer. Using a numerical model (before the physical model) to define the physical domain of the head pool and the approach flow requirements in the physical model is a valuable component for the modelling process. This pre-modelling provides important information so that the physical model can be constructed as large as possible, thereby minimizing possible scaling effects. This pre-modelling also provides information about how the water supply to the physical model should be designed to ensure appropriate approach flows are simulated.

Additionally, the numerical pre-modelling provides information about potential problems associated with the theoretical design or proposed design changes to the structure, thereby reducing the number of physical modelling configurations necessary during the physical modelling portion of the study. This pre-modelling effort saves time and money once the physical modelling has commenced. With a calibrated numerical model, the numerical model is a valuable tool in noting hydraulic roughness sensitivity and other relative changes that are not so easy to modify in the physical model. As previously mentioned, the numerical model provides great benefits for quality control since it can be scaled to model the physical model directly, thereby producing validation of geometry and operating conditions in the physical model. Finally, the numerical model is a very important tool for maintenance and operation of the hydraulic structure for use after the physical model has been dismantled.

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10. ECONOMICS, RISK AND SAFETY IN SPILLWAY DESIGN

10.1 INTRODUCTION

Spillways are the main safety-assurance feature of any type of dam. For this reason as a rule, direct cost reduction attempts for these structures will, in many cases, mean increases in the risk and lowering the safety of the overall dam project. In many projects, the direct cost of spillways may represent a reasonable important portion of a project and its reduction may become a tempting target for designers, but because of their fundamental role in providing safety, the actions or measures intended to obtain savings in spillway design and implementation must be carefully evaluated without losing sight of the role of the spillway. In spite of that, economic and cost optimisations in spillways are possible and desirable. This chapter discusses these aspects.

The importance of a safe spillway cannot be overemphasized as many failures of dams have been caused by improperly designed spillways or by spillways of insufficient capacity. Ample capacity is of paramount importance for earth-fill and rock-fill dams, which are likely to be destroyed if overtopped, whereas concrete dams may be able to withstand moderate overtopping. Usually, an increase in the direct cost of the spillway is not directly proportional to the increase in capacity. Very often, in concrete dam projects, the cost of a spillway of ample capacity will be only moderately higher than that of one with much smaller capacity.

In many dam projects, economic considerations will lead to a design utilising a reservoir surcharge above normal operating level. The most economical combination of surcharge storage and spillway capacity requires flood routing studies and an economic appraisal of the costs of the spillway–dam combinations. However, in conducting these studies, consideration must be given to the minimum size spillway which must be provided for safety. In many cases the information from the inflow flood hydrographs used for defining the spillway design flood (SDF) stresses the flood peak but not the flood volume. Often, such floods may have the highest peak flows but not always the largest volume. When spillways of small capacities in relation to these inflow peaks are considered, precautions must be taken to insure that the spillway capacity will be sufficient to: (i) evacuate reservoir surcharge so that the dam will not be overtopped by a recurring storm; and (ii) prevent the reservoir surcharge from being kept partially full by a prolonged runoff whose peak, although less than the inflow flood peak, exceeds the spillway capacity.

Economics and cost savings in the construction and operation of dams and their flood control features have been the object of various ICOLD Technical Committees and bulletins, but are always guided by the fundamental issue of safety assurance. These questions lead to the risks that must be evaluated by the designer who has the duty of balancing the corresponding effects to an acceptable level.

ICOLD Bulletin 152 – *Cost Savings in Dams*, discusses criteria used for the design of dams and appurtenant structures and specifically, for spillways, from the point of view of cost reductions, economics and risks. The concepts discussed included the selection of spillway design floods (SDF), the difficulties of assigning a failure probability to a spillway arrangement and tendency of design practice to consider only performance and ignoring costs.

In recent years innovative approaches have been developed in spillway design to address dam safety concerns at a lower cost. Such include: (i) the use of overtopping protection on embankments to prevent failure during large flood events; (ii) raising of dams with parapet walls to improve flood routing and to gain greater storage; and (iii) the application of a risk assessment process to determine the appropriate risk in terms of risk to human life and property posed by dam failure. This latter point is discussed extensively in ICOLD Bulletin 154 – *Dam Safety Management: Operational Phase of the Dam Life Cycle.*

Methodologies and guidelines have been developed to analyse the possibility of lowering safety requirements for spillway design criteria when incremental consequences (or damages) induced by a dam failure become insignificant compared to those generated downstream by natural floods themselves. Such approaches, particularly suitable for optimising spillway capacity to level of safety requirements at existing dams, have been implemented in some cases and cost-benefit analysis methods are tested to support the acceptability to avoid spillway capacity enhancement when the cost of spillway reinforcement is deemed disproportionate to the net safety improvement gained through the spillway upgrade project.

In addition, some efficient new spillway alternatives have been developed and used improving, cost wise, traditional solutions. Previous chapters of this bulletin described and discussed some of these alternatives.

This chapter addresses the strategies related to spillway design in dam projects utilizing different possibilities, alternatives and methods to gain overall economic advantages. As a rule, no cost reduction measure should be taken to lower the discharging capacity of the spillway system through transfer of part of that capacity to other structures or arrangements.

10.2 DESIGN FLOOD AND CHECK FLOOD

A survey of existing projects shows that in many modern projects the spillway system is designed for a peak flow value based on criteria, which usually consider the spillway design flood (SDF) as the inflow, which must be discharged under normal conditions, with a safety margin provided by an accepted freeboard limit. In addition a spillway check flood (SCF) is defined as the maximum flood that will not cause the destruction of the dam and beyond which the safety cannot be assured (ICOLD Bulletin 82, 1992). The safety of the project is ultimately based on the spillway system capacity to discharge the SCF. This approach is also the standard criterion in many countries where there is an official recommendation for the design of dams.

In general, the SCF is a flood with a lower probability of occurrence than the conventional SDF. In many cases it is assumed to be the PMF or the theoretical maximum flood that could occur at the dam site. Although the SCF is computed not to overtop the dam, in general, because of its low probability, some degree of damage in the structures is tolerable. As discussed in ICOLD Bulletin 152 (pgs. 133-135) it is possible to reduce the cost of the spillway by increasing the difference between the SCF and the SDF; in other words, by reducing the SDF without diminishing the safety of the project, which is ensured by the capacity of discharging the SCF. This capacity is associated with the possibility of an increase of the elevation of the reservoir water level. Although in many projects this situation corresponds to encroaching into the normal freeboard, raising the dam crest elevation (and computing the corresponding costs) to allow a larger freeboard is also used. The increase in the flood elevation of the reservoir allows a certain degree of flood peak attenuation, which is discussed below.

10.3 ATTENUATION OF FLOOD PEAK

The provision of an additional volume, above the maximum operational level of the reservoir, to store part of the flood hydrograph allowing a reduction of the maximum flow discharged by the spillway system, is a common strategy for reducing the direct spillway cost without affecting the safety of the dam. However, this is only feasible if the area of the reservoir is large enough to provide a large volume with a limited increase in elevation. The elevation considered in the design, to be reached when the reservoir inflow flood occurs, is generally called the maximum flood level. This elevation is higher than the maximum operational level.

Cost wise, the benefit of reservoir attenuation is only effective if the hydrograph coefficient (defined as the ratio of the flood hydrograph volume divided by the product of the reservoir surface area and the height of the dam) is much smaller than unity (Bouvard, 1988).

In any case, a freeboard above the maximum flood level, whether or not coincident with the maximum operational level, should be provided for all dam projects. This provision is intended to protect the dam from waves produced in the reservoir and is normally sized by using standard well established criteria that take into consideration a certain maximum probable wind blowing in the direction of the reservoir's longest fetch with the stored water in the reservoir at the maximum flood level. The probability of a simultaneous occurrence of the maximum flood with the reservoir at its maximum level and the maximum wind producing the maximum wave height, can be assumed as being much lower than the isolated probability of the occurrence of either of them. This has justified the consideration, recommended by some countries (such as China, for example), to consider the reservoir maximum flood level (the SCF level) coincident with, or slightly below, the elevation of the crest of the dam. It should be remembered in this respect, that in a dam where there is a combination of a concrete portion and an embankment portion, it is usual to set the embankment crest elevation about 1m (or more) higher than the concrete crest elevation, to ensure that in the event of an overtopping, the concrete portion would be overtopped first and the risk of overtopping the embankment portion is at least reduced.

This consideration means that the freeboard allowance above the check-flood level will be reduced in relation to the corresponding allowance above the maximum operational level. As mentioned previously, the cost saved for the spillway structure is the result of a smaller SDF to be discharged when the reservoir level is at the maximum operational level. This reduction of the spillway design flood is permitted by the higher water level elevation of the check flood and by the peak attenuation of the flood hydrograph.

10.4 DIVIDING THE SPILLWAY INTO SEPARATE STRUCTURES

One of the challenges to reduce the cost of the spillway for large dams with large flood discharges is to combine different flood discharge outlets. There are many different types of spillways and despite of some types being more adaptable to specific projects or dams, there is the possibility of reducing the investment cost by considering different arrangements or variations of the same type. The economy sought must not be obtained to the detriment of the safety of the project; this means that the design solutions considered must keep the capacity of discharging safely the full project design flood.

Since the frequencies of occurrence of the magnitudes of the flow to be discharged vary, a key strategic approach to the economical design consists in considering the total discharge capacity of the project divided into different type of spillways and designing each one with criteria compatible to its safety and operation expectation. For such cases, clear operating rules must be defined. In concrete dam projects it is possible to have distribution of discharges through chutes, orifices and crest spillways. As shown in Karun III dam and HPP project, (Fig. 10.1) the 20 000 m^3/s flood is discharged through a combination of a chute, orifices and crest spillways. Incidentally, as mentioned in chapter 2 of this bulletin, in Karun III dam, due to deep excavation required at the left bank, a lined plunge pool was provided with a 36 m high tail pond dam which was constructed at the downstream. Here, the chute works as service spillways, orifices as auxiliary spillways and dam crest as the emergency spillway as shown in the figure 10.1.

Fig. 10.1 - *Karun III 205 m high concrete arch dam, service, auxiliary and emergency spillways (Iran)*

10.4.1 Service spillways

The service spillway is the spillway that provides continuous or frequent regulated (controlled) or unregulated (uncontrolled) releases from a reservoir, without causing damages to the dam, dike or appurtenant structures due to the releases up to and including its design discharge. When only one spillway is provided, its design discharge must be equal to the project SDF. Service spillways can be classified into basically two types, surface spillway and orifice or tunnel spillways.

The choice as to whether use a surface spillway or an orifice spillway is generally governed by specific site characteristics. In a restricted canyon, it is often difficult to incorporate a surface spillway and an orifice or tunnel spillway is the only logical choice. An orifice spillway has the disadvantage that discharge through the spillway is a function of the square root of the head ($\sqrt{H}$) available while that of the surface spillway is a function of the head to the power of one and a half ($H^{1.5}$). For this reason orifice spillways are generally found efficient only on high dams where large heads are available.

Surface service spillways can be gated or ungated. Site characteristics, magnitude of the design flood, estimated risk and cost of different possible alternatives are the main parameters associated with the decision of using either type of spillways. Ungated spillways are without question the safest type of spillway. The ungated spillway is less likely to be obstructed by floating debris, and since there is no equipment to operate, its safe operation is not impaired by possible operator errors. However, ungated service spillways are generally more expensive than gated spillways for a given maximum discharge rate since they will involve long crests and consequently wide chutes or conduit diameters. The contradiction to this general rule on relative cost is given in the ungated labyrinth crest. For cases where the reservoir surface area is relatively large compared with the inflowing flood volume, a labyrinth spillway can provide an economical and safe structure. Ungated service spillways have the distinct advantage that they involve no operating equipment and, thus, require little regular maintenance.

Gated spillways are generally chosen when the site is restricted or the magnitude of the design flood is very large, and it is not physically or economically possible to construct the necessary length of an ungated crest. Control of downstream flooding, or maximization of conservation storage, requires more flexible control than would be provided by an ungated spillway. For large SDF the total direct investment cost of a gated spillway will generally be smaller than for an ungated spillway of equivalent capacity. Gated service spillway bays should always be designed and constructed with facilities for placing bulkheads upstream of the gate in order that the gate can be serviced in the dry. Examples of problems involving gates which can occur in an emergency situation are depicted in chapter 2 of this bulletin.

10.4.2 Auxiliary spillways

Where site conditions are favourable, the possibility of gaining overall economy by utilizing an auxiliary spillway in conjunction with a service spillway may be considered. In such cases the service spillway will be designed to pass floods likely to occur frequently and the auxiliary spillway set up to operate only after such frequent floods are exceeded.

It must be noted that the concept of auxiliary spillway as a complement of the service spillway so that both have the capacity to discharge the full SDF under normal conditions with a safety margin provided by an accepted freeboard limit, is not unanimously adopted. In many places the auxiliary spillway is considered also an emergency spillway, built to take care of floods larger than the conventional spillway design flood.

Considering the concept of auxiliary spillway as part of the project spillway system complementing the service spillway so that both are required to discharge the full SDF, the safety criteria regarding gate operation (when auxiliary spillway is also gated) such as redundancy of power supply for gate operation, must be the same for all spillway gates, and no cost reduction should be considered for this part of the auxiliary spillway. However, as a rule, because of its less frequent use some degree of structural damage or downstream erosion may be accepted, provided these structural damages or erosion do not affect the structural safety of the project permanent structures and occur in places that can be accessed and repaired after the passage of the flood.

A common solution used to obtain savings in large projects is to reduce or even omit the concrete lining of the restitution channel of the auxiliary spillway. However the decision to obtain savings by accepting less stringent criteria for the design of restitution channels of auxiliary spillways discharging large floods must be carefully balanced with the possibility of environmental damages such as creating large excavation gradients and landslides. Examples of projects where these solutions were applied are depicted in Chapter 2 of this Bulletin.

One important consideration in designing the service-auxiliary spillway system with less stringent criteria for the auxiliary spillway is the definition of the maximum flood discharge capacity of the service spillway which must be able to discharge all frequent floods. Large projects built with this kind of solution have established the service spillway with a capacity corresponding to return periods varying from 100 to 200 years.

In many cases the auxiliary spillway is located away from the service spillway but there are also cases in which these two structures lie side by side (Fig 10.2). In the first case conditions favourable for the adoption of an auxiliary spillway are the existence of a saddle or depression along the rim of the reservoir leading into a natural waterway, away from the dam or other structure in dam body to allow the construction of a more economical restitution channel without risk of damaging the dam.

Fig. 10.2 – *Xingó Project: service and auxiliary spillway with discharge channel partially lined (Brazil). Discharging capacity 16 500 m^3/s each one.*

10.5. EMERGENCY SPILLWAYS

This is a spillway designed to provide additional protection to the project against overtopping of the dam, and is intended to be used under extreme conditions during the occurrence of floods larger than the project design flood or maceration or malfunction of the service or auxiliary spillway. As the name implies, emergency spillways are provided also for additional safety, should emergencies not contemplated by normal design assumptions arise. Such situations could be the result of a mandatory shut down of the outlet works, a malfunctioning of spillway gates, the clogging of spillway passages by debris or the necessity for bypassing the regular spillway because of damage or failure of some part of that structure. An emergency might arise when flood inflows are handled principally by surcharge storage and a recurring flood develops before the service spillway or the outlet works evacuate a previous flood. Under normal reservoir operation, emergency spillways are never required to function.

In projects where the service spillway alone or in combination with an auxiliary spillway is designed to be apt to discharge the full SDF, the emergency spillway should not be used to justify savings or discharge capacity reductions in the operational spillway system. There are, however, projects in which the functions of auxiliary and emergency spillways are combined, as illustrated by case examples in item 10.6; in these cases the service spillway is reduced in size and provide an economic benefit for the project investment.

Not all projects have an emergency spillway, but, as discussed in ICOLD Bulletin 142, all projects should consider the possibility of occurrence of floods larger than the check-flood.

There are different types of emergency and ICOLD Bulletin 142 describes the most common types. The operating concept of emergency spillways is a structure built with soil or concrete, which is destroyed or removed when the hydraulic head relative to the reservoir water level elevation, is increased beyond a certain limit. With its destruction or removal, the outgoing flow increases, re-establishing the safety elevation of the water level and preventing the overtopping of the dam.

Therefore, the control crest must be placed at or above the designed maximum elevation of the reservoir water surface. The freeboard requirement for the dam is based on a water level elevation for which the emergency dike or concrete facility is removed. However in such a case, and depending upon the height of the destroyed embankment (fuse), it may become impossible to re-impound the reservoir up to its full supply level as long as the fuse is not re built. This may become a real drawback, even if the occurrence of such an event has a very low probability.

10.6. OPTIMISING THE OVERALL PROJECT ARRANGEMENT

In the formulation of the overall dam project arrangement the analysis of different types of spillway alternatives and different types of arrangements for these structures, for the same safety criteria, may have a significant impact on the overall cost of the project. Different spillway configurations, such as gated versus ungated spillways or a combination of these, and the division of the spillway system in service, auxiliary and eventually emergency spillways, for example, may lead to important cost savings for a dam projects. The following three cases described in continuation are examples of cost efficient solutions obtained by optimising the project arrangement by using a reduced service spillway and using an additional combination of an auxiliary and emergency spillway to take care of larger floods.

The first example is the Shamil and Nian dams which are 35 m high, earthfill dams with a common reservoir with a storage capacity of 69 x 10^6 m^3. The crest lengths of Shamil and Nian are 320 m and 530 m, respectively. This project is mainly for domestic water supply of the capital of the Hormozgan Province in the south of Iran. In the design of the project the PMF (13 565 m^3/s) was selected as a peak inflow to the reservoir. Several design alternatives for flood handling structures were considered to control the PMF.

The most economical alternative was found to be a spillway system consisting of a service, an auxiliary and an emergency spillway. This system ensures the safety of the project.

The service spillway is gated and handles floods up to the 150 year flood without assistance from the other spillways. If the flood exceeds this discharge the water will start to spill over the auxiliary spillway, with a 300 m long overflow crest. With both spillways operating the system can safely handle floods up to the 10 000 year flood. If the flood continue to increase a fuse plug used as the emergency spillway will breach, and take care of the remaining discharge. The system has a total capacity to handle the PMF without the dam being overtopped.

The probability of the PMF is always very small and where its value is several times larger than the probabilistic computed SDF, a service spillway designed for handling the SDF may be combined with an auxiliary-emergency spillway. One example of this case is the Mnjoli Project in Swaziland (Engels and Sheerman-Chase, 1985). This project had an ungated concrete spillway designed for the 100-yr flood peaking 950 m^3/s and a PMF estimated to produce a flood peaking 7,650 m^3/s, more than eight times larger than the designed SDF. A fuse-plug spillway was built to discharge the eventual PMF flood, with the peculiarity that it had two levels separated by a dividing wall. One part would overtop when the reservoir reaches the 100-yr flood level and the other, 60 cm higher, would breach for a more severe flood. In January 1984, a tropical cyclone occurred and the erodible bunds of the fuse-plugs were washed away, allowing discharge of the excess flood to be released by the fuse-plug channel and preventing the overtopping of the dam.

The third case is related to hydroelectric undertaking, the 180-MW Mrica project in Indonesia (Soerachmad, S. 1988). The Project is located in a tropical mountainous country with a high mean annual rainfall of 3,839 mm and a very rapid flood increase at the project site. The dam is an earth-rock structure, 109-m high. The flood studies concluded for the following return periods:

Return period (year)	Flood discharge (m^3/s)
100	4 200
1000	6 100
10000	7 400
PMF	9 300

The designed spillway system includes a gated service spillway, a drawdown outlet and an earth dike fuse-plug emergency spillway. The gated spillway and the drawdown outlet have the capacity of discharging the 1000-yr flood of 6 100 m^3/s. Floods above the 1000-yr flood and up to the PMF were designed to be discharged by the service spillway and drawdown outlet and by the destroyed dike of the emergency spillway, preventing the overtopping of the dam. In case all gates are damaged and remain closed the emergency spillway can discharge safely the 100-yr flood. This solution was based on economic and risk assessment studies. The project was completed in 1990 and during construction on March 1986 a flood peaking 4 486 m^3/s occurred and caused a revision of the hydrological studies and a revision of the spillway features, but not altering the design concept.

10.7. RISK-BASED DESIGN APPROACHES

Flood risk at dams is resulting from the combination of a hydrological hazard, the dam vulnerability or sensitivity, and the consequences in case of flood-induced dam failure (ICOLD 2014 – Bulletin 154). Risk-based design approaches allow considering different design basis criteria depending on the level of consequences downstream (and sometimes also upstream) in case of dam failure, and not only depending on the dam and reservoir characteristics.

Incremental damages methods, and cost-benefit analysis methods within incremental damages approaches, can stand as optimization tools to find the appropriate design criteria that suit to the actual level of flood risk, particularly for existing dams. For existing dams, cost-benefit analysis can be derived in the estimation of the net present value (NPV) of a spillway upgrade project – benefits are expressed as avoided damages and consequences.

Consideration to risk acceptability or tolerability criteria related to different categories of consequences or damages (loss of life, loss of property, loss of economical assets) can be found in Appendix B of ICOLD Bulletin 154, even though this bulletin is not specifically devoted to safe management of floods. Some national committees have been working (Australia, Canada) or are even currently working (France) on these issues and approaches. Reference to the publications of these committees is indicated for more details.

10.8. CONCLUSIONS

This chapter tried to demonstrate that cost reduction in the design of spillways are possible without impairing the fundamental safety of the project. It has also been shown that economies may be reached from the formulation of alternative spillway arrangement in the overall project configuration.

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