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Spillway design guide

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Foreword

The Environment Agency has prepared this guide in response to the recommendations of the Toddbrook Reservoir Independent Review Report produced following the reservoir spillway incident that occurred on 1 August 2019.

The purpose of the document is to provide nationally recognised guidance on the main design principles, methods and current best practice for the design of new spillways, and upgrading of existing ones. This is to promote consistency of design and ensure the safety of spillway projects in a technically and economically sound way. It could also serve as a reference for the design reviews of existing spillways to look for potential deficiencies in order to reduce the risk of future failures.

It is based on a comprehensive review of the current UK and international best practice in spillway design and lessons learned from recent spillway incidents. However, the guide is not exhaustive and does not preclude the use of other guidance where applicable and where it originates from a reputable and acknowledged source.

The guide was authored by Viktor Pavlov, Principal Civil Engineer at Binnies UK. It was produced under the general review of Jonathan Hinks, Technical Director at HR Wallingford.

Throughout the production of the guide, several representatives of UK design companies, reservoir owners and individual experts have been consulted and they have provided valuable comments and input into the guide:

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P Dickens	Arup
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While every effort has been made in preparing of the guide to present the state-of-the-art in spillway design, it is not a code of practice. Therefore anyone concerned with spillway design must exercise their own knowledge, experience and judgement.

Executive summary

The recent spillway incidents at the Oroville Dam in the USA and at the Toddbrook Reservoir brought attention to the weaknesses of historic spillway designs.

This guide comes in response to the need for a common nationally recognised guidance for spillway design that reflects UK and international best practice.

The guide focuses on the design of cast in place reinforced concrete spillways which have been most typically used recently in the UK. For completeness, reference is made to the relevant guidance for the design of alternative spillway systems.

The guide is not exhaustive. Instead, it presents the main design principles and methods while referring to the latest standards, guidance and publications.

Section 1 presents the general background, purpose and scope of the guide.

Section 2 describes the various functions performed by spillways with regards to reservoir safety, environmental protection, flood mitigation and control of operation. It also introduces the main spillway components, including the inlet structure, the conveyance structure and the energy dissipation structure.

Section 3 discusses the specific requirements and challenges relating to the process of spillway design and optimisation. It highlights the critical importance of adopting a holistic approach to addressing the complex design needs.

Section 4 presents the main design principles and requirements. It briefly discusses the specific design requirements and effects relating to the spillway design flow rate, type, location, inlet structure configuration and chute size. Guidance is provided with regards to the main aspects of the spillway design and analysis.

Section 5 provides guidance with regards to some specific aspects of the derivation of the reservoir flood inflow and its routing. It discusses the catchment delineation, determination of the hydraulic capacity of spillway inlet structures, catchwaters and by-wash channels, and the effects of blockage.

Section 6 provides general overview of the hydraulic design principles and methods with reference to the latest UK and international best practice. Guidance is provided for the most commonly used types of inlet structures, spillway chutes and energy dissipators. Some specific hydraulic effects associated with high velocity flows, such as air entrainment, cavitation and shock waves are also discussed.

Section 7 deals with the main aspects of the structural design of reinforced concrete open channel reservoir spillways. This includes the determination of the applied actions and their combinations, and specific limit state design considerations.

Specific guidance is provided with regards to the determination of the mean and fluctuating dynamic actions, control of cracking and fatigue verification. Typical construction details and arrangements are provided in order to promote consistency of design in accordance with current best practice.

Section 8 provides general guidance relating to the main aspects of the geotechnical design of reinforced concrete open channel reservoir spillways, including the required ground investigations, foundation preparation and specific limit state design considerations. It also provides specific guidance for the design of spillway drainage systems, seepage cut-offs and anchorage systems.

Section 9 gives a general overview of the most commonly used alternatives to cast in place reinforced concrete systems. These include reinforced-grass, stepped-block, roller compacted concrete (RCC) and rock ramps spillways. Reference is made to useful guidance for the design of such systems.

Sections 10, 11 and 12 briefly discuss some aspects of the spillway instrumentation, operation, inspection, maintenance, decommissioning and replacement. Consideration is given in section 11 to some specific health and safety hazards to the spillway operators and the public and how to manage them.

1. Introduction

1.1. Background

The recent spillway incidents at the Oroville dam in the USA and at the Toddbrook reservoir in the UK brought considerable attention to the deficiencies of historic spillway designs which were not always based on best practice and knowledge at the time, or as have subsequently emerged. In addition, there are currently no specific standards or common guidance in the UK ensuring that the design of new or upgraded reservoir spillways is carried out in a consistent manner and in accordance with current best practice.

Acknowledging the significant risk that a potential spillway failure poses to public and reservoir safety, the Toddbrook Reservoir Independent Review Report (Balmforth and others, 2020) recommended among other things, that the Environment Agency commission guidance on spillway design, based on international good practice and lessons learn from incidents in the UK.

This guide forms part of a suite of guidance documents for impounding reservoir spillways produced in response to the recommendations of the Toddbrook Reservoir Independent Review Report.

The suite comprises the following related guides:

- Spillway design guide
- Guide to spillway failure mechanisms
- Spillway examination guide
- Reservoir owner and operator guidance: spillways

1.2. Purpose

This document aims to provide nationally recognised guidance on the main design principles, methods and current best practice for the design of new spillways, and upgrading of existing ones. This is to promote consistency of design and ensure the safety of spillway projects in a technically and economically sound way. It could also serve as a reference for the design reviews of existing spillways to look for potential deficiencies to reduce the risk of future failures.

1.3. Scope

This guide reflects the latest developments and best practice relating to the design of reservoir spillways, including the lessons learnt from recent incidents. It focuses on the main design principles and methods with regards to the critical dam safety aspects rather than providing comprehensive descriptions of the various types of design calculations and analyses typically used. For these, reference is made to the latest UK and international standards, design guidance, and publications.

The guide is not exhaustive and does not preclude the use of other guidance where applicable and where it originates from a reputable and acknowledged source.

The guide focuses on the design of cast in situ smooth reinforced concrete-lined open channel ungated spillways which have been most typically used as principal ‘service’ spillways in recent UK practice. For completeness, reference has been made to the relevant guidance for the design of alternative spillway systems including reinforced-grass, stepped-block, roller compacted concrete (RCC) and rock ramps spillways.

Where possible, the guide provides typical details in order to promote consistency of design and reduce risk across UK reservoir spillway projects.

2. Spillway function and main components

2.1. Function

The main function of the reservoir spillway is to safely discharge excess flows during storm events, while maintaining appropriate flood freeboard. During spillway operation, unacceptable erosion of the downstream receiving watercourse or natural ground should also be prevented.

Additionally, reservoir spillways would normally allow the safe passage and transport of floating debris to the receiving water course. Where this may not be practical, debris retention and removal could be provided instead (refer to section 6.1.5).

Gated spillways can allow flexible reservoir operation and enhanced flood control. In particular, they can play a role in controlling and maximising the reservoir operating storage. They do this by mobilising, for normal use, upper storage elevations above outlet weirs that are required for flood passage and spillway operation. During a flood event, operation of the gate could reduce the maximum reservoir outflow. As such the gate design and operation also directly affects the required spillway capacity and downstream flood risk.

The spillway width plays a role in the process of flood attenuation and therefore affects the maximum reservoir outflow and downstream flood risk in addition to dictating the overflow capacity.

Reservoir spillways can also act either as main (service) spillways, when they provide a continuous or frequent release of water, or as auxiliary or emergency spillways, where they supplement the capacity of the main spillway and only operate infrequently or under unusual or emergency conditions.

2.2. Main components

The main components of spillway structures are summarised in this section. Further details on their types, hydraulic sizing and performance are presented in section 6, Hydraulic design.

2.2.1 Inlet structure

In most applications the inlet structure represents a freely discharging weir which controls the reservoir outflow.

The weir could take different cross-sectional shapes including ogee, trapezoidal, rectangular, triangular (crump weir or other) or sharp-crested. Different configurations in plan also exist - the simplest and most common being the straight weir. Circular weirs are typically provided as the inlet of shaft (bellmouth) spillways. Non-linear in plan weirs, including labyrinth weirs and their ‘piano-key’ variation, provide an increased weir length where limited footprint is available, or the conveyance structure is narrower.

In some applications, the control of the reservoir level and reservoir outflow could be provided by permanent gated structures, temporary collapsible gates or sacrificial embankments referred to as ‘fuse-plugs’.

Where shallow water depth exists upstream of the inlet structure, an approach channel may be provided to reduce headloss and eliminate the risk of erosion and blockage by floating debris.

Submerged inlet structures control the reservoir outflow either by the geometry of their opening or by the geometry or capacity of the downstream conveyance structure.

Examples of different inlet structure arrangements are shown in Figure 2.1:

Figure 2.1: Inlet structure arrangements. a) Piano-key weir with submerged inlet baffles providing control of floating debris, Black Esk Reservoir, UK (Source: J. Ackers). b) Gated weir equipped with radial (segment) gates, Upper Gotvand Dam, Iran (Source: A. Zia). c) Tipping gates forming a labyrinth weir, Kamuzu Dam II, Malawi (Source: J. Hinks). d) Trapezoidal inlet weir followed by a converging transition to the downstream conveyance structure, New Year’s Bridge Dam, UK (Source: V. Pavlov)

2.2.2 Conveyance structure

The conveyance structure is provided to safely convey the flow from the inlet structure and discharge it downstream of the dam. It could take the form of an open channel, sometimes referred to as a 'chute', or a conduit (pipe, tunnel or culvert) where flow may run under free surface or under pressure.

Open channels are most typically sized to fully contain the design flow, while providing sufficient freeboard to accommodate the effects of flow bulking due to air entrainment, shock waves, surface roughness, spray and splash and any modelling uncertainties. In some cases, where it could be demonstrated that there is no risk to the safety of the dam, it may be economical to allow overtopping of the channel during extreme flood events, while providing erosion protection or accepting the maintenance cost incurred. This would require analysis of the respective hydrodynamic flow conditions and duration of overtopping, in order to estimate the extent of the potential erosion and its impact on the spillway structure, and careful design and detailing. The potential impact of such overtopping on the ability of the drainage system to relief uplift should also be considered.

The main types of open channel conveyance structures are:

- smooth reinforced concrete chute – to minimise channel depth and turbulence.
- stepped or macro-roughness chute – to increase energy dissipation along the chute and reduce the energy dissipation requirement at its base.
- baffled chute – to dissipate all or most of the flow energy along the chute and minimise or eliminate the need for energy dissipation at its base.
- natural ground channel, including cut into rock or built over an embankment, with or without erosion protection – to reduce cost where the ground material is sufficiently non-erodible and no risk to the dam safety is present.

Historically in the UK spillway chutes tended to be stepped such that energy is dissipated as the flows passed down them. This especially suited masonry construction. In more recent years and with the advent of concrete chutes, the tendency has been to minimise hydraulic losses on the chute, therefore maximising velocities. This minimises the cross-sectional area of flow and of the chute, therefore minimising chute costs. Energy dissipation is then concentrated at a downstream terminal structure.

Examples of typical conveyance structures (chutes) are shown in Figure 2.2 below:

Figure 2.2: Typical conveyance structures (chutes). a) Smooth straight reinforced concrete chute during operation with roll waves, Llyn Brianne Dam, UK (Source: J. Hinks). b) Stepped stone masonry chute, curved in plan, Ogden Upper Dam, UK (Source: United Utilities). c) Baffled chute and stilling basin, Lee Green Dam, UK (Source: V. Pavlov). d) Cable-tied reinforced grass auxiliary spillway, Llyn Mawr Dam, National Botanic Garden of Wales, UK (Source: J. Hinks)

Pipes, tunnels and culverts are provided to convey flows from shaft spillways. They are designed with smooth lining to increase hydraulic capacity. In some cases, tunnels and culverts may incorporate a large capacity duct admitting air to maintain stable free surface flow conditions, prevent flow instabilities or choking due to air backflow or entrapment and reduce vibration and cavitation effects.

2.2.3 Energy dissipation structure

Dissipation of the residual kinetic energy of the flow exiting the conveyance structure is normally required in order to prevent excessive erosion downstream. Such erosion could potentially undermine the energy dissipator and the entire spillway structure and initiate head-cutting of the embankment or cause unacceptable erosion of the downstream receiving watercourse or natural ground. The requirement for an energy dissipation structure is therefore based on the risk posed by erosion of the downstream channel and its potential effect on the dam.

Depending on the efficiency of energy dissipation provided, further erosion protection, ground re-profiling or slope stabilisation downstream of the energy dissipator may be required.

Like the conveyance structure, where the energy dissipation structure is located a safe distance away from the embankment, it may be sized for a lower flood, therefore allowing sweep-out or overtopping of the structure during extreme flood events, while providing erosion protection or accepting the maintenance cost incurred. This would require detailed analysis of the respective hydrodynamic flow conditions and reliable prediction of the extent of the potential erosion and its impact on the structure.

The most used energy dissipation structure in the UK and worldwide is the hydraulic jump stilling basin which promotes energy dissipation via the enhanced turbulence occurring within the zone of rapid transition (hydraulic jump) from supercritical to subcritical flow. This energy dissipation mechanism is sometimes assisted by adding chute and/or baffle blocks, or by creating a negative step, to promote further dissipation via jet impact and therefore reduce the length of the stilling basin. Further energy dissipation in stilling basins could also be achieved by providing some lateral expansion of the incoming supercritical flow into the stilling basin.

This type of energy dissipation structure is typically used where the receiving watercourse or native ground is prone to erosion and there is not a need for excessive excavation.

Other types of energy dissipation structures include the flip bucket and plunge pool, roller bucket and United States Bureau of Reclemation (USBR) impact-type energy dissipator.

A typical USBR Type III hydraulic jump stilling basin and a ‘flip’ bucket energy dissipator of Russian type design are shown in Figure 2.3:

Figure 2.3: Energy dissipations structures. a) USBR type III stilling basin controlled by an end sill at Holdenwood Dam, UK (Source: United Utilities). b) Russian type flip bucket energy dissipator with a splitter wall, double curvature and diverging in plan at Orto Tokoi Dam, Kyrgyzstan (Source: J. Hinks)

3. Design process

The design of reservoir spillways is a complex multidisciplinary task involving detailed hydrology, hydraulic, geotechnical, structural and sometimes mechanical and electrical calculations and analyses. Often, the design has also to deal with operational, maintenance, environmental, social or other aspects. These are particularly important where upgrading of the spillway capacity at existing reservoirs is required. For such projects, the ongoing operational requirements, flood conditions to be accommodated during the construction phase, and the forms of temporary works and their potential impact on the integrity of the existing structures may be governing factors for the spillway design.

Optimisation of the design should be achieved through comparative analysis of several viable options developed based on the spillway location, type and size. It may also be affected by the dam type (for example, concrete or embankment) or could affect the latter in the case of new reservoirs. The analysis should consider the cost, benefits and risks associated with each option and make a recommendation on the preferred option.

The spillway design normally starts with the establishment of the design and safety check outflow floods. However, the latter is dependent on some of the spillway parameters, such as the level and width of the control section at the inlet structure, as they affect the attenuation of the reservoir inflow. Therefore, defining the spillway design flood, that is the reservoir outflow flood, is an iterative process which requires identification and preliminary design of several spillway configurations and sizes, possibly based on different locations, geological conditions, civil design arrangements and corresponding critical storm durations. This requires close interaction between the different disciplines from the onset of the project.

Due to the high complexity and interdependence of the various design aspects, the effectiveness of the design process depends on the ability of the design team to adopt a holistic approach. This therefore relies on the competence, experience and good communication between all members. In this connection, while the contribution from all design disciplines is important for producing a safe and economic design, central to the design success is the ability of the lead designer to ensure the efficiency of the critically important interface between the various aspects of design.

In order to do this, lead designers should ideally be experienced in the civil, structural, hydrological, hydraulic and geotechnical aspects of spillway design as well as being familiar with the specific types of structures and materials used. This would allow them to strike a balance between often competing and at times opposing design needs in the interests of the safety and economy of the project.

The role of the lead designer is also critical for the successful interaction between the project manager and the design team, ensuring that any risks and opportunities are efficiently managed throughout the project life cycle. During the process of construction, the lead designer can also act as a representative of the design team when dealing with any technical queries and proposals for design changes ensuring they do not have an adverse impact on the spillway performance and its future maintenance.

4. Main design principles

This section stipulates the main design principles and general requirements which should be considered in the design of reservoir spillways.

The recommended methods of design, specific requirements and construction details reflecting the current best practice in the design of reinforced concrete spillways are provided in sections 6 to 8.

References to the recommended guidance for design of alternative spillway systems including reinforced-grass, stepped-block, gabion, RCC, rock ramp and other spillway systems are provided in section 9.

4.1. Design flow rate

The spillway design flow rate should correspond to the 'design flood' specified in table 2.1 of the Engineering Guide 'Floods and Reservoir Safety', 4th Edition (ICE, 2015. The freeboard referred to in section 4.7.1 should be provided at this flood.

The spillway should also be able to pass the so called 'safety check flood' referred to in the ICE Engineering guide, while exhibiting marginally safe performance characteristics for this condition.

Where the spillway structure is located on or adjacent to an embankment dam, it should be ensured that the 'safety check flood' does not pose any unacceptable risk of overtopping or overflowing of the embankment or out-of-channel flow which could result in severe erosion and failure of the embankment. In this respect, the marginally safe performance of the spillway at the 'safety check flood' should be deemed guaranteed where an adequate margin of safety is provided to allow for the uncertainties inherent in the estimation of this flood and the corresponding water levels and channel depths.

In particular, the uncertainty associated with a flood estimate which is not calibrated against adequate local data should be considered. The accuracy of predicting the highest water levels and maximum channel depths and the presence of air bulking, shock waves, surface roughness, spray and splash should also be considered.

In some cases, it may be beneficial to optimise the hydraulic performance of the spillway structure for a flood lower than the design flood. For example, where the energy dissipation structure is located a safe distance away from the embankment, it may be sized for a lower flood. Therefore, in a larger flood, the efficiency of energy dissipation will be reduced, and this could result in overtopping and erosion around and downstream of this structure. However, it may be preferable to the client to deal infrequently with the consequences of such erosion in exchange for the initial saving achieved, provided that there is no risk to the energy dissipation structure, the dam or any adjacent structures (refer to section 2.2.3). Such design would require a scour assessment to be carried out using a widely accepted method. Useful guidance on this can be found in ‘Manual on scour at bridges and other hydraulic structures’, 2nd edition (CIRIA, 2015)

The design should also ensure satisfactory performance of all structures across the full range of flow rates.

4.2. Design arrangement

4.2.1 Spillway location and type

The spillway location and type are governed by economy and safety considerations depending on the type of dam. These are in turn dominated by the topographical and geological conditions on the reservoir banks and at the receiving watercourse. On projects for upgrading the spillway capacity at existing reservoirs, the location and type of the new spillway would also be influenced by the location and condition of the existing spillway, the interfaces of the new structure with the existing dam (including potential differential settlement) and the conditions for flood diversion during construction.

The spillway structure of embankment dams should preferably not be located on or within the embankment due to the inherent risks of differential settlement, leakage or overtopping that could result in excessive deformations, erosion and embankment failure. Where this may be the only viable option for increasing the spillway capacity, such an arrangement should generally be adopted only for auxiliary or emergency spillways, constructed on consolidated embankments. These are designed to operate infrequently, therefore facilitating the regular inspection, maintenance and repair of these structures (refer to Figure 4.1 below).

Figure 4.1: Ogden Upper Reservoir. Main stone masonry spillway on the abutment and auxiliary baffle spillway on the embankment
(Source: V. Pavlov)

The design of such spillways should allow sufficient structural articulation to accommodate the increased risk of differential settlement and should provide a high level of redundancy.

A deviation from this approach should require detailed justification considering the cost, site conditions, constructability, operation, maintenance, reservoir safety and other relevant aspects of the alternative spillway locations on the reservoir abutments or through its banks.

With concrete dams, the preferred location may be on or within the dam.

Shaft spillways, which use a closed conduit conveyance structure (culvert or tunnel), could normally be used as a practical alternative to high capacity open channel spillways where long channels are required due to the height of the dam, As such, they would only be used where there is limited space available. This could be either on the natural ground alongside the dam, at a separate site, such as a saddle of suitable elevation, or, in the case of a concrete dam, over the dam crest, and where suitable geological conditions are present. Often, shaft and conduit spillways are connected to the structure used for diverting floods during construction. However, they present several disadvantages and constraints including:

- increased risk of blockage by floating debris and ice and difficulty in removing it
- need for more frequent inspection for potential blockage, defects and deterioration and potential constraints that this would pose on the reservoir operation
- increased cost of inspection and maintenance due to the access for inspection and any work having to take place under confined spaces regulations and so using a specialised team rather than an individual
- severe limitations in upgrading the spillway capacity if required
- potential for unstable flow operation due to transitions between flow modes or the transport of air under pressure (refer to Section 6.2.3)
- public safety risk

Open channel spillways are normally cheaper to construct (they can be built with the reservoir full or partially full) and easier to inspect for potential defects and deterioration, as well as to maintain. They are therefore predominantly used in projects for upgrading existing spillway capacity.

Gated spillways are rarely used in the UK, but they can allow flexible reservoir operation and enhanced flood control. These benefits could however be offset by the need to provide power supply (except for tipping gates) and to reliably manage the risk of abnormal gate operation through redundancy measures, continuous surveillance and regular maintenance of their mechanical, electrical and control systems.

Examples of spillway design arrangements are shown in Figure 4.2:

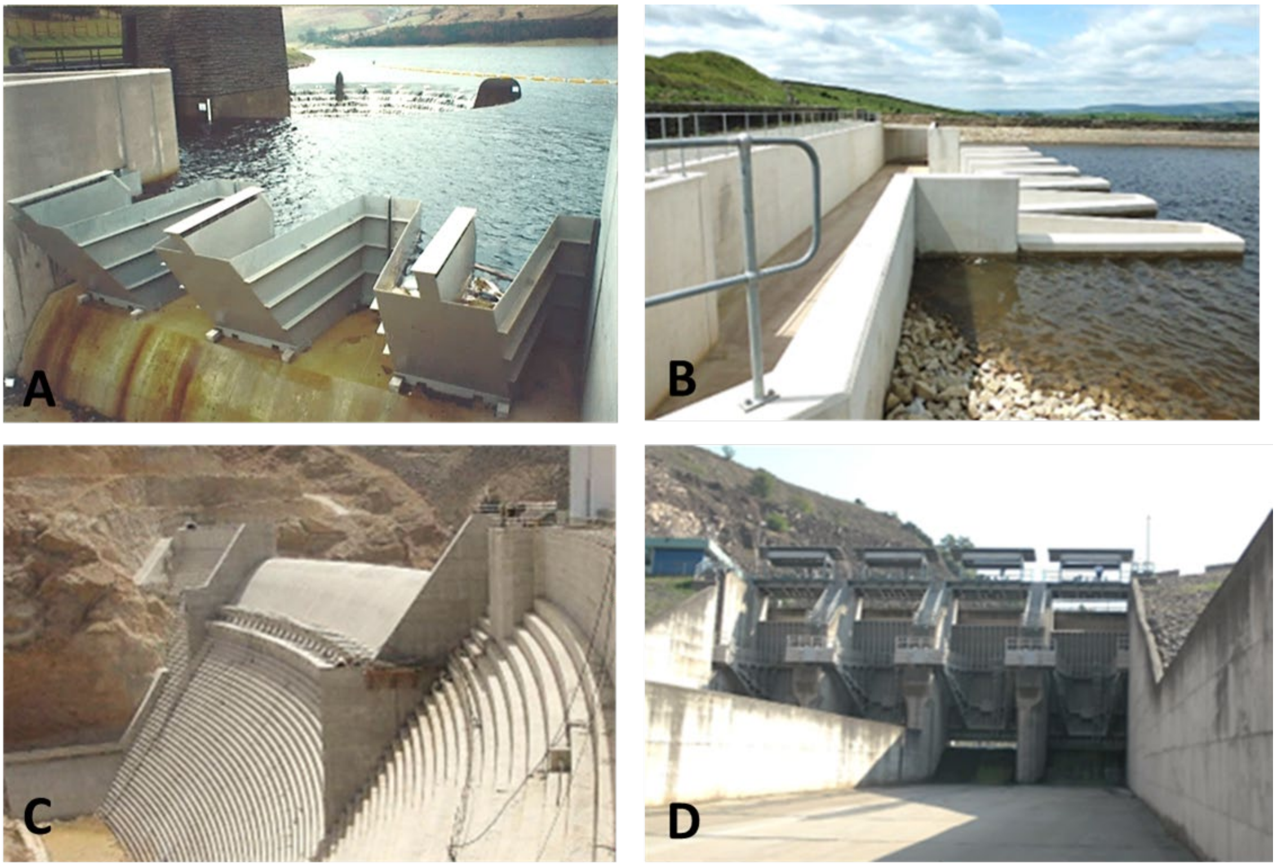

Figure 4.2: Spillway design arrangements. a) Steel tipping gates mounted on top of a concrete weir and masonry shaft spillway at Dovestone Reservoir (Source: Hydroplus). b) Reinforced concrete side labyrinth weir and tumble bay spillway at Swinden I Dam, UK (Source: V. Pavlov). c) RCC spillway with flow deflectors and aerators at Wadi Dayqah Dam (Oman) (Source: Binnies Ltd.). d) Radial (segment) spillway gates and downstream chute at Yuvacik dam, Turkey (Source: J. Hinks)

Designers may be able to influence land use decisions, typically made by reservoir owners, so that the risk to life and property loss during spillway operation or potential abnormal operation could be efficiently mitigated.

4.2.2 Inlet configuration

The levels and configuration of the inlet structure should be designed to meet the following main requirements at both the design and safety check floods:

- provide sufficient flood freeboard to the dam crest or to the water-tight element of the embankment (clay core or other).
- allow safe passage, or retention and removal, of floating debris (refer to section 6.2.5).
- prevent erosion upstream of it, including by reducing approach velocity.
- achieve acceptable flow transitions into the conveyance structure (refer to section 6.2.4).

- avoid significant vibrations due to un-cushioned fall of water from height or hydraulic jumps in the vicinity of the embankment.
- control the reservoir outflow and downstream flooding.
- control the frequency of operation of the auxiliary or emergency spillway of to effectively manage its risk of failure and maintenance cost.
- prevent significant noise generation by reducing the energy dissipation at the inlet as much as practically possible, where it is located close to an inhabited area.

In addition, for projects involving upgrading the spillway capacity of existing reservoirs, there may be a requirement to prevent any increase of downstream flood risk in low-order flood events.

For such projects, the combination of the length and level of the inlet weir (or control section), which govern the acceptable stillwater rise, would often be constrained by the pre-existing levels of the dam crest and top of the impermeable embankment element. Where this is the case, the inlet weir level of service spillways could be set as low as practically possible relative to the embankment crest, in agreement with the client. This is in order to maximise the attenuation of the flood through temporary retention in the reservoir, therefore minimising the reservoir outflow and spillway size. Where lowering of the inlet weir level is unacceptable, the reservoir outflow and spillway size could be reduced by providing a gated spillway while considering the inherent risks of abnormal operation.

Where low-cost or flexible spillway conveyance structure linings are used, such as reinforced-grass, stepped blocks (wedge blocks) or others, the inlet weir level and its width should be set such that the unit discharge (flow rate per metre) or maximum velocity are in accordance with applicable design guidance for the respective materials used. Flow at the approach to such linings should not be supercritical and cross-waves or hydraulic jump conditions should not be present.

4.2.3 Spillway chute size

The size of the conveyance structure is largely dictated by the hydraulic capacity required to pass the predicted reservoir outflow and, for open channels, by the additional need to provide a suitable freeboard to accommodate air bulking, shock waves, surface roughness, spray and splash and any modelling uncertainties in order to prevent overtopping.

However, In the most typical case of an open channel spillway structure, there could be different width/height combinations that would allow the passage of this flow. The selection of the optimal parameters should be based on comparative analysis of a range of spillway chute width/height configurations giving due consideration to all relevant design aspects and implications resulting from variations in chute width.

For example, reducing the spillway chute width could have several beneficial effects, including:

- reduced number of slab joints
- potential improvement of the dissipation of the maximum uplift pressures acting along the centreline on the base slab
- easier access for cleaning and inspection of under-drains, where present
- improved resistance to uplift and increased structural strength/reduced bending moments

However, consideration should also be given to the possible adverse effects of reducing the spillway width, including:

- increase in the reservoir maximum water level, where the chute width may affect the overflow weir length or its submergence
- increased structural depth and water depth resulting in increased bearing pressure, settlement and loading on the side walls
- potential increase in the foundation depth of the collecting channel (or tumble bay) where flow is subcritical. This may be due to the need to prevent the subcritical flow controlling the reservoir level. An increase in the foundation depth might increase uplift pressure, where it is present, and may also require a larger drainage capacity and more costly access and maintenance arrangements. It would also increase the construction cost
- increase in the unit discharge and corresponding hydrodynamic forces acting on the spillway structure and transferred to its foundation
- increase in the length and depth of the energy dissipator
- potential generation of cross waves and resulting dynamic forces where the conveyance structure becomes narrower than the inlet structure
- increased risk of cavitation where such risk is present
- increased risk of blockage by floating debris where the width reduction becomes significant enough to cause blockage
- increase in the freeboard required to accommodate air bulking, surface roughness, wave action, spray, splash

An increase in the spillway width would naturally result in the opposite effects.

Examples of modern wide and narrow chutes are shown in Figure 4.3:

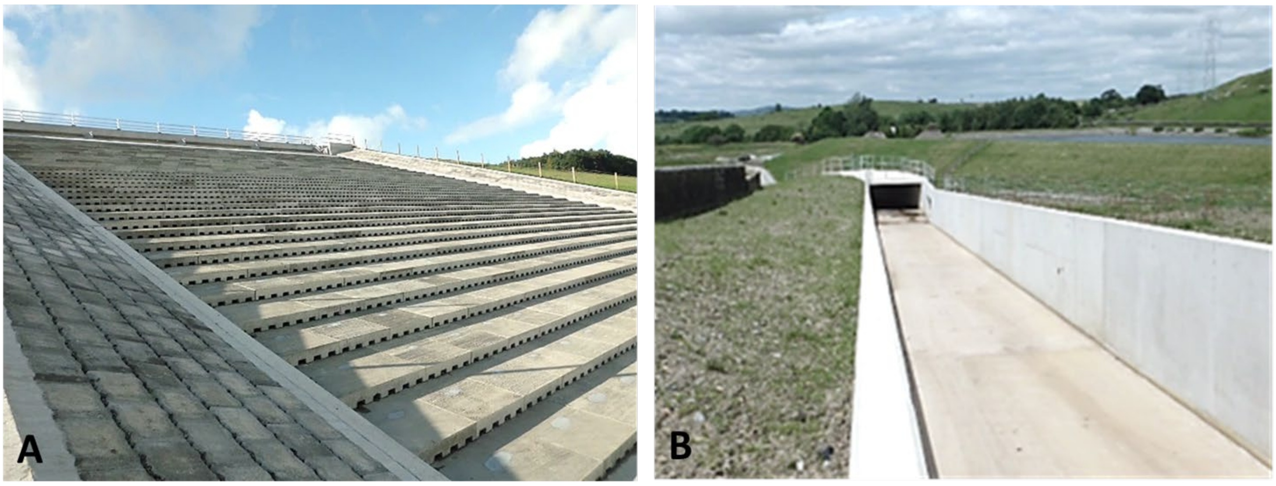

Figure 4.3: Wide and narrow spillway chutes. a) Wide wedge-block chute with inclined side walls formed of solid revetment blocks at Ogden Dam, UK (Source: United Utilities). b) Narrow RC chute with a covered 'roof' section at the change in flow direction at Swinden Dam, UK (Source: V. Pavlov)

4.3. Energy dissipation

The extent of energy dissipation to be provided downstream of the conveyance structure generally depends on the erodibility of the receiving watercourse and the risk of undermining of the spillway structure. This could result in head-cutting of the reservoir embankment or pose a risk to adjacent structures or the environment.

Normally, the safest, but most expensive, method of preventing undue erosion downstream of the conveyance structure is to provide full energy dissipation of the high-velocity supercritical flow within a concrete-lined energy-dissipation structure. In this way, flow will enter the receiving watercourse in subcritical mode with a well-established, close to uniform, velocity profile and will not pose a risk of erosion. However, this would require a very long structure which is usually not considered economical.

Therefore, it would be acceptable that a portion of the high turbulence generated during the process of energy dissipation persists downstream of the energy dissipator. This is subject to flow being subcritical and suitable erosion protection, along with a concrete or piled cut-off, provided to prevent undue erosion and undermining of this structure. Such erosion protection should be sized to resist the maximum predicted bottom velocity and would normally take the form of rip-rap (refer to Figure 6).

Where the energy dissipation structure is founded on competent rock, it may be possible to demonstrate that the structure will not be undermined by erosion of the overlying soil material during the design flood event. In this case, the extent of erosion protection provided could be limited to the risk associated with the infrequent maintenance or reinstatement that the client would be willing to accept in exchange for the initial saving achieved.

In order to achieve subcritical flow conditions downstream of hydraulic jump stilling basins, sufficient water depth and basin length must be provided. The tailwater water depth required for submergence of the hydraulic jump should exceed the subcritical conjugate depth of the incoming supercritical flow by 10 to 15%. It could be achieved via an end sill such that the water depth remains independent of any changes occurring within the receiving watercourse during major storm events.

It is important that the end sill is carefully designed so as not to cause undue erosion downstream of the stilling basin, while creating a ‘reverse hydraulic roller’ ensuring that downstream scoured bed material is driven upstream against the downstream end of the basin, effectively protecting the downstream end from being undermined (Mason, 2004). A suitable drainage arrangement should also be provided to allow the stilling basin to be regularly inspected.

In order to prevent hydraulic jump sweepout, the stilling basin should be long enough to accommodate the hydraulic jump roller length and the backwater curve upstream of the end sill. For preliminary design purposes, a total stilling basin length 25% greater than the predicted hydraulic jump length could be used.

Where the stilling basin is constructed on rock, and the tailwater depth is generated by the receiving watercourse, there may be potential for the stilling basin length to be shortened to about 60% to 80% of the length of the hydraulic jump. However, this would require a robust assessment of the potential impacts and risks to the downstream environment, infrastructure or public.

Where the tailwater depth is controlled by an end sill, reducing the length of the stilling basin below the length of the hydraulic jump could cause a severe sweepout and damages further downstream which may be difficult to predict (refer to Figure 4.4):

Figure 4.4: Stilling basin sweepout resulting from insufficient tailwater depth or stilling basin length (Source: USBR)

The flood for which the energy dissipator is sized could be lower than the spillway design flood (refer to section 4.1).

The final design should be optimised and confirmed by physical model testing, potentially used in conjunction with a computational flow dynamics (CFD) model (refer to Section 4.7.1).

Where, low-cost or flexible spillway conveyance structure linings are used, such as reinforced-grass, stepped blocks or others, careful consideration should be given to the design of the downstream toe arrangement. This is because most linings are not well suited to accommodate the significant turbulence and pressure fluctuations associated with the flow transition at the hydraulic jump where the flow plunges into the tailwaters. Where failure of the conveyance structure lining could pose a risk to the dam, monolithic reinforced concrete linings could be used within the full possible extent of the hydraulic jump at such locations.

Energy dissipators should normally be designed to be freely draining so that their surface, joints and any dissipation features could be regularly inspected. However, it is acknowledged that this may not always be practical.

For details on the hydraulic analysis of energy dissipators, refer to section 6.4.

4.4. Drainage

Spillway drainage should be provided where there is no other more cost effective or more reliable means of eliminating the risk of flotation rather than being provided as a redundancy measure.

Where drainage is provided, it would also mitigate the risks of frost heave, scour of foundation material, and reduction of the shear strength of the foundation material due to partial saturation and dynamic effects as described in section 8.4.1.1.

The drainage system should be designed, as well as adequately and periodically monitored, inspected, cleaned and maintained, to ensure its safe spillway performance under all design and operational situations.

All drainage systems in contact with soil material should be provided with suitably designed graded sand filters to prevent erosion of the underlying soil material while preventing erosion of the filter material into the drainage system. Filters should also have sufficient permeability to pass the required drainage flows without risk of clogging.

Geotextiles or other woven or nonwoven fabrics should not be used instead of mineral filters in any underdrainage systems or in back-of-wall drainage systems which are critical to the safety of the spillway, due to their vulnerability to clogging.

Where spillways are installed over earth-fill embankments, a reliable drainage solution with a high degree of redundancy would be required. This could be achieved by providing a continuous drainage layer consisting of a course granular material and a suitable filter layer. Both layers should be sized and compacted to provide ample drainage capacity and prevent liquefaction and frost heave (refer to section 8.4.1.2. Measures should be taken to prevent concrete or blinding concrete from entering the drainage layer, such as using suitable barriers, low slump class blinding concrete or other measures.

Piped drainage systems should provide ample redundancy to allow for cleaning and maintaining of the system without loss of drainage capacity. This would be particularly important where longitudinal collector drains are provided under wide chutes.

All pipes should be designed with sufficiently large diameters, long radius bends and steep gradients to allow the drainage system design flow rate to be conveyed under free-surface flow conditions with a velocity preventing sedimentation (refer to section 8.4.1.5). They should not be positioned very close to the spillway structure or laid at too shallow a depth, so they do not get damaged by the spillway vibrations and are not at risk of freezing. Further details for drainage pipework design are provided in section 8.4.1.5.

Where the insulation requirements for protecting the underdrainage pipework from freezing cannot be met by the spillway concrete slab thickness and the thickness of the drainage layer above the drainage pipe alone, suitable additional insulation should be provided (refer to section 8.4.1.7).

No drainage outlets should discharge within the spillway channel or energy dissipation structure where high water levels and pressure fluctuations could be present, as they could be transmitted to under the upstream slabs or behind the spillway walls. This could increase the uplift pressure acting on parts of the spillway above the pressure they have been designed for.

The spillway under drainage system should be separated from the spillway walls backfill and hillside drains so that it does not become overwhelmed during a severe rainfall event in line with the spillway design event.

4.5. Anchorage

Where an adequate factor of safety against uplift cannot be achieved by the structure’s self-weight and by any mobilised soil weight and friction alone, and, if the cost of providing a reliable drainage system is prohibitive, anchorage of the spillway structure could be considered to increase the effective weight of the slab by the weight of the foundation material to which the anchor can be tied. Anchorage is typically used with rock foundations.

Passive (bonded) anchors should normally be used but micro-piles could also be used in accordance with BS EN 14199 (2015).

Anchors should be designed with due consideration of all static and dynamic actions and their potential effects. Ample provision for redundancy and uncertainty should be made, including by allowing for the potential stagnation pressures being transmitted from the flow to the underside of the chute slabs.

On-site anchor tests should be carried out to accurately determine the anchor embedment depths and capacities for the given geological conditions.

The design of the anchorage system affects the structural design of the reinforced concrete spillway structure which should be designed to provide sufficient length of embedment of the anchors into the base slab.

Further guidance on the design of anchors is provided in Section 8.4.3.

4.6. Actions

4.6.1. Static actions

The internal hydrostatic action at each location of the spillway structure should be conservatively based on the ‘channel-full’ condition where no overtopping is allowed. Where channel overtopping is allowed by design, the hydrostatic loading should be based on the predicted overtopping water level (refer to section 7.3.1.2 for further details).

The external hydrostatic action should be conservatively based on a water/ground water level taken as the maximum of the following: highest ground level, highest potential level of water ponding or highest tailwater level where the reliability of drainage system cannot be ensured. Where a reliable spillway drainage system is provided, the design external hydrostatic action should be based on the most unfavourable ground water level derived from a seepage model (refer to section 7.3.1.2 for further details).

Earth pressure acting on the side walls should be derived by considering the magnitude of the expected rotation of the wall at its base and its deflection at the top. Where, these are too small to mobilise active or passive pressure, which is typically the case for relatively stiff reinforced concrete walls, pressure at rest should be considered, also allowing for any locked-in pressures due to soil compaction (refer to section 7.3.1.3 for further details).

In some specific cases, wind action and ice action may also need to be considered (refer to section 7.3.1.4).

Where all design measures recommended in this guide are taken to prevent the development of stagnation pressures, but a reliable drainage system is not provided, a residual stagnation pressure equivalent to 20% of the velocity head is recommended to be considered for design purposes in order to allow for any remaining uncertainties.

4.6.2. Dynamic actions

Dynamic actions on the structure caused by mean hydrodynamic forces developing at flow deflection, flow separation and direct impact, as well as by flow-induced vibrations, should be considered.

The dynamic loading due to flow-induced vibrations could be modelled as a spectrum of the p'_{rms} value of the wall pressure fluctuations taken as a percentage of the velocity head, while allowing for their spatial correlation (refer to section 7.3.2.3 for further details).

In some cases, other dynamic actions including vibrations due to wind turbulence, wave action, dynamic impact from floating debris and/or ice would also need to be considered (refer to section 7.3.2.5 for further details). In rare cases, seismic actions may also be relevant (refer to section 7.3.2.4).

4.7. Analysis

4.7.1. Hydraulic capacity

The spillway hydraulic capacity should be established via analysis of the non-uniform (gradually or rapidly varied) flow conditions, considering the effects of air bulking due to air entrainment, surface roughness, splash and spray. The analysis should also take into consideration the effects of cross-waves and standing waves developing at changes in flow direction.

The hydraulic capacity analyses shall consider the design flood, safety check flood and spillway sizing and optimisation flood if lower than the design flood.

The spillway hydraulic capacity should allow for the provision of adequate freeboard to the top of the impermeable embankment element, the embankment crest and the top of the conveyance structure and energy dissipator side walls.

In particular, the spillway should also be able to pass the 'safety check flood' while providing a safe flood freeboard to accommodate the wave surcharge and future embankment settlement. Depending on the fetch and orientation of the reservoir, and considering climate change and its possible effects, there may be a potential for significant spray due to breaking waves to be blown over the embankment crest and downstream shoulder as shown in Figure 4.5:

Figure 4.5: Spray due to breaking waves blown over the embankment and downstream shoulder (Source: South Staffordshire Water)

Where this could pose a risk to the embankment, an increased flood freeboard or improvement of the upstream face to reduce run-up or absorb the wave energy should be considered.

The freeboard provided above the non-aerated (black water) calculated flow depth along open spillway conveyance structures operating in supercritical mode, including the effect of cross-waves, should not be less than 600mm (refer to section 6.3.2.1). It should also allow for air bulking, surface roughness, spray and splash and any modelling uncertainties. The freeboard provided at energy dissipators should depend on the supercritical approach flow velocity and the subcritical conjugate depth (refer to section 6.4). This freeboard should be provided at the ‘design flood’. Where practical, and acceptable to the reservoir owner, for example for newly built spillways, this freeboard could be provided at the ‘safety check flood’ instead in order to provide resilience for potential future increase in flood estimation and climate change, for a relatively small increase in the initial capital cost.

Where gated inlet structures are proposed, analysis of the transient flow conditions developing as a result of gate operation should be carried out. In this respect, particular consideration should be given to the potential adverse effects of a sudden release of water by tipping gates or similar structures in the absence of any tailwater depth. Such flow conditions have the potential to generate high transient hydrodynamic forces, spilling out of channel or sweep-out of the hydraulic jump.

The hydraulic capacity of reinforced grass conveyance structures is dominated by the flow velocity and duration. In order to establish the most critical operating conditions for such structures, a range of velocity/duration scenarios should be developed by varying the storm duration parameter in the flood model. Therefore, maximising the storm duration will maximise the duration of operation of such spillways while reducing the maximum flow rate and corresponding velocity and vice versa.

The numerical methods of hydraulic analysis present some limitations when dealing with complex three-dimensional, multi-phase or highly turbulent flows with vortices. Therefore, physical models are often used as a reliable and cost-effective method of ascertaining the hydraulic behaviour of spillway structures where such flow patterns are present.

Examples of physical models of spillway structures are shown in Figure 4.6:

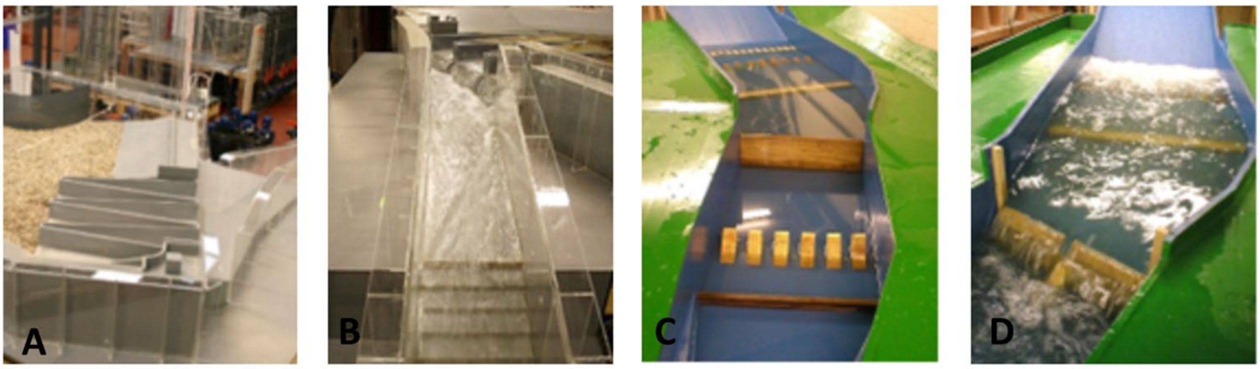

Figure 4.6: Physical models of spillway structures. a) Physical model of a labyrinth weir and stepped cute at Penwhirn Dam Spilway, UK (Source: Hydrotec Consultants Ltd.). b) Physical model of smooth chute and USBR type III stilling basin at Holdenwood Dam, UK (Source: CRM Ltd.)

However, physical models also have some limitations related to their size scaling effects, conflicting Froude and Reynolds similarity requirements and uncertainty related to the accuracy of measurement.

Therefore, acknowledging the limitations of both physical and numerical modelling, the current trend for modelling complex hydraulic structures and flow patterns is to utilise both a numerical and physical model in series or in parallel during a study, also known as ‘composite modelling’ (ICOLD, 2016). This modelling technique allows the strengths of both physical and numerical modelling to be combined in order to improve modelling accuracy and reduce uncertainty.

Composite modelling would normally be required where the risk associated with the inherent modelling uncertainties is high or where there is a high potential for cost savings to be achieved via improved modelling accuracy.

4.7.2. Cavitation and forced aeration

High velocity flows within conveyance structures and energy dissipators could potentially cause cavitation which could be damaging to the lining of these structures. This potential may generally exist on smooth spillway chutes where cross-sectional averaged velocities (afterwards simply referred to as ‘velocities’) approach 30m/s and on stepped chutes where velocities approach 15 m/s (ICOLD, 2016).

However, at stilling basins, where water depth is much greater while the air concentration near the bottom is much lower, cavitation could occur where the velocity exceeds 20m/s (Hager and others, 2020).

Furthermore, according to USBR (2014), where block-outs, exposed expansion joints or other flow surface irregularities are present, cavitation problems could occur at velocities as low as 15 m/s.

Therefore, for every spillway project the cavitation potential should be assessed and, if required, suitable measures including air entrainment, improved surface finishing and streamlining or expansion joint or block-out covers should be considered.

For example, with high velocity flows, it may be desirable to have aeration steps at intervals down the chute. There should typically be a local jump in the flow over the channel bringing in air from the sides of the chute. Further details on forced aeration of chutes are provided in section 6.

The design of aerators to prevent cavitation should not be overconservative as this could result in excessive air-bulking and increased risk of chute overtopping. To further mitigate this risk, aerator facilities should be designed to allow subsequent control and adjustment of the air flow. Figure 4.5 shows a typical aeration arrangement.

Figure 4.5: Vertical steps and aeration orifices in a smooth reinforced concrete chute at Upper Gotvand, Iran (Source: A. Zia)

Where a cavitation potential exists, chute or baffle blocks should not normally be used in stilling basins.

The cavitation potential at high head weirs should also be investigated and mitigated accordingly.

For details on the assessment of the cavitation potential and design of mitigation measures refer to section 6.3.2.3.

4.7.3. Uplift

As stated in section 5.4, stability against uplift should preferably be achieved by providing sufficient structural weight, including any mobilised soil weight and side friction or reliable drainage. Where this is not practically possible or economically viable, suitable anchorage should be provided.

When analysing the spillway structure, resistance to uplift all critical external ground water levels and design situations should be considered. In particular, the uplift analysis of stilling basins should consider the effects of maximum tailwater level, stilling basin sweep-out and negative pressure fluctuations occurring within the hydraulic jump as detailed in section 8.3.3.

The flotation analysis for the permanent design case should ignore the favourable effect of any drainage systems provided which have not been designed to be reliably monitored, inspected, cleaned and maintained at all times.

The contribution of side friction due to lateral earth pressure acting along the structure perimeter could be considered for the permanent situation while the resistance provided by any anchor forces could be considered for both temporary and permanent situations.

4.7.4. Strength

The strength of the spillway structures should be analysed for all plausible static and dynamic loading conditions and design situations (refer to sections 7.3 and 7.4.1).

The analysis should allow, among other things, for the conditions at the supports, soil-structure interaction and dampening effects during dynamic loading conditions. This could be particularly important for relatively thin slabs in the spillway chute and stilling basin.

The strength analysis should also consider the possible fatigue effects due to prolonged flow-induced vibrations and dynamic actions (refer to section 7.4.2.3).

4.7.5. Cracking

All elements of the spillway structure should be designed to effectively control cracking due to static and dynamic structural loading and due to restraint to contraction and plastic settlement.

The effect of the surface temperature of the spillway chute rising due to solar radiation and then suddenly being reduced due to a flood of much colder water should be allowed for.

The analysis of cracking should also consider the potential fatigue effects due to prolonged flow-induced vibrations which could cause the opening and further growth of cracks as well as deterioration of the bond strength at the anchor-grout interface of passive anchors.

The design should meet the requirements of Tightness Class 1 in accordance with Eurocode 2, Part 3, while considering the maximum internal and external hydrostatic pressures acting on members of the structure. Where baffles, blocks, sills or other features are placed on a spillway chute or within an energy dissipator to dissipate energy, a stagnation pressure corresponding to twice the velocity head should be considered in controlling cracking around such features (USBR, 1987).

The concrete tensile strength used in the calculations should reflect the time when constant load is first applied.

The design should consider whether early-age and long-term thermal and shrinkage cracks could be additional to cracks due to flexure and direct tension.

Further guidance on the control of cracking is provided in section 7.4.3.1.

4.7.6. Liquefaction

Much like seismic action, flow-induced vibrations acting on the spillway structure could cause liquefaction of the foundation, filter or backfill materials in cases where they could become partially or fully saturated. This would result in a reduction of the shear strength of such materials, which could cause bearing pressure or sliding failure as well as excessive settlements and a significant increase in the uplift pressure acting on the structure. Such conditions could occur where there is a risk that:

- structures and their surrounding ground could be subjected to high water levels for a prolonged period of time such as energy dissipators built within the receiving watercourse
- spillway chutes are cut deep into the ground and the drainage system could become overwhelmed or blocked
- undrained spillway chutes are built over the embankment and significant seepage could develop through or above the impermeable element of the embankment
- spillway chutes could develop cracks or water-stops at joints could be damaged and allow water to enter from the chute into the foundation

Where such risks are present, an evaluation of the liquefaction susceptibility should be carried out for all soil and filter materials at the foundation or around the spillway structure, considering their thickness and specified degree of compaction.

Where necessary, measures for soil improvement should be taken to eliminate the risk of liquefaction. Such measures may include, but not be limited to, improved drainage, replacement of any soil material susceptible to liquefaction with the appropriate amount of gravel material, increased thickness and higher degree of compaction of sand filter material or other materials (refer to sections 8.3.5 and 8.4.1.6).

4.8. Construction details

4.8.1. Joints

Movement and construction joints play an important role in the way spillway structures behave when subjected to static and dynamic loading and restrained deformations.

In general, the selection of the type and spacing between movement joints should allow optimisation of the design of the structural elements in order for them to resist the internal forces and moments resulting from external loading and thermal effects at the lowest cost. In particular, joints could be used as a cost-efficient alternative to increasing the element structural strength by providing:

- longitudinal displacement to accommodate thermal expansion and contraction and contraction due to shrinkage.
- rotation to accommodate limited differential ground settlements.

Spillway movement joints should be designed to control cracking which could lead to corrosion of the reinforcement and may allow water to enter from the chute into the foundation.

All movement joints should include water-stops. Where expansion joints are provided, the water-stops should be protected by non-absorbent compressible fillers and suitable surface sealant.

All contraction joints should be complete contraction joints (without continuous reinforcement across the joint).

Contraction joints spaced at no more than 12.0m are normally used in spillway structures on soil foundation (USBR, 2014). On rock foundations, spacing between the joints may need to be reduced to mitigate the risk of cracking due to restrained shrinkage and thermal contraction.

Contraction joints should preferably be provided with foundation keys (refer to section 4.8.3) and should be doweled to ensure shear transfer along the plane of the joint in order to prevent vertical offsets developing.

The number of expansion joints should be minimised as much as possible. This is because the high velocity flow has the potential to damage and remove the joint sealant and compressible filler, which would then need replacing. Damaged and unrepaired expansion joints could lead to vegetation growth, damage to the water-stops, increased flow disturbance and, with high velocities, could cause cavitation. Where expansion joints are provided, the initial gap should be 20 to 25mm in order to allow replacement of the compressible filler. These joints should be highlighted to the clients as requiring thorough monitoring, maintenance and filler replacement if damaged. If no expansion joints are provided, the concrete strength and reinforcement provided should be such that concrete crushing due to thermal expansion cannot occur. In such cases, 50mm deep and 10mm wide block-out groves should be provided at each contraction joint to prevent delamination at the joint near the slab surface due to solar radiation-induced surface expansion of concrete (refer to section 7.5.2.4).

Where isolation joints are required to prevent the transfer of load from the spillway to adjacent pre-existing structures, the selected method and materials used to achieve water-tightness of the joint should consider the magnitude of possible differential movements.

Where high velocities are expected, it used to be considered good practice to offset locally the downstream slab at transverse joints by 12 mm relative to the upstream slab. However, this is thought to have caused cavitation at several spillways and it is now recommended that such joints be cast flush.

Reinforcement should be continuous across construction joints. Vertical construction joints should generally be avoided within the spillway slabs. A suitable alternative to vertical construction joints in spillways slabs, where moment transfer is required, is to use control joints (refer to section 7.5.2.2). These joints provide reliable water-tightness and better mitigation of the risk of local corrosion, spalling and delamination, while being easier to construct with the high quality of workmanship required for them to be effective.

Where baffles, blocks, sills or other features are placed on a spillway chute or within an energy dissipator to dissipate energy, it is inevitable that locally high stagnation pressures will develop at those features under flow. Ideally, no floor or wall joints should be provided in such areas. Where they are provided, it is important to detail them and all surrounding local areas, so as to avoid the injection of local high pressures into the underside of the chute and also to avoid any other undesirable consequences (refer to sections 4.7.5 and 4.8.4).

The design and execution of all joints is critical for the safety and durability of the spillway structure and therefore should be closely supervised by the construction engineer.

4.8.2. Seepage cut-off walls

Seepage cut-off walls could be provided at suitable locations under the spillway structure in order to control seepage flow and velocity, uplift pressure and/or to prevent the development of large concentrated seepage flow paths having the potential to cause internal erosion.

To do so, cut-off walls should be designed to either increase the length of the seepage path under the spillway structure or tie it into less permeable strata, therefore increasing the seepage energy dissipation and reducing the seepage flow.

In order for cut-off walls to reduce uplift pressure, they should be used in conjunction with a suitable drainage and filter system. When doing so, it should be ensured that this arrangement would not cause an unacceptable increase of the seepage flow gradient and therefore would not pose a risk of internal erosion (refer to section 8.4.2).

A cut-off wall element should be provided at the upstream end of the spillway structure, where it would normally be tied into the impermeable embankment element (clay core or other).

Where there is a risk of concentrated seepage flow paths developing, cut-off walls may also be used along the spillway chute, typically under the transverse joints (refer to section 8.4.2 for typical details) and under the side walls.

Where the spillway structure is crossing the embankment clay core, cracks or gaps may form at the interface between the wall and the embankment due to settlement of the embankment material. Such cracks and gaps could initiate a concentrated leak erosion. To mitigate the risk of this happening, the spillway side walls could be provided with a uniform slope “flatter than about 0.25H:1V” (Fell and others, 2015).

4.8.3. Foundations keys

Foundation keys should be provided where possible in order to prevent:

- differential vertical movement of the spillway slabs across the spillway joints due to uplift or settlement, therefore reducing the risk of stagnation pressures developing
- joint opening, therefore preventing the transmission of stagnation pressures and flows under the spillway slabs

Foundation keys could be provided for steep slopes (USBR, 2014) in order to complement the resistance of the spillway chute to sliding, while making allowance for any displacements required to mobilise the passive soil resistance.

Where the spillway is constructed on a relatively steep slope and expansion joints are provided, the foundation keys could be extended in depth in order to be keyed into the in-situ foundation. This would minimise the downhill creep, caused by contraction and expansion due to temperature change (refer to section 7.3.1.4), therefore eliminating the risk of excessive opening or closure of the expansion joints. Failure to do so could cause damage to the water-stop of the upstream expansion joint and spalling of the downstream joint.

Deep foundation keys could also be provided at the downstream end of the energy dissipation structure to prevent undermining of the spillway structure where there is a potential for erosion downstream of it. If possible, they should penetrate to a sufficient depth into competent bedrock. Where this may not be practical, they should extend to a depth below the maximum estimated depth of erosion at the safety check flood. Care should be taken to ensure that such deep foundation keys do not interfere with the drainage system, for example, by taking any drainage pipework through them or by providing suitable filtered drainage holes.

Foundation keys should preferably be provided at each movement joint for spillways built on soil foundations, as well as on rock foundations where there is a potential for differential settlement. They should be suitably reinforced to resist the external loads without cracking and should be formed where installed in soil foundations.

Where foundation keys, by virtue of their depth, have the potential to cut-off seepage flows, they should also satisfy the requirements of section 8.5.

4.8.4. Water-stops

Water-stops provide a physical barrier to prevent water from the spillway passing into the foundation at movement joints, including due to stagnation pressures. Water-stops would also prevent internal erosion of the spillway foundation if there is a potential for such joints to provide unfiltered release of seepage flow.

Where movement joints are designed in accordance with this guidance, water-stops should be sized to resist a stagnation pressure corresponding to at least half the velocity head in order to allow for any construction tolerances and spalling effects (refer to section 7.5.2.5).

Where movement joints cannot be avoided around baffles, blocks, sills or other features placed on a spillway chute or within an energy dissipator to dissipate energy, their water-stops should be designed to resist a stagnation pressure corresponding to the full velocity head.

Water-stops should be suitable for resisting the expected dynamic loading and structural displacements.

Water-stops should be installed in accordance with manufacturer’s instructions.

Further guidance for the design and installation of water-stops is provided in section 7.5.2.5.

5. Flood estimation

5.1. General

This section provides guidance with regards to some specific aspects of deriving the reservoir flood inflow and its routing relating to catchment delineation, hydraulic capacity of spillway inlet structures, catchwaters and by-wash channels, and the effects of blockage.

It also highlights the importance of deriving an outflow/frequency curve and, in some cases, establishing the stage/discharge curve at the spillway outlet for design purposes.

It is outside the scope of this guide to detail the general methods and techniques for deriving the reservoir flood inflow. These should be based on the procedures for rainfall and flood frequency estimation provided in the latest versions of the Flood Estimation Handbook (FEH) and the Revitalised Flood Hydrograph Model, considering the recommendations made in the Engineering Guide ‘Floods and reservoir safety 4th edition’ guide (ICE, 2015) and all other relevant guidance.

5.2. Catchment delineation

5.2.1. General

Catchment areas in the UK can be derived from the digital data provided by the FEH Web Service. However, the catchment boundaries based on the digital terrain model are inevitably approximate due to low relief, braided rivers or catchments with diverted natural drainage paths.

Therefore, digital catchment data should be checked against and complemented, if necessary, by data from OS maps (UUGIS) on which some unusual catchment features could be identified.

Typically, catchment areas could be more accurately derived based upon 1:25 000 OS topographic maps and LiDAR data. However, these too could be problematic and often an additional site survey of specific terrain features may be necessary to obtain accurate catchment delineation. In particular, the presence of catchwaters, by-wash channels, open channel or piped water transfers, including any pumped inflows, should be carefully established.

Specific consideration should be given to the challenges related to the delineation of urban and small catchments.

5.2.2. Urban catchments

The presence of a relatively high proportion of urbanised areas within the catchment creates a particular challenge for obtaining an accurate catchment delineation. This is due to several effects, including:

- gentle ground surface gradients and lack of (or missing) contours on 1:25 000 OS maps
- gravity and pumped surface water and combined sewer systems could remove part of the surface run-off from the direct catchment or could bring run-off from indirect catchments. In this respect, data from utility companies would need to be obtained and analysed in order to determine the maximum rate of contribution of such systems during relatively low return period flood events, typically less than 1 in 100-year flood
- road and railway embankments could retain water and attenuate surface run-off. Therefore, the potential for failure of such embankments due to overtopping or otherwise would need to be considered
- culverts and underpasses under road and railway embankments control the above attenuation effects. Since they may be small, the potential for them to become blocked by floating debris would need to be considered

5.2.3. Small catchments

Small catchment areas could sometimes be significantly affected by the presence of man-made features such as roads and boundary walls, which could act as ‘catchwaters’ to either increase or reduce the effective catchment area (Pavlov, 2015).

Examples of roads acting as ‘catchwaters’ typically include roads with kerbs, raised edges or boundary walls. Also, roads laid on steep consistently downward-sloping terrain or presenting a convex large radius curve, where the latter is designed with a transverse gradient towards its centre could also act as catchwaters.

Where such features could increase the effective catchment area, their effect should be considered. However, if they could potentially divert the natural surface run-off away from the subject site, a risk-based judgement would need to be made as to whether these features will be permanently present or whether there would be a risk for them to be removed at some point in time.

5.3. Flood attenuation and capacity of spillway inlet structure

The spillway inlet structure capacity is normally governed by its control section but, in some cases, it may be further influenced by any upstream head loss and submergence of weirs or orifices. In this respect, rounded wing walls upstream of the weir will give a higher coefficient of discharge than straight walls. Also, bridge piers will reduce the spillway inlet structure capacity to different degrees depending on their location relative to the control section and their shape in plan.

It also affects the attenuation of the reservoir inflow and, therefore, the outflow flood passing through the spillway at different return periods. It should be noted that a conservative estimate of the spillway inlet structure capacity would not be conservative in assessing the maximum outflow flood discharge.

Often, the flow control at the inlet is achieved via a freely discharging weir of standard (well-studied) type. In such cases, the head-discharge curve could be accurately predicted based on the empirical formulae available in the technical literature (refer to section 6). Very often, such predictions would be accurate enough to derive the reservoir outflow.

Where a relatively small-scale physical model is used to analyse the overall hydraulic performance of the spillway, it should be borne in mind that it may not be as accurate as the empirical methods with regards to the capacity of freely discharging weirs for several reasons, namely:

- the empirical methods are based on extensive large-scale and prototype tests and measurements which reduce or eliminate scale effects
- small physical models are prone to additional inaccuracies related to model-built tolerances and potential errors
- the flow rate and level measurements in a small scale are subject to greater inaccuracies

Therefore, where physical models are used for freely discharging weirs, the physical model test results should always be compared with the results from empirical calculations and any discrepancies should be highlighted to the modelling laboratory for reconciliation. Where the weir structure being modelled is not of standard type, a separate larger physical model could be built to accurately predict its head-discharge curve. Such non-standard weir types, mainly triangular and trapezoidal, with or without rounding, are typical for some historical spillways as documented in the CIRIA Report 34.

Similarly, where head loss upstream of the control section could be significant, its magnitude may be more accurately predicted by applying the available theoretical methods and therefore should always be checked against the physical model results.

5.4. Catchwater and by-wash channel capacity

The hydraulic capacity of catchwaters and by-wash channels is governed by several factors relating to the channel geometry and roughness. However, it also depends on the magnitude of the inflow fed at their upstream end, as well as on any concentrated or ‘uniformly’ distributed lateral inflows and their location.

Therefore, maximum flow discharged at the downstream end of open channels characterised by subcritical flow conditions is normally highly dependent on the lateral inflow towards their end. Thus, the maximum flow at their downstream end could be significantly underestimated if the normal depth concept is applied where long ‘mild’ gradient channels are involved. This is illustrated by the dashed line between locations A and C in Figure 5.1 (where the water depth at location A approaches the normal depth) which neglects the lateral inflows contributed by the channel-own catchment (Pavlov, 2015).

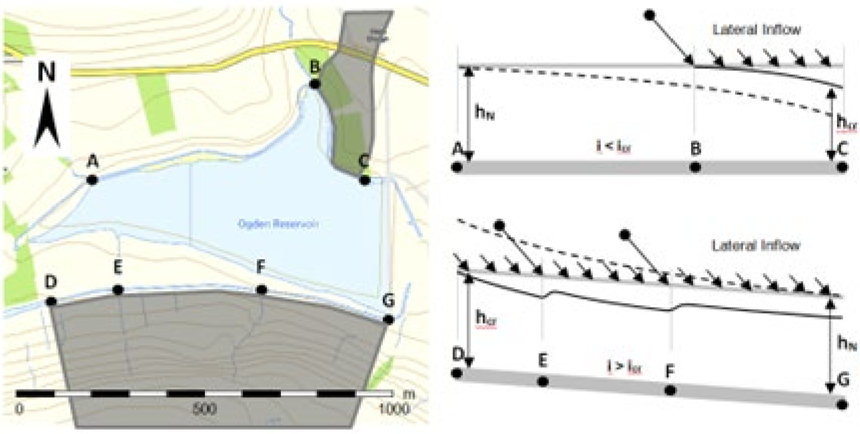

Figure 5.1: Flow surfaces in catchwater and by-wash channels receiving lateral inflow (Pavlov, 2015)

Where such channels are laid at a steep gradient, which is rather uncommon, that promotes supercritical flow conditions, assessing their hydraulic capacity based on a normal depth calculation would be conservative. However, it could potentially significantly overestimate the maximum flow discharged at the downstream end where the lateral flow along the channel is relatively low. This is illustrated in Figure 5.1, where the dashed line between locations D and G represents the imaginary drawdown curve due to the gradually varied flow that would need to be accommodated within the channel if a normal depth equal to the channel depth was to be approached at its downstream end (Pavlov, 2015).

Since channels are normally built with a constant depth, they would be overtopped at their upstream end. Therefore, the maximum flow that could enter the channel will be that corresponding to a critical depth matching the channel depth as illustrated on the same figure by the solid black line, depicting the real gradually varied flow water surface.

After the inlet, flow starts accelerating down the ‘steep’ channel, therefore water depth reduces, which would allow the channel to receive additional flows. When ‘uniformly’ distributed lateral inflows or relatively small concentrated flows are added (as shown at locations E and F), they would normally be carried forward by the high velocity supercritical flow, causing a gradual increase of the water surface at locations E and F and progressively raising the water depth. If the channel’s own catchment area (greyed area with solid line contour) could contribute a sufficient lateral inflow, the channel could flow full at its downstream end, which could then approach the normal depth channel capacity.

However, a relatively significant concentrated inflow could have the potential to break the supercritical mode. This would need to be analysed considering factors such as the flow magnitudes, velocities, and channel configuration at the junction.

Since the capacity of these channels could have a significant impact on deriving the reservoir inflow, every effort should be made to ensure it is reliably represented in flood estimation models based on analysis of the respective gradually varied flow.

5.5. Flood frequency curve and flood hydrographs

Deriving a flood frequency curve and corresponding flood hydrographs can benefit the spillway design in many ways, in particular:

- where a reinforced grass or other low-cost or flexible auxiliary spillway is designed, they would inform the decision on the level and width of its inlet weir in order to achieve acceptable frequency and duration of operation
- where embankment overtopping is considered, they would allow the risk of erosion for different overtopping velocity/durations scenarios to be established
- they would allow optimisation of the design of the energy dissipator with regards to the acceptable frequency of overtopping or downstream erosion (refer to section 4.1)

In order for the flood frequency curve and flood hydrographs to adequately inform the critical storm duration of an overtopping event, they should take the form of a series of curves based on a range of storm durations comprised between the plausible shortest and longest storm durations.

It should be noted that the critical storm duration maximising the flow rate of the reservoir design outflow flood, or the safety check outflow flood, would not be the same as that achieving the critical combination of velocity and duration of overtopping. For example, the maximum probable maximum flood (PMF) flow rate used for checking the available freeboard may have a much shorter duration than the PMF which achieves the worst-case combination of velocity (flow rate) and duration for the purposes of assessing the risk of embankment overtopping.

5.6. Receiving watercourse stage-discharge curve

In some cases, the flood model built for the purposes of deriving the reservoir outflow could be extended downstream of the reservoir in order to generate a stage-discharge curve of the receiving watercourse at the exit from the spillway.

This would be mainly beneficial where the receiving watercourse is relatively flat and receives substantial inflows downstream of the reservoir, which could result in high tailwater levels around the energy dissipator or the embankment toe. Such tailwater levels could cause saturation of the foundation of the energy dissipator and therefore would increase the uplift forces and the potential for soil liquefaction due to the inherent dynamic effects (Refer to section 8.3.3).

Such analysis should consider the potential for any bridges or culverts to become blocked by floating debris discharged by the spillway.

5.7. Spillway blockage

Spillway inlet structures which have relatively small clear openings may be prone to becoming blocked by floating debris. This could typically be the case where small service spillways are retained to take a proportion of the maximum reservoir outflow, the remainder being discharged over a new auxiliary spillway.

Also, screens are sometimes used in front of spillways of reservoirs operated by fishing clubs to prevent fish escaping. These screens could be blocked by autumn leaves or algae blooms.

In these cases, unless measures to prevent these blockages are taken in accordance with section 6.2.5, the reservoir outflow may need to be derived assuming partial or even full blockage of the inlet of the respective spillway.

In this respect, model tests and operational experience indicate that once a partial blockage of a spillway by a large piece of floating debris occurred, this would start retaining smaller branches leaves and other debris, and would have the potential to form a complete blockage (ICOLD, 2021).

The potential for this happening should be assessed, considering the possible volume of floating debris entering the reservoir, their accumulation and rate of transport to the spillway, and the availability and capacity of any systems for removing floating debris. (ICOLD, 2021).

Page Break

6. Hydraulic design

6.1. General

The hydraulic design of reservoir spillways should be carried out in accordance with the latest international guidance and engineering practice. It is therefore important that design engineers should continuously keep abreast of the latest developments and publications on this subject made by renowned national and international organisations.

Unlike the geotechnical and structural design, the hydraulic design is largely non-standardised and therefore relies on the design engineer's thorough understanding of the respective theoretical design principles and their ability to apply engineering judgement when faced with uncertainty and complexity in dealing with engineering problems.

Therefore, unlike sections 7 and 8, which only provide complementary guidance, not contradicting the applicable Eurocodes and British Standards, this section should in principle provide design engineers with full up to date design guidance on the hydraulic design principles and calculation methods to be used.

Since providing such comprehensive guidance in this document is not deemed practical, this section largely refers to the recently published book 'Hydraulic Engineering of Dams' (2020) which includes extensive coverage of practically all aspects of the hydraulic design of reservoir spillways.

Reference is also made to some additional guidance documents that either complement the above book, with regards to some specific hydraulic design aspects, or have been published at a later date.

6.2. Inlet structure

6.2.1. General

The inlet structure generally comprises an approach channel, overflow structure and transition to the conveyance structure. The hydraulic design of the inlet structure is primarily concerned with achieving the minimum required flood freeboard and suitable hydraulic conditions. In addition, the inlet structure should normally allow free passage of floating debris.

6.2.2. Approach channel

The approach channel achieves the transition between the reservoir and the overflow structure. Generally, it has to promote minimum headloss and uniform distribution of flow over the weir.

Where the natural ground upstream of the inlet structure is too high relative to the weir crest, the resulting high velocity at the approach to the weir would increase the headloss arising from hydraulic resistance and could cause erosion of the channel bed. In some cases, it may also have the potential to form a flow control section upstream of the overflow weir. To mitigate these effects, the local ground could be re-profiled to achieve gradual transition curves and low approach velocities, typically less than 3 m/s (Khatsuria, 2005) and suitable erosion protection could be provided.

In addition, at high Froude numbers of the approach flow, there is a risk that floating debris could be drawn down therefore reducing the overflow capacity and promoting blockage at the inlet (refer to section 6.2.5).

6.2.3. Overflow structure

There are 3 main types of overflow structures depending on their alignment relative to the conveyance structure (Hager and others, 2020):

- frontal overflow
- side-channel overflow
- shaft overflow

Depending on its configuration in plan, the overflow structure could take the form of a straight crest overflow, polygonal crest overflow, including labyrinth and piano-key, and curved crest overflow (including circular).

Finally, depending on its cross-sectional shape (profile), the overflow weir could generally be divided into ogee weir, sharp crested weir, broad-crested weir and triangular, including ‘Crump’ weir or trapezoidal weir.

The types of overflow structures most typically used in the UK in conjunction with open channel spillways, are the frontal overflow and the side-channel overflow. With a few exceptions, both of these types could use any of these overflow configurations in plan and cross section.

Therefore, guidance is provided for the design of the main types of overflows depending on their configuration in plan and their cross-sectional weir shape, which, for simplicity, are discussed in the context of the frontal overflow.

Detailed guidance for the hydraulic design of side-channel overflows is provided in Hager and others (2020). This overflow type is typically used at embankment dams where the spillway structure is normally built on native ground outside of the embankment. This normally requires flow to change direction after spilling over the weir, whereby the downstream channel receives lateral inflow.

Detailed guidance on the design of shaft overflows, sometimes referred to as bellmouth 'Morning Glory' or drop-shaft overflows, is provided in Hager and others (2020).

A particular type of overflow, combining some of the features of the frontal overflow and the shaft overflow, is the siphon overflow.

Ackers and Thomas (1975) have reviewed the design and performance of spillway siphons and illustrated their company's experience up to the mid-1970s. More recent experience and the use of model studies is given in Ackers and Akhtar (2000).

Useful guidance for the design of black-water and white-water siphon overflows is provided in Hager and others (2020).

6.2.3.1 Overflow configuration in plan

The overflow configuration in plan depends on several factors, including the design flood and (safety check flood) unit discharge, available flood freeboard, available footprint, configuration of the conveyance structure, topographical and geological conditions, constructability and cost.

Straight crest overflows are typically used where sufficient footprint (depending on the layout) and flood freeboard are available, and the conveyance structure is preferred to be of the same width. This may be the case where the overflow structure is designed for a high unit discharge and any flow transition into a narrower chute could create significant shock waves (refer to Figure 6.1):

Figure 6.1: Straight overflow structure followed by a conveyance structure of the same width designed for high unit discharge, Holdenwood Dam, UK
(Source: V. Pavlov)

However, this may also occur where a wide lowcost or flexible conveyance structure is used (reinforced grass, stepped blocks or other), requiring a low unit discharge and regular flow distribution (refer to Figure 6.2).

Figure 6.2: Straight overflow structure followed by a low-cost or flexible conveyance structure designed for low unit discharge, Ogden Dam, UK
(Source: V. Pavlov)

Polygonal crest overflows are normally used where there is limited space available or where it is required or cost effective to reduce the inlet structure footprint. The most typical example is the labyrinth weir and its more recent variation – the piano key weir (refer to Figure 6.3 below):

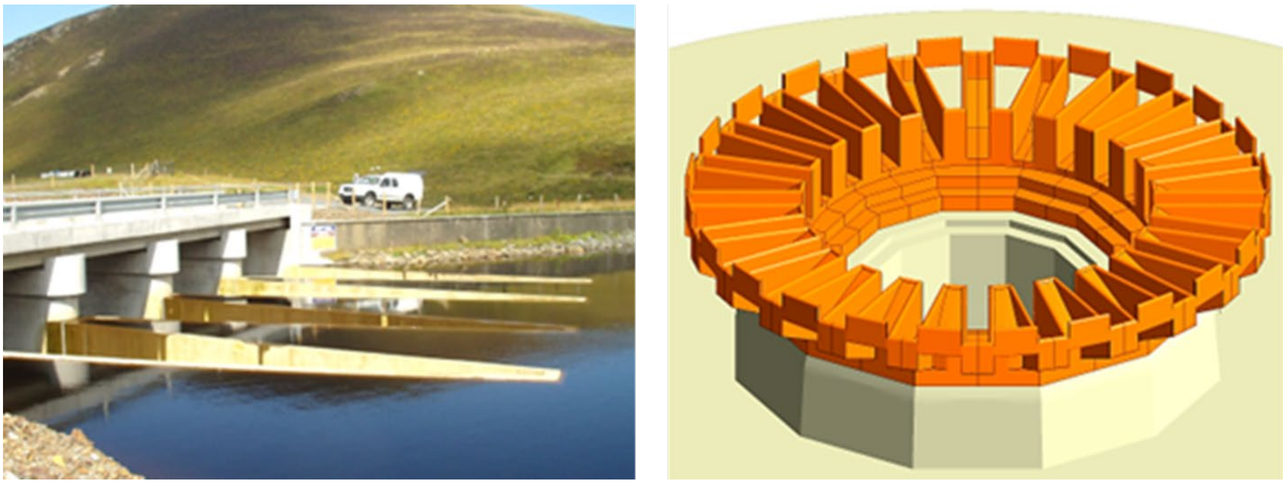

Figure 6.3: Polygonal crest overflow structure (labyrinth weir (left) and piano key weir (right) (Source: J. Ackers)

The piano key weir requires an even smaller footprint and therefore lends itself to retrofitting on top of existing structures as shown in Figure 6.3 (right).

This polygonal configuration allows the overall weir length, relative to the straight crest overflow, to be increased by a factor (referred to as a ‘magnification ratio’) of between 3 and 8. This increases the flow over the weir by a similar factor. However, its efficiency slightly reduces with increasing discharge due to the increased effect of the shared approach channels, angled flow over the weir tops and overflowing nappes interfering, compared to the straight overflow.

Detailed guidance for the design of labyrinth and piano key weirs is provided in Hager and others (2020).

Sometimes, polygonal crest overflows are used to improve the transition between a wide inlet weir required to control the flood surcharge within the reservoir and a narrower conveyance structure as a pragmatic alternative to a fully curved crest overflow (refer to Figure 2.1 d).

Curved crest overflows could be used to increase the weir length (refer to Figure 6.4) where this does not have a significant cost or constructability impact or where the benefits of achieving a smooth transition to the conveyance structure outweigh the cost.

Curved crest overflows are also typically used at the inlet to shaft overflows (refer to Figure 6.4):

Figure 6.4: Curved crest overflows with large radius (left) and small radius (right) (Sources: V Pavlov (left) and Binnies UK Ltd. (right)

Much like the labyrinth and piano key weir, where the radius of the crest curve is rather small, the efficiency of the curved weir would reduce with increasing the discharge due to the increased effect of the overflowing nappes crossing (refer to Figure 6.5 (right). At the other extreme, where the radius of the weir curve is relatively large, the hydraulic capacity would remain practically the same as that of a straight crest weir (refer to Figure 6.5 (left)).

Guidance for the design of circular crest overflows is provided in Hager and others. (2020).

6.2.3.2 Cross-sectional weir shape

The most efficient cross-sectional weir shape for a given flow rate, often referred to as an ‘ogee’ shape, is that defined by the lower nappe profile of a sharp-crested weir. This is illustrated in Figure 6.5 where H_{max} is the head over the ogee weir at the safety check flood and H_{sharp} is the head over the corresponding sharp-crested weir at this flood. Therefore, at the safety check flood, the contact pressure over the ogee weir would be atmospheric.

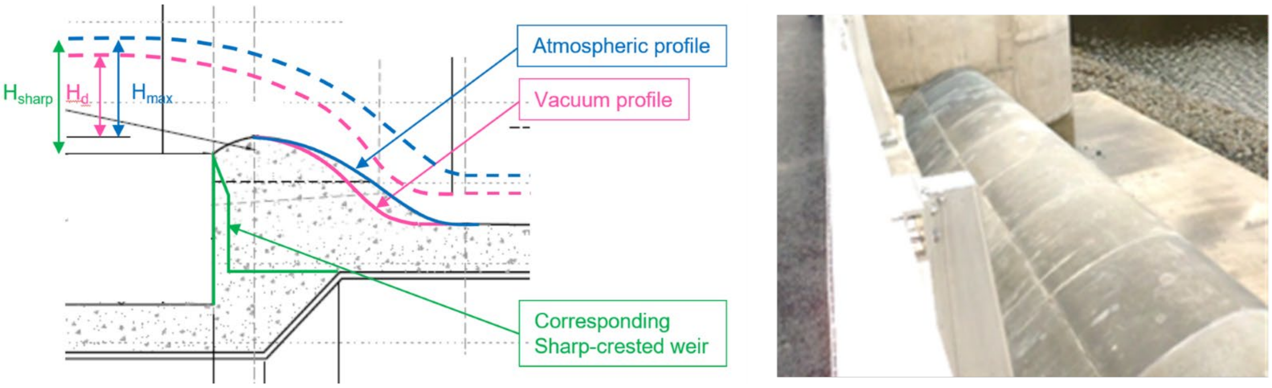

Figure 6.5: Ogee shape weir profile and flow nappes under design flood and safety check flood conditions (Source: United Utilities (right))

If H_d is the head over weir at the design flood, it could be seen that the lower nappe of the equivalent sharp-crested (vacuum profile) is lower than that corresponding to the safety check flood. Therefore, if the weir was profiled for the design flood, this will cause a partial vacuum at the safety check flood, which, in turn, will increase the weir discharge coefficient. However, to prevent actual flow separation, which would increase the head over the weir, the maximum head/design head ratio should not exceed approximately 1.3.

Due to their high efficiency and stable flow operation, ogee weirs are widely used in dam engineering for high unit discharge overflows and have been studied extensively in the past. They are also able to accommodate relatively large downstream submergence without significantly reducing the discharge coefficient. This could be beneficial where such weirs are used in side-channel and shaft overflow arrangements where there is a potential for partial submergence of the weir due to the effect of downstream flow restrictions.

Detailed information for the design of ogee weirs is provided in Hager and others (2020).

Sharp-crested weirs are approximately 20% less efficient than ogee weirs. They have also been studied in great detail and are often used as a flow measurement and control structure. To achieve stable flow over the weir, the lower nappe needs to be well aerated. Sharp-crested weirs become submerged as soon as the tail water level rises up to the weir crest, which has an immediate and noticeable impact on their discharge coefficient.

Many inlet gates would perform as sharp-crested weirs when overtopped.

Detailed guidance for the design and performance of sharp-crested weirs is provided in BS ISO 1438:2017.

Broad-crested weirs are easier to construct and to achieve stable flow control. Their performance is very similar to that of local channel contractions (flumes). Much like flumes, broad-crested weirs could accommodate downstream submergence up to a level corresponding to about the critical depth over the weir.

Detailed guidance for the design and performance of broad-crested weirs is provided in BS ISO 3846:2008.

Various triangular and trapezoidal cross-sectional shape weirs have been used in the past. Information on their discharge coefficients could be found in CIRIA Technical Note 134 (1989). However, many of them have not been subject to as detailed research as the ogee, sharp-crested or broad-crested weirs and therefore there is a degree of uncertainty with regards to their discharge coefficient and performance under partially submerged or partial vacuum conditions.

A specific shape of triangular weir, characterised by an upstream slope of 1:2 and a downstream slope of 1:5, known as a Crump weir, has also been extensively studied in the past (White, 1971). It was found to present several advantages, including easy construction, relatively high and constant discharge coefficient over a wide range of flow rates and high modular limit.

Detailed guidance for the design and performance of triangular weirs is provided in BS ISO 4360:2020.

It should be noted that the weir discharge coefficient of all weirs generally varies as a function of the ratio of the head over the weir to the approach depth. For broad-crested weirs, it also depends on the length of the weir in the direction of flow.

Also, depending on the head over the weir, walls acting as weirs could initially behave as broad-crested weirs at low flows and transition to sharp-crested weirs at high flows. Normally, sharp-crested weir behaviour develops where the head over the weir is at least twice greater than the length of the weir in the direction of flow (Chugaev, 1982).

6.2.3.3 Hydraulic design considerations

The hydraulic design of the overflow structure generally involves calculating the flood still-water level based on the weir discharge coefficient. For straight crest overflows, the discharge coefficient should allow for the specific cross-sectional weir shape, effects of any local contraction at piers and abutments, height of the weir above the approach channel and any downstream submergence.

In addition, the head loss due to friction along the approach channel may need to be considered where relatively shallow approach depths may be present.

Where polygonal or curved crest overflows are used, the discharge coefficient is further affected by a combination of shared approach channels, oblique flows over the weir and intersecting nappes. The specialised design guidance referred to in section 6.2.3.1 can be used to determine it.

6.2.3.4 Gated overflow structures

The hydraulic design of gated overflow structures is mainly concerned with determining the optimal configuration and profile of the overflow crest, piers and abutments in view of minimising the head loss, while preventing flow separation and cavitation and reducing the flow-induced vibration effects.

Examples of gated overflow structure crest profiles to prevent flow separation are provided in Figure 6.6:

:

Figure 6.6: Gated overflow structure crest profiles achieving smooth flow conditions without flow separation (Source: J. Hinks)

As discussed in section 4.7.1, analysis of the transient flow conditions developing as a result of gate operation should be carried out considering all possible operational scenarios and degrees of gate opening. In this respect, an overflow structure equipped with tipping gates could be designed at different weir levels, allowing the gates to operate sequentially in order to reduce the magnitude of the sudden increase in flow rate when the gates operate.

Hydraulic effects to be considered include, but are not limited to, dynamic impact due to hydrodynamic forces and impact from floating debris and ice on spillway walls and stilling basin chute blocks, baffles or sills and stilling basin sweepout.

Consideration should also be given to determining the optimal gate operation in view of mitigating the risk of blockage by floating debris. In this respect, asymmetric gate operation and complete opening of a few gates would be preferred to partial opening of all gates (Boes and others, 2017). However, the potential for such an operation to have a detrimental effect on the performance of the energy dissipation structure should also be considered.

6.2.4. Transition structure

This part of the inlet structure achieves the transition between the overflow structure and the conveyance structure.

Where the spillway is installed over a gravity dam, such a transition is not normally present.

Where the spillway is installed outside of the embankment or on top of embankment dams, the transition structure normally must fulfil one or more of the following functions:

- Promote supercritical flow conditions. This may be necessary where flow must pass under a bridge structure and water depth should be kept low in order to provide sufficient clearance to allow the passage of floating debris. It is normally achieved by providing a relatively steep channel slope and relatively gradual flow contractions.
- Achieve a smooth transition to the conveyance structure without unacceptable shock waves or separation of the supercritical flow from the invert. This would normally require gradual channel transitions in plan and longitudinal profile.
- Promote subcritical flow conditions. This may be necessary where efficient ‘cushioning’ and energy dissipation is required to reduce vibrations and/or achieve uniform flow distribution into the conveyance structure. It could be achieved via an end weir or a relatively significant channel contraction.

It should be noted that supercritical flow conditions could normally be guaranteed in a prismatic channel (with constant cross section) with a slope of 1 in 100 typically provided for self-drainage purposes. However, where a significant channel contraction is present, there is a potential for flow to ‘choke’ at the downstream end, therefore forcing subcritical flow conditions and hydraulic jump in the upstream channel.

Also, prismatic channels receiving lateral inflow, typically used in conjunction with side weirs, normally generate subcritical flow conditions even at slopes much steeper than 1 in 100, which is generally beneficial as long as this does not cause significant weir submergence.

6.2.5. Passage of floating debris

Free passage of floating debris is normally required at reservoir spillways in order to prevent the spillway becoming blocked and its capacity being reduced. This also generally improves the ecological functioning of a water body as the natural floating debris passed downriver can contribute to the formation of riverbeds, by providing shelter as well as habitat and food sources for many species (Boes and others, 2017).

To allow free passage, the minimum clearance width and height of the spillway opening should normally be greater than 80% and 20% respectively of the expected maximum floating debris length (Godtland and Tesaker 1994).

In addition, where the Froude number (Fr) at the approach to the weir is greater than 0.15, there is a potential for some floating debris to plunge under others and a multi-layer matt to form. Then, at Fr > 0.30, floating debris could be drawn down towards the spillway crest and get stuck upstream of it, which would reduce the overflow capacity. In the interval Fr=0.15-0.30, the density of the trees is the governing factor for whether the trees are drawn down or not (Hartlieb, 2015).

The definition of the approach velocity and Froude number are shown on Figure 6.7:

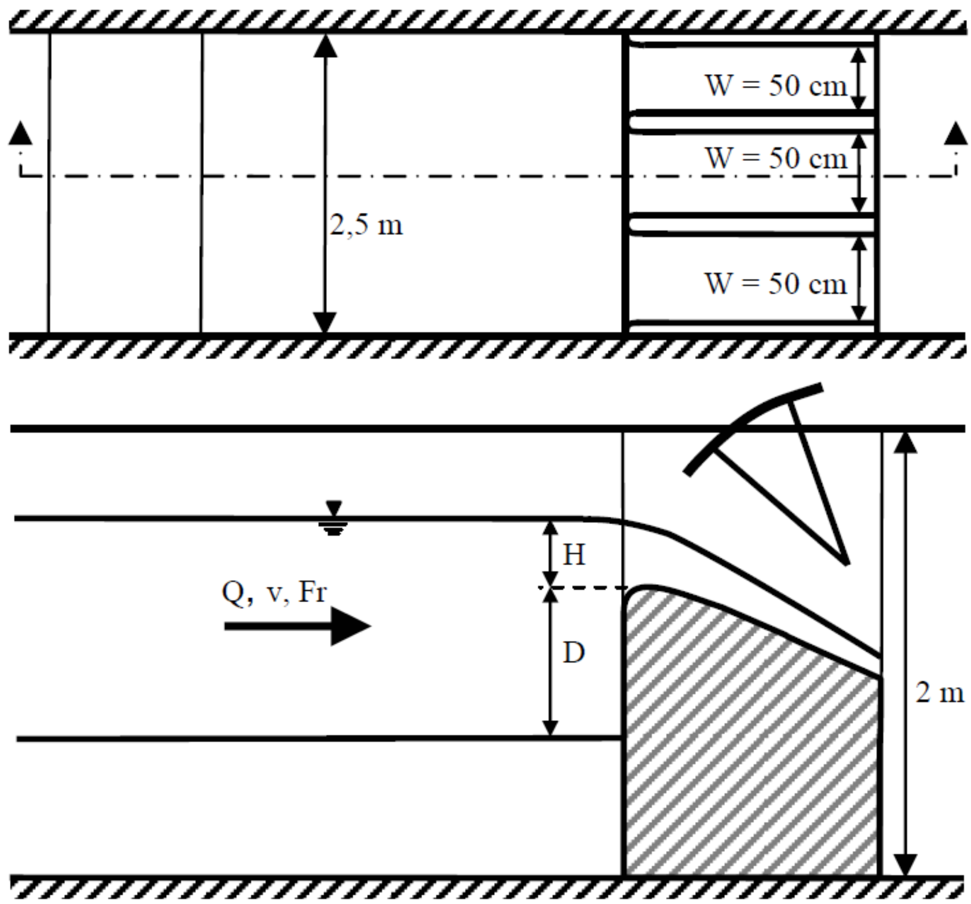

Figure 6.7: Layout and section of the spillway model (Hartlieb, 2015)

It should be noted that the location at which the Froude number applies should be as close as possible to the spillway structure, while being clear of the effects of draw-down, typically at a distance of approximately 2H upstream of it, H being the head over the weir (ICOLD, 2021).

Further guidance on design measures for improving the conditions for passage of floating debris is provided in ICOLD (2021).

Where ensuring free passage of floating debris may not be practical, various measures to mitigate the risk of spillway blockage could be considered. Such measure may include catchment management, debris retention (via wood racks, skimming baffles or floating booms), operational measures and contingency planning (ICOLD, 2021).

6.3. Conveyance structure

6.3.1. General

Reinforced concrete open channel conveyance structures are generally designed as smooth, stepped or baffled chutes depending on the proportion of the total energy available that it would be economical to dissipate along the chute and downstream of it respectively.

Alternative low-cost or flexible types of conveyance structures could be used in accordance with section 9 of this document.

In plan, the chute could be straight, curved, converging or diverging and could also incorporate changes in grade. As flow in the chute is normally supercritical, to prevent the formation of significant cross-waves, flow separations or unacceptable pressures, all horizontal and grade transitions should be gradual, normally achieved using long radius curves or small angles of contraction/expansion.

Due to the relatively high velocities involved, flow down the chutes is normally subject to significant aeration, which needs to be addressed by providing adequate freeboard to the channel coping. Where the flow velocity is high, there is also a risk of cavitation damage (refer to section 4.7.2).

6.3.2. Smooth chute

With the advent of reinforced concrete, the smooth chute became the most common structure used to convey high velocity flow. This is due to a number of beneficial features, including high resistance to abrasion and dynamic action, low friction and resulting flow depth, small number of joints and resulting reduced maintenance cost (compared to masonry spillways).

Due to the high flow velocity, the hydraulic design of smooth chutes is mainly concerned with predicting the water depth, considering the effects of air bulking due to air entrainment, shock waves at changes in flow direction and roll waves. This would, in turn, allow determining the required freeboard to the spillway channel cover level.

Where the high flow velocity is combined with a relatively high flow depth, the potential for cavitation would also need to be evaluated considering the presence of features such as abrupt horizontal changes in flow direction, chute blocks, and expansion joints. This is because the natural air concentration near the spillway invert reduces significantly as the flow depth increases, therefore increasing the cavitation potential.

The black-water (non-aerated) flow depth along the chute should be calculated considering the non-uniform flow conditions using Manning's equation. For smooth reinforced concrete chutes a roughness of $n=0.016\ s/m^{1/3}$ should be used to calculate the maximum water depth, while $n=0.010\ s/m^{1/3}$ should be used to calculate the water depth at the downstream end of the chute for the purposes of designing the energy dissipation structure and assessing the cavitation risks.

6.3.2.1 Surface air entrainment

In fully developed turbulent flow, where the flow velocity exceeds 10 to 15m/s (Hager and others, 2020), air is naturally entrained into the flow due to the exchange of energy at the interface between the water surface and the air above it. The entrained air therefore increases the flow depth downstream of the so called 'inception point' at which the turbulent boundary has fully developed.

Detailed guidance for calculating the aerated flow depth is provided in Hager and others (2020).

6.3.2.2 Channel freeboard

Once the maximum aerated flow depth along the spillway chute is established (including any super-elevations due to shock waves or roll waves), it is recommended that a minimum 600mm freeboard be provided to the chute wall cover level at the design flood. This would allow for the combined effects of air bulking, surface roughness, wave action, spray and splash and any modelling uncertainties.

As recommended in section 4.7.1, where practical and acceptable to the reservoir owner, for example for newly built spillways, this freeboard could also be provided at the 'safety check flood' in order to provide resilience for potential future increases in flood estimation and climate change, for a relatively small increase in the initial capital cost.

It is also recommended that the spillway chute wall height estimated using the above freeboard allowance should not be lower than the sum of the non-aerated (black-water) flow depth (including any super-elevations due to shock waves or roll waves) and the freeboard estimated following the guidance provided in USBR (2014) as follows:

$$\text{Channel freeboard} = 0.6 + 0.025\,V(d)^{1/3} \text{ [ft]}$$

Where: *V* – average non-aerated flow velocity [ft/s]

d – non-aerated flow depth [ft]

6.3.2.3 Cavitation and forced aeration

As discussed in section 4.7.2, a cavitation potential may generally exist on smooth spillway chutes where velocities approach 30m/s (ICOLD, 2016).

However, as highlighted by USBR (2014), joint block-outs and other similar flow surface irregularities may have the potential to exacerbate adverse hydraulic effects such as cavitation at flows exceeding a velocity as low as 15m/s. In this respect, most hydraulic analyses and designs of spillways carried out by USBR “will include evaluation of cavitation potential and subsequent mitigation” (USBR, 2014).

The cavitation potential is generally estimated using the cavitation index σ as a function of the total pressure at the spillway surface, vapour pressure of water, water density and average flow and velocity. According to Falvey (1990), significant cavitation damage is probable for smooth concrete chutes where $\sigma < 0.2$.

Detailed guidance for evaluating the cavitation potential and design measures to control cavitation are provided in the USBR Engineering Monographs No. 41, Air-water flow in hydraulic structures (Falvey, 1980) and No. 42, Cavitation in chutes and spillways (Falvey, 1990). In accordance with this guidance, where the cavitation index is less than 0.6, consideration could be given to specifying more stringent flow surface construction tolerances and/or higher-class concrete. Where the cavitation index is lower than 0.2, consideration could be given to changing the geometry of the spillway or providing forced flow aeration.

It should be noted that the cavitation potential is also largely affected by the bottom air concentration and therefore is significantly reduced at shallow water depths (ICOLD, 1992). In this respect, Peterka (1955) evidenced that practically no cavitation erosion occurs where a minimum air concentration near the channel bottom (depth of ~ 0.2m) of 6% to 8% is provided.

Where the natural air entrainment is insufficient to achieve the bottom air concentration required to prevent cavitation, special aerators must be provided.

When estimating the risk of cavitation erosion and the need to mitigate it, one should also consider the frequency of specific flood occurrence.

Further guidance for evaluating the cavitation potential and design of chute aerators is provided by Hager and others (2020).

6.3.2.4 Shock waves and roll waves

Shock waves and resulting cross waves are generated in high velocity supercritical flow as a result of flow perturbation such as occurring at relatively abrupt changes in flow direction, contractions and expansions, channel junctions, piers (rooster tails) and others.

Roll waves (or wave trains) could also form in steep spillway chutes subject to high Froude number flow (Hager and others, 2020).

Both shock waves and roll waves are normally accompanied by air entrainment and generate super-elevations which need to be considered in the design of the channel wall heights.

An example of a spillway chute with a sharp bend, which in 1946 was washed away in a flood by shock waves and was subsequently strengthened twice, is shown in Figure 6.8:

Figure 6.8: Spillway chute with sharp bend built over a railway line and subject to shock waves at the bend (Source: D.Brown)

An example of roll waves forming on a steep chute is shown in Figure 2.2 a).

Detailed guidance for considering the effects of shock waves and roll waves in smooth chutes is provided in Hager and others (2020).

6.3.3. Stepped chute

Stepped chutes were historically the main type of conveyance structures used and were normally constructed of stone masonry, although often backed and underlain by mass concrete. Nowadays, stepped chutes are mainly constructed using roller compacted concrete (RCC), either as an integral part of a gravity RCC dam or as an overtopping protection of embankment dams. Other materials used include reinforced concrete, gabions, stepped wedge blocks among others.

An example of a modern reinforced concrete stepped spillway chute is shown in Figure 6.9:

Figure 6.9: Modern concrete stepped spillway on the face of a concrete gravity dam discharging onto large stepped reinforced concrete channels built on the abutments, Clywedog Dam, UK (Source: N. Prytherch)

Stepped chutes promote increased energy dissipation via jet impact, increased flow disturbance and air entrainment and therefore allow a reduction in the size of the terminal energy dissipation structure.

The main drawbacks of stepped chutes related to their hydraulic regime are the increased flow depth and dynamic forces acting on the structure, as well as the increased cavitation potential upstream of the inception point.

In this respect, the velocity threshold upstream of the inception at which there is a potential for cavitation to occur is suggested to be as low as 15m/s, based on analysis of the pressure field (Amador and others (2005). Further downstream, where flow is significantly aerated, the risk of cavitation is generally negligible (Chanson, 2015) and (ICOLD, 2016).

In addition, unlike reinforced concrete structures, masonry stepped chutes are prone to dislodgement of the stone sets due to deterioration of the jointing and bedding mortar, reduced (submerged) stone weight and the dynamic forces acting on them.

When modelling stepped chutes, it is important to consider the impact of scale effects on the pressure fields, air concentration and energy dissipation. In this respect, Boes (2000) suggests that a minimum Froude scale of between 1:10 and 1:15 should be used for hydraulic models in order to minimise these effects.

Useful guidance for the hydraulic design of stepped chutes is provided in ICOLD (2016) and Hager and others (2020).

6.3.4. Baffled chute

In general, baffled chutes aim to prevent excessive acceleration of flow down the chute. Two types of chutes could be considered, namely chutes inducing ‘tumbling’ flow (a succession of hydraulic jumps between the baffle elements) and a USBR type of ‘baffled apron’ where flow passes over and between the baffles (Peterka, 1984).

Both types of chutes generally apply to gradients of 2:1 or flatter. They both achieve a quasi-critical flow regime where the specific energy is maintained close to its minimum level, that is, the velocity at the outlet is nearly equal to the critical velocity.

The ‘tumbling’ flow chute normally requires a baffle spacing-height ratio of 7.5 to 12 (a value of 10 being recommended). It could use either continuous transversal baffles or staggered cubes. Guidance for the design of this type of chute is provided in (Khatsuria, 2005).

The design capacity of ‘baffled apron’ chutes per linear meter of width generally ranges from $0.5m^2/s$ to $1m^2/s$, therefore achieving relatively mild flow conditions on the chute and significantly reducing the need for energy dissipation downstream of it (Peterka, 1984).

An example of a ‘baffled apron’ chute is shown in Figure 6.10:

Figure 6.10: Baffled apron chute and inlet structure with crest bridge, Lee Green, UK (Source: V. Pavlov)

Baffled chutes present an increased risk of floating debris becoming wedged/trapped by the baffles where highly vegetated catchments are present.

Detailed guidance for the design of the ‘baffled apron’ chutes is provided in Peterka (1984).

6.4. Energy dissipation structure

6.4.1. General

The energy disszization structure should ensure that the kinetic energy remaining at the downstream end of the spillway chute is efficiently dissipated in order to prevent erosion that could potentially undermine the structure and cause head cutting, thereby posing a risk to the safety of the dam. In addition, it should generally prevent significant erosion of the natural riverbed and/or banks or of any adjacent structures.

The main types of energy dissipators used in the UK are hydraulic jump stilling basin, ’impact’ type energy dissipator and ‘ski-jump’ and plunge pool energy dissipator.

The typical range of applicability of the classical hydraulic jump stilling basin (sometimes referred to as ‘simple hydraulic jump basin’), stilling basin with chute blocks, baffle blocks and sills (sometimes referred to as ‘baffle basin’) and ‘ski-jump’ and plunge pool energy dissipator (sometimes referred to as ‘free trajectory jet dissipator’) is illustrated in Figure 6.11 (Mason, 1982):

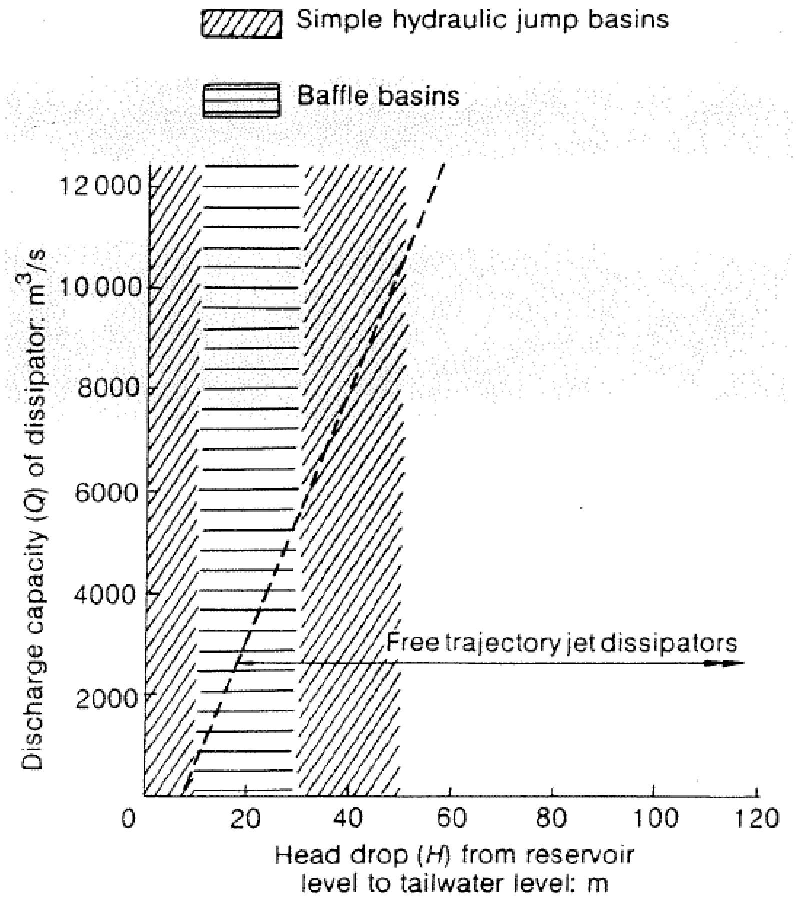

Figure 6.11: Preferred ranges of use for the main types of energy dissipators (Mason, 1982)

The selection of the type of energy dissipator and its hydraulic design should consider, among other things, the amount of energy to be dissipated, the return period and duration of the flood events, the hydraulic, geological/geo-morphological characteristics of the downstream watercourse and the risks to the dam or to any adjacent structures posed by potential erosion and undermining of the energy dissipation structure.

It should also consider any potential health and safety impact on the reservoir operators or members of the public (refer to section 11).

In particular, the analysis of the hydraulic conditions downstream of the energy dissipator should consider the impact of any tributaries, bridges or other terrain or man-made features on the transient flood routing conditions at different flow rates. In some specific cases, it may be prudent to allow for the potential for erosion of the riverbed and banks causing adverse alterations of the watercourse cross sections and resulting tail-water depths.

The protection of the riverbed and banks downstream of energy dissipators should consider the bottom velocity of flow exiting the stilling basin and should be extended over a sufficient distance downstream of it where the effects of significant flow disturbance and non-uniform velocity distribution are manifested. This should normally be about 10 times the subcritical conjugate depth USACE (1992) to prevent potential undermining of the energy dissipation structure, unless it is keyed into high quality bedrock.

The energy dissipation structure is subject to significant hydrodynamic loads which may cause limit state failure or structural damages due to uplift and vibration (refer to sections 7.3.2 and 8.3.3).

In addition, there is normally an increased risk of cavitation within the energy dissipation structure, in particular at chute blocks and baffles where flow separation occurs, due to the increased water depth and reduced bottom air concentration.

Except for cavitation, all other effects are to be addressed by the structural and geotechnical design, while the hydraulic discipline has to advise on all pertinent hydraulic parameters, including maximum water depths and flow velocities.

6.4.2. Stilling basin

The stilling basin is the most commonly used type of energy dissipation structure, both in the UK and internationally. This is due to its simplicity of construction and reasonable predictability of operation based on the extensive research and testing carried out over the years.

The stilling basin achieves energy dissipation through the enhanced turbulence generated in the hydraulic jump that occurs within it.

The 2 fundamental hydraulic design requirements that a stilling basin must satisfy are to force a hydraulic jump and to contain it under all flow and tailwater conditions.

A hydraulic jump is forced to occur by creating conditions such that a water depth slightly greater than the subcritical conjugate depth is maintained within the stilling basin under all flow conditions. This could be achieved by either lowering its level sufficiently below that of the natural tailwater or by artificially creating the required tailwater depth via an end sill.

In order to contain the hydraulic jump, the stilling basin should have sufficient length, normally equal to or greater than the hydraulic jump length, taking into consideration the backwater curve length where an end sill is used. In some cases, an additional factor of safety of 1.2 or so could be applied to the hydraulic jump length in order to determine the stilling basin length (Stojnic and others, 2020).

It should be noted that, for practical reasons, the use of stilling basins is generally limited to approach velocities of about 20m/s due to cavitation problems, tailwater requirements and extreme dynamic pressure fluctuations (Hager and others, 2020), which become increasingly expensive to address and accommodate. For higher velocities, ski jump/flip bucket and plunge pool energy dissipators are normally provided.

At relatively low Froude numbers, the length of the hydraulic jump can be reduced by installing chute blocks, baffles and sills or by providing an abrupt expansion relative to the width of the spillway chute. This would also improve the stability of the stilling basin, that is, its capacity to maintain the position of the hydraulic jump under variable tailwater conditions.

6.4.2.1 Classical hydraulic jump stilling basin

The classical hydraulic jump stilling basin forces a hydraulic jump to occur within a straight, rectangular, horizontal channel without the aid of chute blocks and baffles.

This type of stilling basin is generally used at relatively high Froude numbers (typically between 4.5 and 9.0) where it provides efficient energy dissipation between 45% and 70% and achieves well-balanced stable flow conditions (Peterka, 1984). At Froude numbers above 9, the efficiency of energy dissipation increases further, while the hydraulic jump is characterised by a rough water surface with strong surface waves downstream from the jump (Peterka, 1984). This needs to be carefully considered in the design of the downstream erosion protection.

In this respect, a literature survey of 370 prototype energy dissipators, from dams in 61 countries, carried out by Mason (1982) indicates that this type of energy dissipations structure is most typically used at heads up to 10m and above 30m. According to this survey, the head range between 10m and 30m is generally suitable for stilling basins with baffles (refer to section 6.4.2.4) due to their potential to provide enhanced energy dissipation within this range of heads and corresponding Froude numbers.

The length of the classical hydraulic jump is approximately 6 times the subcritical conjugate depth, while the roller length is generally 25% shorter (Hager and others, 2020).

It should be noted that the hydraulic jump length occurring in stilling basins downstream of stepped chute is about 15% greater than that downstream of smooth chutes due to the different approach flow conditions (Stojnic and others, 2020).

Detailed guidance for the hydraulic design of classical hydraulic jump stilling basins is provided in Hager and others (2020).

6.4.2.2 Spatial hydraulic jump stilling basin

A spatial hydraulic jump occurs where the supercritical flow at the end of a spillway chute enters a wider channel, either through a sudden or a gradual expansion. This is also referred to as a “hydraulic jump in non-prismatic rectangular channels” (Hager, 1985).

The spatial hydraulic jump type of stilling basin could be used where it may be preferable to increase the stilling basin width but to reduce the requirements for tailwater depth and stilling basin length. While the relationship between the conjugate depths in a spatial hydraulic jump occurring in such a type of stilling basin is well established, experimental results relating to the length of the roller present some significant scattering, especially in the case of an abrupt expansion (Hager, 1985). In addition, there is evidence that the hydraulic jump is relatively unstable in the case of an abrupt expansion, which otherwise promotes better energy dissipation and is more cost effective. Normally, this could be successfully mitigated by providing different types of bottom irregularities such as sills or baffle blocks and therefore, this type of stilling basin could best be optimised during model testing.

In order for flow within the stilling basin to be uniformly distributed and not to oscillate too much in a transverse direction, a maximum expansion of 3 times the approach width is normally considered (Rajaratnam and Subramanya, 1968).

The required length of the ‘spatial’ hydraulic jump stilling basin is about 6 to 8 times the ‘spatial’ jump conjugate depth (Hager and Schleiss, 2009).

An example of a spatial jump stilling basin is shown in Figure 6.12:

Figure 6.12 Spatial hydraulic jump stilling basin with baffle blocks at the inlet to the basin. Swinden I Dam, UK (Source: V. Pavlov)

Detailed guidance for the hydraulic design of non-prismatic rectangular stilling basins is provided in Hager (1985). Further guidance for the hydraulic design of a spatial hydraulic jump in abruptly expanding stilling basins with transverse central sill is provided in Hager (2020).

6.4.2.3 Positive and negative step stilling basin

Positive and negative step stilling basins are often used in practice where the stilling basin may need to be deepened relative to the tailwater level in order to force a hydraulic jump or where it may be cost efficient to lower the apron of the stilling basin relative to the downstream invert level of the spillway chute.

The resulting flow conditions are different from those occurring in classical hydraulic jump and therefore require special consideration.

Useful guidance for the design of hydraulic jump stilling basins involving positive and negative steps is provided in Hager (1986).

6.4.2.4 Stilling basin with chute blocks, baffle blocks and sills (baffle basin)

Chute blocks, baffle blocks and sills could be provided within stilling basins in order to reduce their length and improve their hydraulic stability.

Different stilling basin configurations have been developed by USBR (Peterka, 1984) as a result of an extensive research programme, including laboratory and field testing. These basins were developed to be used for different ranges of Froude numbers and are known as USBR Basin types I (classical hydraulic jump stilling basin), II, III and IV.

An example of an USBR type III stilling basin is shown in Figure 2.3 a).

Detailed guidance for sizing the USBR types of stilling basins is provided in Peterka (1984).

6.4.3. Impact type energy dissipator

The ‘Impact’ type energy dissipator developed by Peterka (1984) requires no tailwater depth to perform successfully. It achieves energy dissipation via the impact of the incoming jet onto a vertical hanging baffle by water cushioning of the deflected jet.

This type of energy dissipator was designed for use with a closed upstream conduit but it is considered that it could be also used with an open channel entrance. Its capacity is generally limited to about 10m^3/s and it could handle approach velocities of up to about 9m/s. For larger discharges, multiple basins could be placed side by side. When a shallow depth inlet channel is used instead of a pipe, care should be taken to ensure that the bottom of the baffle extends sufficiently below the incoming channel invert level so that the jet trajectory does not miss the baffle at all flow rates.

An example of an ‘Impact’ type energy dissipator is shown in Figure 6.13:

Figure 6.13: ‘Impact’ type energy dissipator and receiving channel, Ogden Dam, UK (Source: United Utilities)

This type of structure is subjected to large dynamic forces and increased turbulence which need to be considered in the structural design.

Guidance for the design of the ‘Impact’ type energy dissipator is provided in USBR (1987).

6.4.4. Ski-jump and plunge pool energy dissipator

The ‘ski-jump and plunge pool’ arrangement, also sometimes referred to as a ‘flip bucket’, achieves energy dissipation mainly via jet dissipation through interaction with the air boundary and through water cushioning within the plunge pool and impact of the jet onto the riverbed.

This energy dissipation arrangement is normally used to dissipate high head energy at dams and is considered to be generally problem-free within its typical range of application (Mason, 1982). In particular, it is used as a cost-efficient alternative to a stilling basin structure where the velocity at the downstream end of the chute exceeds about 20m/s (Hager and others, 2020). This is due to the significant dynamic loading on the stilling basin base slab and walls and the risk of cavitation associated with such high velocities and significant water depths.

In addition to achieving energy dissipation, the ski-jump arrangement aims to direct the high velocity flow away from the dam to a location where potential erosion would not endanger it. Due to the inevitable scour caused by the impact of the water jet, this type of energy dissipator is normally used where the riverbed material is highly durable bedrock.

Where this may not be the case, suitable erosion protection, including concrete lining is typically provided.

While the classic ski-jump is normally arranged in a prismatic channel, therefore not generating any changes of flow direction in plan, flip buckets sometimes deflect the high velocity flow in plan in cases where there is a need for re-adjustment of the jet direction (Hager and others, 2020).

It should be noted that erosion of the riverbed could affect the river turbidity, which may be harmful for the fish population or may have any other adverse environmental effects that need to be assessed.

At flows much lower than the flow for which the ski jump was sized, it could operate as a hydraulic jump stilling basin on a curved apron. Consideration should therefore be given to providing adequate channel freeboard and erosion protection downstream of it to accommodate this flow condition.

An example of a ski-jump and plunge pool energy dissipator is shown in Figure 6.14:

Figure 6.14: Ski-jump energy and plunge pool energy dissipator under operation with highly aerated flow, Tarbela Dam, Pakistan (Source: J. Hinks)

Useful guidelines for practical design of flip buckets and plunge pools is provided by Mason (1993).

Further detailed guidance for the design of ski-jump/flip bucket and plunge pool energy dissipators, including guidance for the assessment of granular and rock scour is provided in Hager and others (2020).

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7. Structural design

7.1. General

This section provides guidance relating to the main aspects of the structural design of reinforced concrete open channel reservoir spillways, including the determination of the applied actions and their combinations, and specific limit state design considerations.

In particular, it provides references to the design methods and requirements set out in the latest and most pertinent specialised guidance with regards to:

- determination of the mean and fluctuating dynamic actions
- control of cracking caused by restrained deformations
- limit states criteria and verifications relating to fatigue

Typical construction details and arrangements are provided in order to promote consistency of design in accordance with current best practice.

This section is not dealing with the spillway stability conditions relating to uplift, overturning and sliding as they are largely governed by geotechnical and hydrogeological factors and as such are discussed in section 8, Geotechnical design.

7.2. Basis of structural design

The design methods used for the structural design of reservoir spillways should be in accordance with the applicable Eurocodes and related British Standards, duly considering all plausible static and dynamic actions and combinations.

In this respect, further to the stipulations of section 2 of BS EN 1992-1-1:2004+A1:2014 and section 2 of BS EN 1992-3:2006, the structural design should give due consideration to the dynamic and fatigue actions inherent in reservoir spillways and their analysis in accordance with the provisions of Cl. 4.1.4, 4.15 and 5.1.3 of BS EN 1990:2002 +A1:2005 and sections 2.2 and 3.3.1 of BS EN 1991-1-1:2002.

Considering the significant risks that a potential spillway failure poses to public safety, the structural behaviour of all elements of the spillways structure should remain within the elastic range under all actions, combinations and flow conditions including the 'safety check flood'. Therefore, the maximum stress in the concrete should not exceed the design compressive strength of concrete, and the maximum stress in the reinforcement should not exceed its design yield strength.

The design life of the reinforced concrete spillway structure should normally be 100 years or more based on the relatively significant replacement cost. The design life should be specified by the client by also considering the potential effects of climate change and possible future changes of the design standards and requirements.

The basis of structural design should be duly documented in a ‘Basis of Design’ statement which should include, but not be limited to, design approach and methods used, level of analysis (approximation), consequences class, tightness class, design actions and situations, and concrete specification. This should be a live document, initially created at outline stage and continuously developed as required, which could be used for obtaining qualified civil engineer and client approval at important stages of the design.

The concrete specification should consider, among other things, the nominal concrete cover referred to in section 7.4.4.1 and the potential for acid attack on concrete due to fast flowing acidic reservoir water.

7.3. Actions

All relevant actions shall be determined for each design situation identified in accordance with Eurocode and the applicable British Standards.

Further clarifications and non-contradictory complementary information relating to the main actions applied at reservoir spillways are provided where.

7.3.1. Static actions

7.3.1.1 General

The main static actions to be considered for the structural design of spillways include: structure self-weight, hydrostatic pressure and external earth pressure, including from storage of construction materials or construction equipment moving or placed adjacent to the spillway walls. In some specific cases, wind action and ice action may also need to be considered.

Static loading on the spillway inlet structure from floating debris would not normally be considered where the risk of blockage of the spillway inlet structure has been adequately mitigated (refer to section 6.1.5). However, the design should consider the dynamic impact load imposed by floating debris (refer to section 7.3.2.5).

7.3.1.2 Hydrostatic pressures

The characteristic internal horizontal hydrostatic pressure acting on the walls of the structure during spillway operation should be conservatively based on the ‘channel-full’ condition where no overtopping is allowed by design. Where channel overtopping is allowed by design, the characteristic internal hydrostatic pressure should be based on the predicted overtopping water level.

In both cases above, the design value of the internal hydrostatic pressure should be calculated by using a partial factor of γ_F=1.2 for ultimate limit state (ULS) design under the 'design flood' condition with reference to Cl. B.3 (2) of BS EN 1991-4:2006.

The 'safety check flood' condition could be assimilated to an accidental design situation whereby a partial factor of γ_F=1.0 could be used for ULS design with reference to Cl. B.3 (4) of BS EN 1991-4:2006.

The characteristic internal vertical hydrostatic pressure acting on the base slabs of the structure during spillway operation should be based on the predicted clear water depth. The internal vertical hydrostatic pressure acting as a uniformly distributed load (UDL) generally has a favourable effect by reducing the bending moment in the base slab and contributing to damping of the flow-induced vibrations. Therefore, it should not be normally factored except where it is used for estimating the bearing pressure and under the 'soft spot' load case (refer to section 7.4).

The design values of the external hydrostatic pressures relative to the base of the spillway structure should be assessed directly and should represent 'the most unfavourable values that could occur during the design lifetime of the structure' (Eurocode, 1997[1]) (in accordance with Cl. 2.4.6.1 (2) and Cl. 2.4.6.1 (6), where:

- the external ground water level has been derived from a seepage model and a reliable drainage system is installed
- the external water level corresponds to a tailwater depth derived from hydraulic modelling of the receiving watercourse

Where the external ground water level has been taken equal to the maximum adjacent ground level or equal to the maximum level of ponding controlled by the local terrain features, the corresponding pressure could be assimilated to an accidental design situation and a partial factor of γ_F=1.0 could be used for ULS design.

7.3.1.3 Earth pressures

The nature and magnitude of the lateral earth pressure acting on the spillway walls should be determined with due consideration of the soil-structure interaction and resulting combined effects of rotation at the wall base and wall deflection.

Consideration should be given to the stiffness of the spillway structure, the modulus of the spillway foundation, the nature and degree of compaction of the backfill material, the density of the soil material under water, as well as the presence of base slab outstands and joints and their effect on rotation at the wall base.

In general, 'for relatively thin walls where adjacent fill has not been compacted, there may be sufficient deflection that the active soil wedge will form. However, for more rigid walls or features, or where adjacent fill has been compacted, at-rest lateral loading should be considered' (USBR, 2014).

Also, rotation at the wall base would be more pronounced for wider spillway structures due to the increased deflection of the base slab, while any base slab outstands provided would reduce it.

Useful guidance on the approximate rotation of vertical walls backfilled with drained, non-cohesive soil required to mobilise active or passive earth pressures is provided in Annex C.3 of BS EN 1997-1:2004+A1:2013.

It is prudent that “a check should be made to ensure that that factored passive pressures do not create more resisting load than driving load. If this occurs, resisting pressures should be reduced to balance factored driving loads’” (USACE, 2016).

Lateral earth pressure resulting from surcharge load should be based on the anticipated maximum site-specific load imposed by storage of construction materials or equipment moving or placed adjacent to the spillway walls. If the surcharge load cannot be reasonably established at design stage, a minimum surcharge load of 15kN/m^2 could be conservatively assumed.

Where the spillway side walls are backfilled with clay soil material, the possibility that the backfill within the zone of seasonal movements of clay soils could shrink away from the walls should be considered (Eurocode, 1997[1]), as this would subject side walls to the full channel-side water load before deflecting enough to gain support from the backfill (USBR, 1987).

7.3.1.4 Thermal actions

The forces resulting from thermal expansion and contraction of the structural elements should be duly considered in the design of the spillway system of joints. Also, the effects of downhill creep due to seasonal thermal contraction and expansion should be considered where expansion joints are provided on a relatively steep slope and, where, if necessary, deep foundation keys could be provided. Useful guidance on this could be found in Hydraulic Structures (Smith, 1995).

7.3.1.5 Other static actions

Other static actions which in some specific cases would need to be considered include: wind action, ice load and traffic loading.

Wind action would normally need to be considered where tall exposed spillway walls are present such as the walls of stilling basins without substantial backfill. It could also be relevant during construction, before wall backfilling has taken place. The wind action (allowing for mean and short-term velocity fluctuations) should be determined based on BS EN 1991-1-4:2005+A1:2010 and the National Annex to it. The design should consider specifying any necessary requirements or constraints relating to the support and timing or removal of the formwork in order to efficiently control the adverse effects of wind action during construction.

Where there is a risk of ice cover forming in the reservoir and passing through the spillway structure, consideration should be given to the resulting ice load. Normally, ice load is based on site-specific data. Where such data are not available, 'a default acceptable estimate of static ice load is 10 000 pounds per linear foot (lb/lf) (13.6 kN/m) of contact between the ice and structure for an assumed depth of 2 feet (0.6m) or more when basic data are not available (USBR, 2014).

Traffic loads transferred onto the spillway structure via a bridge structure should be determined in accordance with BS EN 1991-2:2003 and the applicable British Standards and relevant UK guidance.

7.3.2. Dynamic actions

7.3.2.1 General

The main dynamic actions to be considered for the structural design of spillways include: mean hydrodynamic forces, flow-induced vibrations, seismic action, vibrations due to wind turbulence, wave action, dynamic impact from floating debris and/or ice and traffic loading.

In defining the dynamic actions, consideration should also be given to the provisions of section 5.1.3 of Eurocode (1990).

7.3.2.2 Mean hydrodynamic forces

The time-mean hydrodynamic forces acting on various parts of the spillway structure subject to high velocity flow, including piers, chute blocs, baffle blocks, baffle walls, end sills, spillway chute walls or base slabs at changes in horizontal alignment or gradient could be determined based on the guidance provided in 'Hydraulic structures design manual, No.3, Hydrodynamic forces' (Naudascher, 1991).

The U.S. Army Corps of Engineers Engineer Manual EM 1110-2-1603, Hydraulic design of spillways (1990) provides an empirical method for calculating the average minimum combined static and dynamic unit force, varying along the length of stilling basin. The proposed method is based on tests conducted at the US Army Engineer Waterways Experiment Station with an instrumented sidewall in a stilling basin that did not contain baffles or an end sill. The tests were conducted with approach flow Froude numbers that varied between 2.7 and 8.7 (USACE, 1990).

7.3.2.3 Flow-induced vibrations

High velocity flow in reservoir spillways chutes is characterised by extensive turbulence and energy dissipation along these structures. Flow turbulence is even greater within stilling basins and other energy dissipation structures where a significant loss of energy is forced to occur over a short channel length within the hydraulic jump or via jet impact. The high turbulence and associated fluctuations of the velocity and pressure give rise to flow induced vibrations.

According to Naudascher (1991), the fluctuating component of the hydrodynamic forces and their corresponding pressures, which are responsible for the excitation of structural vibrations, could be represented by their root-mean square (rms) values: $(p')_{rms} = \sqrt{\overline{p'^2}}$ (refer to Figure 7.1):

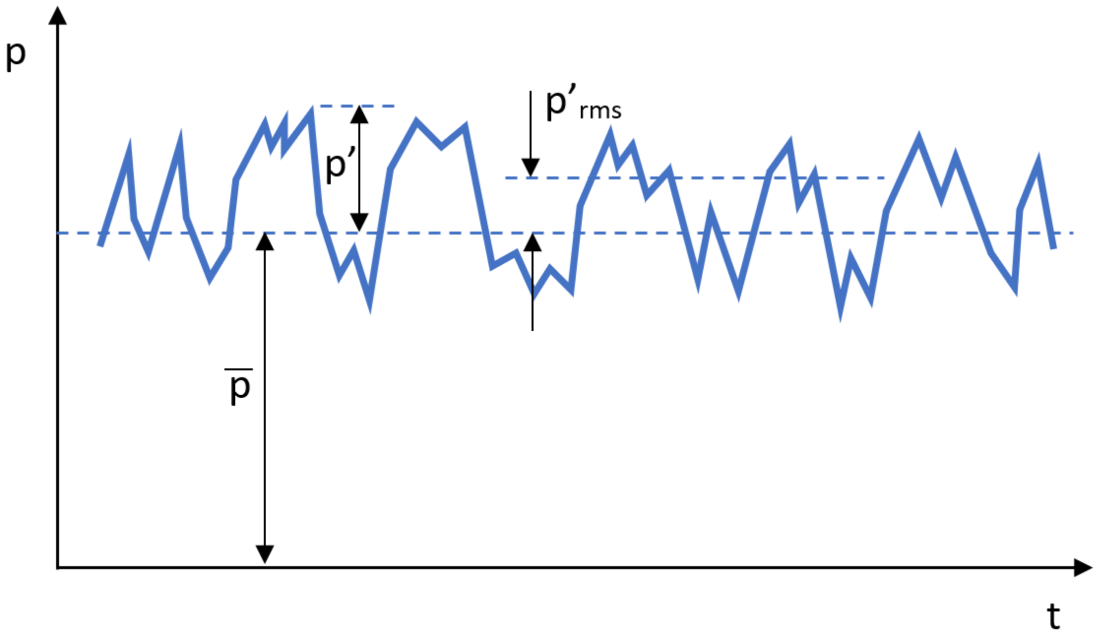

Figure 7.1: Mean and fluctuating hydrodynamic force components

The p_{rms} value of the wall-pressure fluctuations on spillway chutes, in relation to the dynamic pressure (velocity head) $\rho v_0^2/2$ has been found to be approximately constant, and equal to (Naudascher, 1991):

$p'_{rms} = 0.01\ \rho v_0^2/2$ - for smooth boundary

$p'_{rms} \leq 0.05\ \rho v_0^2/2$ - for rough boundary

In the current absence of detailed information on the p'_{rms} values relating to stepped, baffled or macro-roughness spillway chutes, the above value for rough boundary could be used, before any more detailed research is carried out.

The p'_{rms} value of the wall pressure fluctuation at hydraulic jump stilling basins has been found to be an order of magnitude higher than in smooth boundary chutes (Narayanan, 1978). This is also corroborated by the results of laboratory studies on free hydraulic jumps carried out by Khader and Elango (1974) (refer to Figure 7.2):

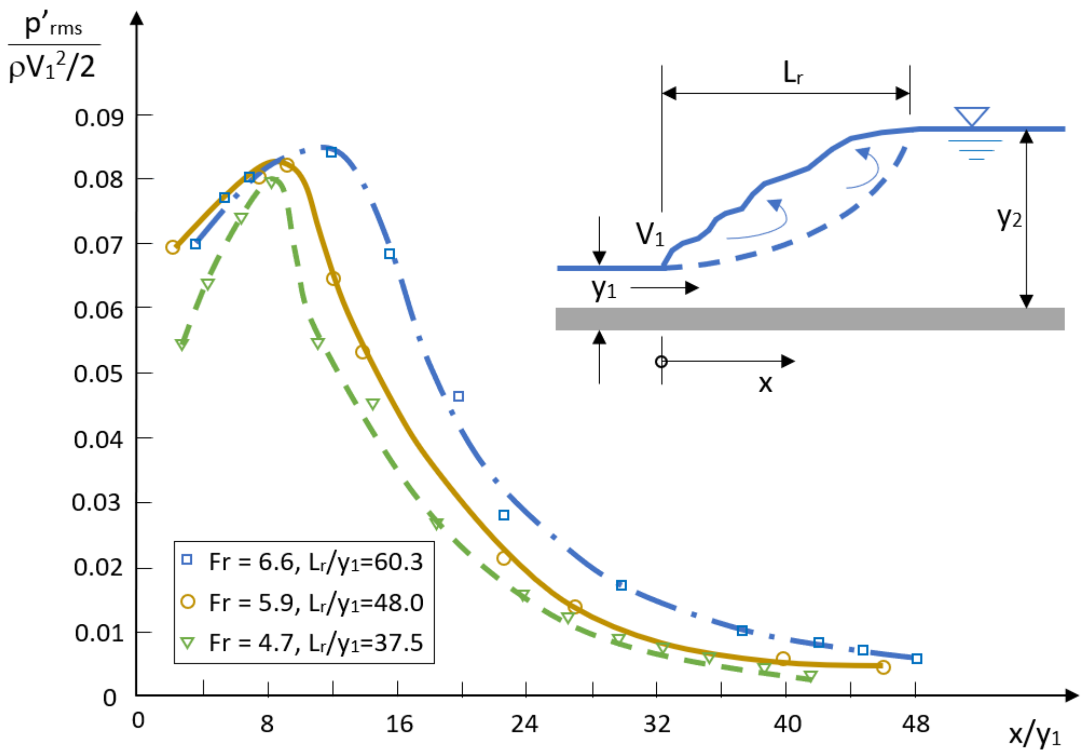

Figure 7.2: Free hydraulic jump p'rms value of pressure fluctuations (after Khader and Elango, 1974)

While Naudascher (1991) suggests that the above and other similar laboratory study results should be used with care, a trend for much higher flow-induced vibrations being generated at stilling basins is apparent.

Therefore, the p'_{rms} value of the wall pressure fluctuation for use at stilling basins could be assumed:

$p'_{rms} = 0.10\ \rho v_0^2/2$ – for stilling basins

This would be consistent with the widely acknowledged significant turbulent pressure fluctuations occurring within stilling basins which have been identified as the root cause of several failures in the past. It should be noted that such value of the wall pressure fluctuation would also need to be considered for the lower section of the chute if the hydraulic jump is likely to take place at this location due to its submergence at lower flow rates.

Based on prototype tests it has been established that the typical spectra of the wall-pressure fluctuations in a turbulent boundary layer along a smooth boundary is random in nature (Naudascher and Rockwell, 2016). Furthermore, the tests confirmed that the broad-band spectrum inherent in the flow-induced vibrations could excite relatively low frequency vibrations corresponding to the lowest modes of oscillation of such structures and therefore could cause their resonance.

Therefore, the dynamic loading due to flow-induced vibrations could be conveniently modelled as a spectrum of the p'_{rms} value of the wall pressure fluctuations taken as a percentage of the velocity head, while allowing for their spatial correlation. With this approach, the spectrum used should include the frequency range corresponding to the lowest natural modes of oscillation of the structure where the most significant resonance could occur (Pavlov, 2021).

The loading from flow-induced vibrations could be applied as a UDL, while applying a spatial correlation factor to the p'_{rms} value of the wall pressure fluctuations. At both spillway chutes and stilling basins the spatial correlation factor could be assumed equal to the coefficient related to the instantaneous spatial distribution of the pulsating pressures for sealed joints determined from the 'Design of Stilling Basin Linings with Sealed and Unsealed Joints' (Barjastehmaleki and others, 2016). It is determined as a function of the lining slab length and width relative to the approach supercritical flow depth.

Where necessary, the value of the coefficient related to the instantaneous spatial distribution of the pulsating pressures for sealed joints could be estimated by extrapolation of the empirically produced curves, but should not be taken lower than 0.3 due to the uncertainty inherent in such an estimation.

7.3.2.4 Seismic actions

Where a need to carry out an explicit seismic design has been identified by the designer (refer to section 7.4.2.4), the seismic actions should be determined in accordance with the relevant parts of Eurocode 8.

7.3.2.5 Other dynamic actions

Other dynamic actions, which in some specific cases would need to be considered, include vibrations due to wind turbulence, wave action, dynamic impact from floating debris and/or ice.

The potential for vibration loading due to wind turbulence and its effects on the spillway structure, including possible resonance response, should be considered in accordance with BS EN 1991-1-4:2005+A1:2010, along with any additional measures during construction specified in section 7.3.1.4. However, wave action on the inlet structure weir, piers and wind walls should be considered based on the guidance provided in BS 6349-1-2:2016+A1:2017.

Dynamic impact loads could be imposed by floating debris and/or ice on the weir, piers and training walls of the inlet structure as well as on the walls, slabs and other features of the spillway structure, especially where abrupt changes in direction, chute blocks or baffles are present. Such loading should be determined as a function of the size and density of the expected floating debris or ice, their approach velocity and structural displacement.

Useful guidance for determining the hydrodynamic pressures acting on stepped spillways is provided in 'Guidance for the Design and Maintenance of Stepped Masonry Spillways' (Defra, 2010) and in Hager (2020).

7.4. Structural design

7.4.1. Design situations

In accordance with BS EN 1990:2002+A1:2005, 'the selected design situations shall be sufficiently severe and varied to encompass all conditions that can reasonably be foreseen to occur during the execution and use of the structure'.

The structural design of open channel reservoir spillways and energy dissipators should consider, but not be limited to, the following design situations:

- temporary (construction) situation before side walls backfilling
- permanent situation with no flow
- permanent situation under 'safety check flood' (including 3m wide 'soft spot')
- permanent situation under 'design flood'
- permanent situation under 50% of 'design flood'

The 50% 'design flood' will allow the user to establish the density of the structure to the reduced water damping of the flow induced vibrations.

Other design situations that may need to be considered depending on the specific spillway arrangement and operation include:

- transient (first opening) flow situations during operation of gated spillways
- discharge of ice or floating debris under low flow conditions

Both of the above situations could be critical in determining the maximum dynamic impact actions.

7.4.1.1 Temporary (construction) situation before side walls backfilling

Under this situation, the structure walls should be loaded with the net wind pressure in accordance with BS EN 1991-1-4:2005+A1:2010 while the structure base should be loaded with the soil bearing pressure (soil reaction) resulting from the soil-structure interaction.

This situation may be critical for the design of the spillway base slab to resist hogging moments, especially if no outstands are provided to restrain end rotation. This situation could also be used to identify any requirements and constraints relating to the support and timing or removal of the wall formwork in order to efficiently control the adverse effects of wind action during construction.

7.4.1.2 Permanent situation with no flow

Under this situation, the structure walls should be loaded with earth pressure (refer to sections 7.3.1.3), external hydrostatic pressure (refer to section 7.3.1.2) and net wind pressure above ground level in accordance with BS EN 1991-1-4:2005+A1:2010, while the structure base should be loaded with the soil bearing pressure (soil reaction). All applied soil pressures should be compatible with the soil-structure interaction and resulting structural deflections and rotations.

This situation would normally be critical for the design of the spillway walls to resist bending and may be critical for the design of the base slab to resist sagging moments near the walls.

7.4.1.3 Permanent situation under ‘safety check flood’ (including 3m wide ‘soft spot’)

Under this situation, the external loading will be the same as specified in section 7.4.1.2, while the internal hydrostatic loading will in accordance with section 7.3.1.2. A 3m wide ‘soft spot’ will be considered while assuming fixed slab ends and ensuring that both hogging and sagging bending moments are adequately resisted.

Mean hydrodynamic forces and flow-induced vibration loading should be applied in addition to the internal hydrostatic loading in accordance with sections 7.3.2.2 and 7.3.2.3.

This situation may be critical for the design of the base slab of the spillway chute and for the design of the base slab and side walls of the stilling basin.

7.4.1.4 Permanent situation under ‘design flood’

This design situation is similar to that under the ‘safety check flood’ except that no ‘soft spot’ should be considered and that the characteristic internal vertical hydrostatic load should be based on the clear water depth predicted for the ‘design flood’.

This situation may be more critical than the ‘safety check flood’ situation due to the reduced water depth and damping of the flow-induced vibrations. In some cases, this situation could also significantly increase the flow-induced vibration at the bottom of the spillway chute where the hydraulic jump may take place on the chute slope.

7.4.1.5 Permanent situation under 50% of 'design flood'

This design situation is similar to that under the 'design flood' whereby the effects of reduced damping and hydraulic jump taking place on the chute slope would be even more pronounced.

7.4.2. Ultimate limit states design

In accordance with Eurocode (1990), in addition to the ultimate limits states (ULS) relating to structural failure by excessive deformation or rupture, the ULS related to failure caused by fatigue or other time-dependent effects should be verified (Eurocode, 1990). Therefore, the ULS should be verified for both the static and the dynamic loads inherent in the high velocity flow in reservoir spillways and energy dissipators.

7.4.2.1 Static design

The static design should be based on an appropriate representation of the soil-structure interaction and its effect on the mobilised lateral soil pressures, bearing pressure distribution and resulting internal forces and moments induced into the spillway structure. In this respect, the design should give due consideration to the respective rotations and deflections of all structural members. In order to identify the most critical loading condition, it is recommended that all structural deformations should be determined considering both the short-term and long-term loading effects.

7.4.2.2 Dynamic design

Since all spillway structures are susceptible to flow-induced vibrations, 'dynamic models of imposed loads should be considered' (Eurocode, 1991).

The dynamic design procedure is given in section 5.1.3 of Eurocode (1990), which stipulates that "the structural model to be used for determining the action effects shall be established taking account of all relevant structural members, their masses, strengths, stiffnesses and damping characteristics, and all relevant non-structural members with their properties" and that "the boundary conditions applied to the model shall be representative of those intended in the structure."

Since resonance effects are to be expected in reservoir spillways and energy dissipators due to the broad band spectrum of the inherent flow-induced vibrations, the "load model should be determined for special dynamic analysis" (Eurocode, 1991).

7.4.2.3 Fatigue verification

In accordance with Eurocode 2, Part 1, a "fatigue verification should be carried out for structures and structural components which are subjected to regular load cycles." Such load cycles could be generated by relatively regular and prolonged spillway events in spillways and energy dissipators subject to high velocity flows and resulting flow-induced vibrations.

In accordance with the FIB Model Code for Concrete Structures 2010, "fatigue design must ensure that in any fatigue-endangered cross-section, the expected damage D will not exceed a limiting damage D_{lim}. The verifications of this requirement can be performed according to 4 methods of increasing refinement' (FIB, 2013).

7.4.2.4 Seismic design

The UK national forewords to Eurocode (1998[1]) and Eurocode (1998[2]) state that "there are generally no requirements in the UK to consider seismic loading, and the whole of the UK may be considered an area of very low seismicity in which the provisions of EN 1998 need not apply. However, 'certain types of structure, by reason of their function, location or form, may warrant an explicit consideration of seismic actions" (Eurocode, 1998[1]).

It should be noted that the open channel spillway structures subject to this guide, by virtue of their nature characterised by relatively high stiffness and good lateral restraint, are not particularly vulnerable to seismic loading.

However, it is the responsibility of the designer, respectively of the qualified civil engineer, to establish whether statutory or other considerations require an explicit seismic design. Background information on the circumstances in which this might apply in the UK has been published in the BSI document PD 6698 (2009).

With regards to foundations, retaining structures and geotechnical considerations, the PD 6698 (2009) states that "the assessment of the liquefaction potential of soils using BS EN 1998-5 should be carried out taking account of the additional advice contained in the IStructE/AFPS Manual for the seismic design of steel and concrete buildings to Eurocode 8." It has been recognised that, in most cases, this would require specialist advice.

7.4.3. Serviceability limit states design

For serviceability limit states load combinations, the tensile stress in steel reinforcement should normally not exceed $0.8f_{yk}$ to avoid plasticity. Where the stress is due entirely to imposed deformations due to shrinkage of concrete and temperature variations, $1.0f_{yk}$ is acceptable (FIB, 2013).

Under compression, the concrete short-term stress should not exceed $0.6f_{ck}$ to avoid microcracking, while the long-term stress should not exceed $0.4f_{ck}$ in order for the creep strain to remain proportional to the initial elastic strain. This would avoid non-linear creep, which would lead to accelerated deformation of concrete elements (FIB, 2019).

While the Model Code 2010 states that “the limitation of tensile stresses in the concrete is an adequate measure to reduce the probability of cracking”, no stress limit is normally specified for concrete as cracking is allowed in areas exposed to tension.

7.4.3.1. Cracking

Crack width under load control cracking or displacement control cracking should be generally calculated in accordance with Eurocode 2 Part 1 and Part 3.

The design should meet the requirements of Tightness Class 1 in accordance with Eurocode 2, Part 3 in order to promote healing of cracks over time and therefore ensure the durability of the spillway structure. The maximum internal and external hydrostatic pressures should be considered in accordance with section 7.3.1.2. However, where baffles, blocks, sills or other features are placed on a spillway chute or within an energy dissipator to dissipate energy, a stagnation pressure corresponding to the full velocity head should be considered in controlling cracking around such features in addition to the respective mean hydrodynamic actions on these members.

The CIRIA Report C766 on Control of cracking caused by restrained deformation in concrete provides useful complementary guidance in accordance with Eurocode 2. The report also provides a method for checking that the reinforcement provided will be sufficient to control early-age cracking, while also being adequate for controlling cracks that may develop due to long-term deformations caused by temperature change and shrinkage (CIRIA, 2018).

Where cracking is due to the combined effect of imposed deformations and loads, the steel stresses at the cracks due to loads, as well as imposed deformations, should be considered (FIB, 2013). Such a combined effect is typical for spillway base slabs which are subject to hogging bending moments under the action of ground bearing or external hydrostatic pressure, while being restrained at their ends to accommodate imposed deformations. A method for calculating the reinforcement required to control the crack width under such combined action is presented in FIB (2019).

The stresses induced into the structural member by long-term loading may be analysed using an effective modular ratio $\alpha_{e,eff}=E_s/E_{c,eff}$ = 15 allowing for the effects of creep (FIB, 2019). However, this modular ratio corresponds to loading at 28 days approximately. Where concrete elements could be loaded at an earlier stage, for example, where during construction the spillway base slab is loaded by bearing pressure induced by the weight of the side walls, a higher creep coefficient and corresponding modular ratio may be used to reflect the reduced effective elastic modulus of concrete at the time of loading.

It should also be noted that the creep coefficient increases with reducing concrete strength and with increasing the concrete member surface area to volume ratio (as relatively thin members dry out quicker).

The concrete tensile strength used in the calculations should reflect the time when constant load on the structure is first applied.

The efficient control of cracking is significantly affected by the properties, composition and execution of the concrete used. It is therefore important that the concrete specification should ensure that all factors affecting the early-age and long-term thermal and shrinkage induced cracks are duly considered in accordance with the CIRIA Report C766 (CIRIA, 2018).

Special consideration should also be given to efficiently controlling the effects of cracking due to early plastic settlement of concrete as this could increase the risk of corrosion to the reinforcement and reduce the bond strength. This could be achieved by (CCAA, 2005):

- using concrete mixes with lower bleeding characteristics, for example, lower slump and more cohesive mixes
- increasing the ratio of cover to reinforcing bar diameter, that is, by increasing the cover or decreasing the size of the bars
- providing a nominal cover not less than twice the maximum aggregate size (CIRIA, 2010)
- avoiding the use of retarding admixtures, if practical

Re-vibration of the concrete after plastic settlement crack have formed could substantially eliminate plastic cracks and their consequences (CIRIA, 2010).

In addition, all good construction practices should be adhered to with regards to the placing, compaction and curing of the concrete.

7.4.3.2. Deformation

Deformation of members of the spillway structure are not normally likely to adversely affect its proper functioning or appearance. Therefore, deflections and rotations are not typically controlled against any limiting values.

However, where the external loading of the spillway base slabs or walls varies significantly between adjacent sections of the structure, it would be important to ensure that no significant differential deformations could occur. Such differential deformations could create offsets which would increase the flow turbulence and cause stagnation pressures with all ensuing adverse effects on the structure.

Also, rotation and deflection of the members of the spillway structure will impact the soil-structure interaction and therefore need to be determined as accurately as possible for both SLS and ULS (refer to section 7.4.2.1).

In calculating the long-term deflection, the mean long-term curvature should be calculated using the effective elastic modulus of concrete. The coefficient β, taking account of the influence of the duration of the loading or of repeated loading on the average strain in equation 7.19 of Eurocode 2, Part 1, should be taken as 0.5 to allow for the loss of tension stiffening with time. This would also allow for the effects of cyclic dynamic loading due to flow-induced vibrations.

7.4.3.3 Fatigue loading effects

Spillway structures are exposed to prolonged repeated flow-induced vibrations during their design life. This creates variable stress-strain conditions which could increase the cracking and deflection of these structures.

In this respect, fatigue cracks have been reported by USBR (1979) at the bottom of the centre walls of several stilling basins.

A method for calculating the increased deflections under fatigue loading in the SLS is presented in FIB (2013).

7.4.4. Other structural design considerations

7.4.4.1 Concrete cover

In accordance with section 4.15 of CESWI 7th Edition (2011) the nominal cover on the inside of spillway channel, which is in contact with flowing water, should be 40mm. Allowing for a deviation Δc_{dev} = 10mm, the minimum cover becomes 30mm. Considering the critical importance of ensuring the durability of spillway structures, and the increased risk of abrasion due to high velocity flow, it is recommended that the following nominal covers be used:

- 60mm for surfaces in contact with water within the spillway inlet structure and the conveyance structure
- 75mm for the surfaces in contact with water within the energy dissipator

The above concrete covers are lower than those specified in the US Army Corps of Engineers Engineer Manual, Strength design for reinforced concrete hydraulic structures, 1110-2-2104 (2016), that is, a minimum concrete cover of 75mm for spillway chute slabs and 150mm for stilling basin slabs subject to cavitation or abrasion erosion. This is due to the more moderate climate conditions in the UK and the relatively low spillway flow velocities involved.

7.4.4.1 Detailing of reinforcement

In accordance with Eurocode (1992), the rules of detailing of reinforcement given in Section 8 ‘may not be sufficient for elements subjected to dynamic loading caused by seismic effects or machine vibration, impact loading and to elements incorporating specially painted, epoxy or zinc coated bars’. The application of this clause could be extended to the dynamic loading caused by pressure fluctuations and hydrodynamic forces inherent in the reservoir spillways.

7.5. Construction details

7.5.1. Construction joint (CJ)

The positions of the construction joints (CJs) should be “specified by the designer and indicated on the drawings. If there is a need on-site to revise any specified position or to have additional joints, the proposed positions should be agreed with the designer.” (BS, 1987)

In accordance with USBR, for reservoir spillways, ‘”he CJ orientation is typically horizontal (separating one concrete placement from the next concrete placement, such as placing a spillway conduit arch section on the previously placed conduit base section). An exception is using CJs normal to the flow surface in tunnels. Vertical and/or diagonal orientation of a CJ can be satisfactorily achieved with appropriate levels of care and oversight during construction.” (USBR, 2014).

Where CJs are provided, “high quality workmanship is necessary when forming the joints to ensure that the load-bearing capacity of the concrete in the area of the joint is not impaired” (BS 8110, 1997) and to ensure that the CJs do not have the potential to compromise the effectiveness of the crack control, durability of the reinforced concrete members or cause leakage into the foundation.

Where a vertical construction joint is provided in a spillway wall, it “should be treated as a movement joint if it is considered that it may open sufficiently to permit the passage of liquid.” (BS 8110, 1997) In this case, the provisions of section 7.5.2 should apply.

Considering the difficulty of execution of vertical construction joints, it is recommended that their use be limited to tunnel spillway sections. For open channel spillways, it is recommended that contraction or control joints are used instead to separate adjacent concrete placements in accordance with sections 7.5.2 and 7.5.3.

7.5.2. Movement joints

Movement joints should be provided where “effective and economic means cannot otherwise be taken to limit cracking” (Eurocode 2, Part 3, 1992). In general, “the use of fewer contraction joints with slight increases in shrinkage and temperature reinforcement provides a more practical design with good service performance.” (USACE, 2016).

If necessary, for example where the spillway is located over the embankment, ‘hinged’ movement joints could also be provided to allow sufficient structural articulation to accommodate any differential settlement.

The location and spacing of movement joints “should be governed by the physical features of the spillway, temperature study results, concrete placement methods, and the potential concrete placing capacity.” (USBR, 2014).

Recommended movement joint details are provided in section 7.5.3.1 for both transverse and longitudinal spillway joints as a function of the spillway slab thickness.

Useful information and practical details of movement joints used in reservoir spillways is provided by Mason (2017).

7.5.2.1 Contraction joint (CrJ)

The contraction joint (CrJ) allows effective relief of tensile stresses and cracking induced by shrinkage and thermal contraction without risk of local corrosion of the reinforcement, which normally is not continuous across the joint. It also represents a suitable alternative to vertical CJs in spillway slabs as they provide reliable crack control and water-tightness and practically eliminate the risk of corrosion at the joint.

For spillways structures, the CrJ typically represents a formed unbonded surface incorporating a water-stop to prevent leakage (refer to section 7.5.2.5) into the foundation and a dowel to enhance the structural response to dynamic loading and to prevent damage of the water-stops due to vertical displacements (refer to section 7.5.2.6).

Useful guidance for the design of contraction joints for reservoir spillways is provided in the USBR Design Standard No.14, Appurtenant structures for dams, Chapter 3: General spillway design considerations.

The USBR standard specifies, among other things, that “transverse floor CrJs are normal (90 degrees) to the centreline of the spillway and normal to the slope of the flow surface” and that the “reinforcement is not continuous across CrJs to prevent any moment transfer’ except for floor CrJs ‘where plain reinforcing dowels may extend across the CrJs.”

It also states that spacing between contraction joints for use in concrete spillways typically ranges from 4.5m to 12.0m and highlights that large spacing (typically greater than 6m) could be more susceptible to shrinkage cracking. The standard recommends that “when evaluating large spacing of joints, considerations should be given to undertaking concrete mix designs and temperature studies to evaluate cracking potential and joint spacing.”

In this respect, USACE (2016) stipulates that ‘”or longer monoliths or concrete features, contraction joints within the monolith should be considered, and if used, should be spaced no more than 1 to 3 times the height of the monolith or the feature’s transverse (shorter) dimension.” It goes onto clarify that typically, “taller or wider features would tend toward the lower end of the stated range’, while ‘shorter features (2.4m and less) would tend toward the higher end of the stated range.”

The USBR standard provides details of flow surface contraction joints for rock and soil foundation and for flat and steep slopes as illustrated in Figures 7.3 and 7.4 (USBR, 2014[1]):

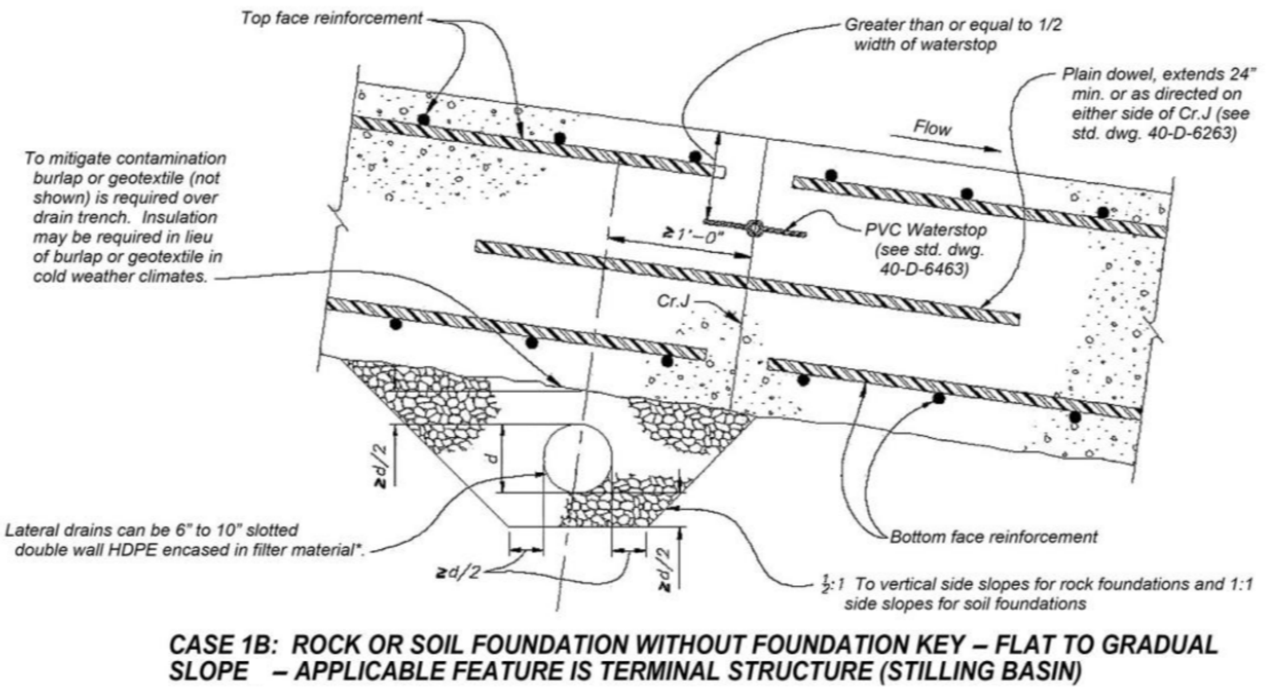

Figure 7.3: Recommended contraction joints for flat/gradual slope spillways on rock or soil foundation (after USBR, 2014[1])

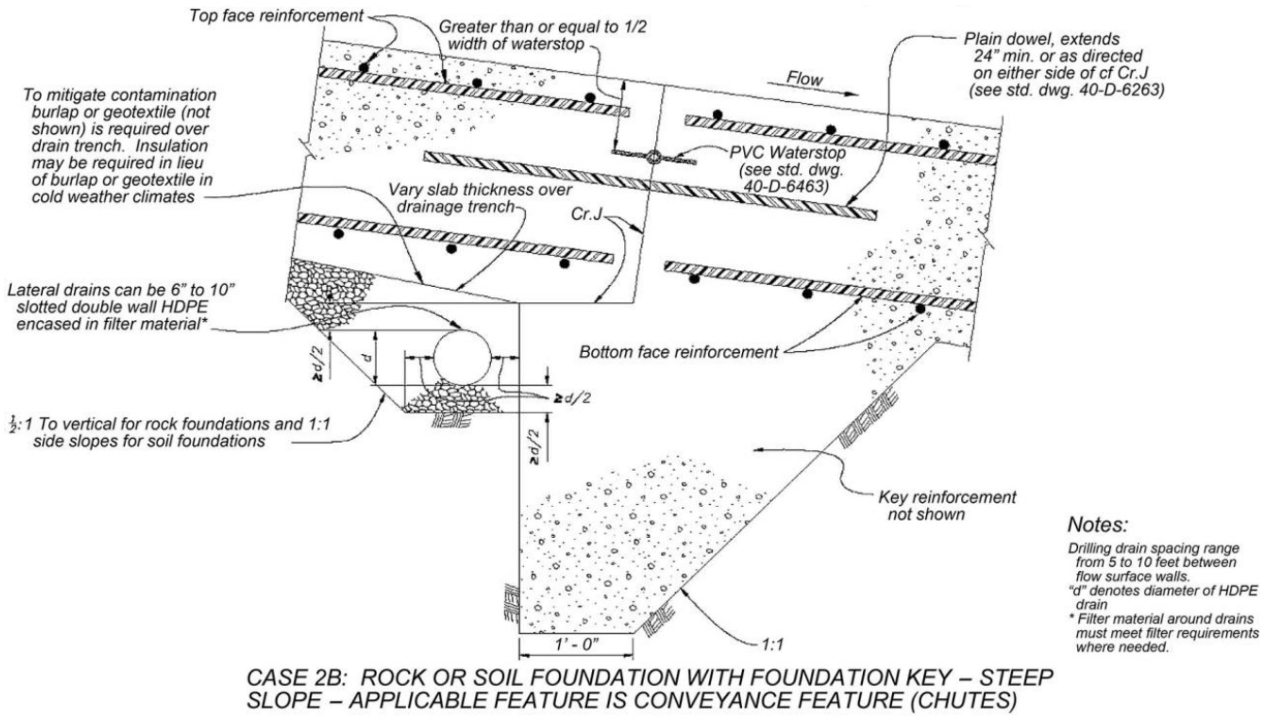

Figure 7.4: Recommended contraction joints for steep slope spillways on rock or soil foundation (after USBR, 2014[1])

It should be noted that where the slope is not too steep to pose a risk of sliding, the foundation keys could be omitted (refer to Figure 7.4). However, they would provide further protection against water injection into the foundation due to stagnation pressures and are recommended for use where potential failure of the spillway poses a risk to safety of the dam.

Where floor slabs can be constructed in alternate panels, the initial placement shrinkage of the concrete may then afford sufficient joint opening for subsequent expansion (USBR, 1987).

Further details are provided by USBR (2014[1]), including CrJs on rock foundation, longitudinal CrJs for flow surface slabs and transverse CrJs for flow surface walls.

7.5.2.2 Control joint (CtJ)

The control joint (CtJ) combines the features of the CJ and the CrJ. As with CJs "reinforcement is continuous across CtJs to allow moment transfer and can facilitate the bridging of the concrete feature over localised differential movement (settlement) of the foundation." (USBR, 2014). However, similar to CrJs they also represent a formed unbonded surface to relieve tensile stresses and cracking induced by shrinkage and thermal contraction and incorporate a water-stop to prevent leakage into the foundation, the vertical displacement of the joint being controlled by the continuous reinforcement.

The rules for the design of CtJs and their construction details recommended by USBR (2014) are practically the same as those for CrJs, except that the reinforcement is continuous across the joint and therefore no dowels are provided.

In addition, for closely spaced reinforcement, a different splice detail for the top face has been used by USBR (2014[1]) "to reduce the chance of delamination at the joint. Instead of locating the splice in the same plane as the reinforcement pattern (and potentially introducing a plane of weakness), the reinforcing bars are stopped on each side of the joint, and a single splice bar is placed below the reinforcement that splices to each bar on either side of the CtJ." This also increases the concrete cover at the joint and therefore further mitigates the risk of corrosion of the reinforcement. This detail is illustrated in Figure 7.5 below for the case of a steep slope spillway on rock or soil foundation (the reinforcement should be lapped on either side of the joint in accordance with Eurocode (1992)):

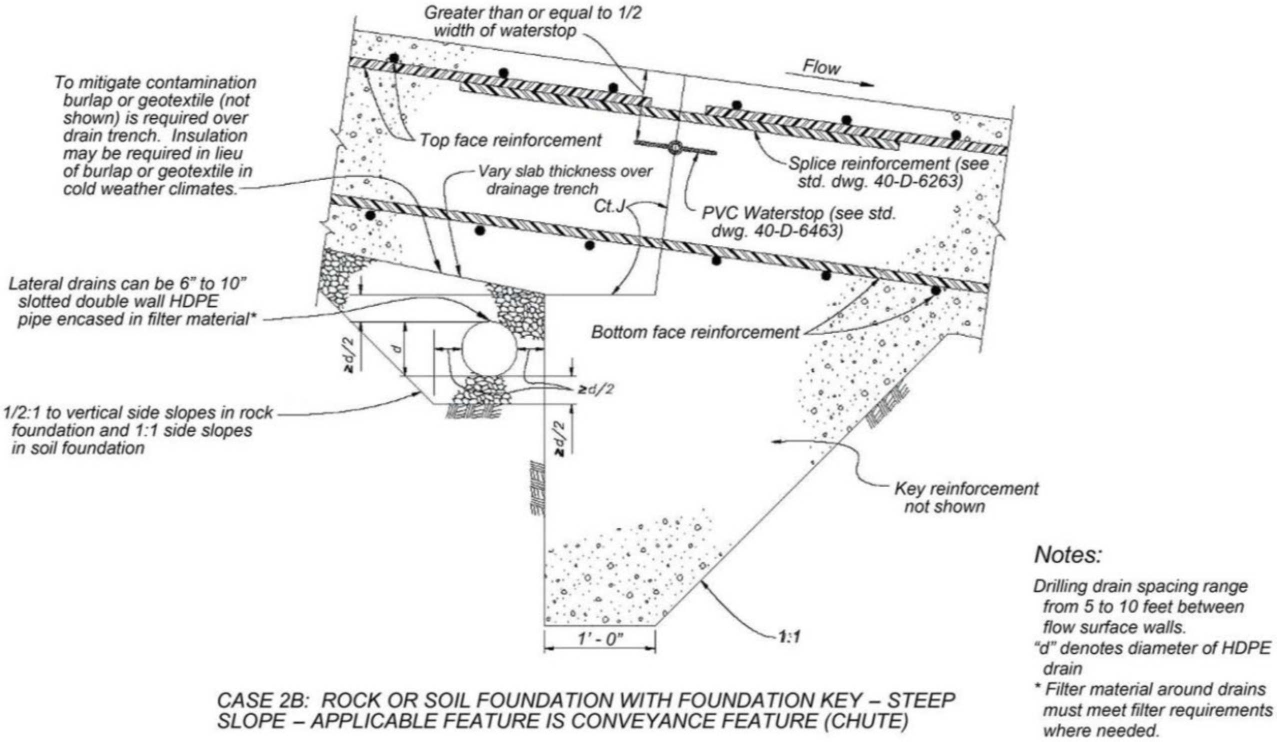

Figure 7.5: Recommended contraction joints for steep slope spillways on rock or soil foundation (after USBR, 2014[1])

The CtJs recommended by USBR therefore represent a suitable alternative to vertical CJs in spillways slabs where moment transfer is required as they provide reliable water-tightness, better mitigation of the risk of local corrosion, spalling and delamination, and do not rely on high quality workmanship for their effectiveness.

When using this joint detail, it is recommended that the splice reinforcement used is made of stainless steel.

It is recommended that the design of CtJs considers "'undertaking concrete mix designs and temperature studies to evaluate cracking potential and joint spacing."' (USBR, 2014)

7.5.2.3 Expansion joint (EJ)

An expansion joint (EJ) allows effective relief of compressive stresses due to thermal expansion by separating adjoining parts of the structure with a physical gap and therefore preventing the transfer of axial loads. It also relieves tensile stresses due to contraction and would normally allow rotation of adjacent concrete structures relative to each other. It could therefore be useful where there is a need to accommodate any differential settlement, for example where the spillway is located over the embankment.

Similar to the other movement joints, transverse floor expansion joints should be normal to the centreline of the spillway and normal to the slope of the flow surface.

The EJ gap is typically filled with a compressible joint filler and protected by a joint sealing compound. As with the other movement joints, the EJ also incorporates a water-stop to prevent leakage (refer to section 7.5.2.5) into the foundation and a dowel to enhance the structural response to dynamic loading and to prevent damage of the water-stops due to vertical displacements (refer to section 7.5.2.6).

Despite the benefits offered by EJs, according to USBR, “they are seldom applicable to a reinforced concrete spillway. Exceptions are EJs associated with spillway bridges, hoist decks, and parapet walls.” (USBR, 2014[1]). The Independent Forensic Team Report on the Oroville Dam spillway incident (2018) also states that “expansion joints on high velocity flow surfaces were not considered good practice at the time (and still are not today).” This is probably due to the difficult maintenance of the compressible filler material used in these joints (USBR, 1960) and the frequent need to replace the sealant material often damaged or removed by the high velocity flow.

Furthermore, “joints sealants frequently have a life considerably shorter than the design working life of the structure and, therefore in such cases, joints should be constructed so that they are inspectable and repairable or renewable.” (Eurocode 2, Part 3, 1992). Where this may not be possible, consideration should be given to omitting the expansion joints or adopting a design arrangement and using materials that would ensure the durability and correct functionality of the joint over the entire design life of the structure.

It is therefore recommended that EJs are only provided in the spillway chute and stilling basin where they are critically important in relieving compressive stresses, providing structural articulation to accommodate differential settlement. In this respect, expansion joints would not normally be required where the spillway chute or stilling basin invert is cast on competent rock and is fully anchored into it.

EJs should be designed with joint filler, sealant and bond breaker materials which are compatible with each other and with the water-stop material. Where EJs are not used in conjunction with a foundation key, a suitable ‘dirt-stop’ should be provided to prevent the infiltration of soil material from the foundation or the surrounding backfill such as blinding concrete or geotextile covered with treated timber.

Where high flow velocities are present, the EJ could be provided with a removable steel cover plate to prevent damage to the joint.

7.5.2.4 Concrete spalling and delamination at joints

Concrete spalling and delamination could occur at joints other than EJs due to thermal expansion at the concrete surface and corrosion of reinforcement at open CJs and CtJs.

To eliminate the risk of this happening, the design of the respective structural members should allow them to withstand the compressive thermal forces involved.

Where CrJs are provided, “a surface blockout may be considered to reduce the effects of temperature-induced (thermal) expansion” (USBR, 2014[1]), as shown in Figure 7.6:

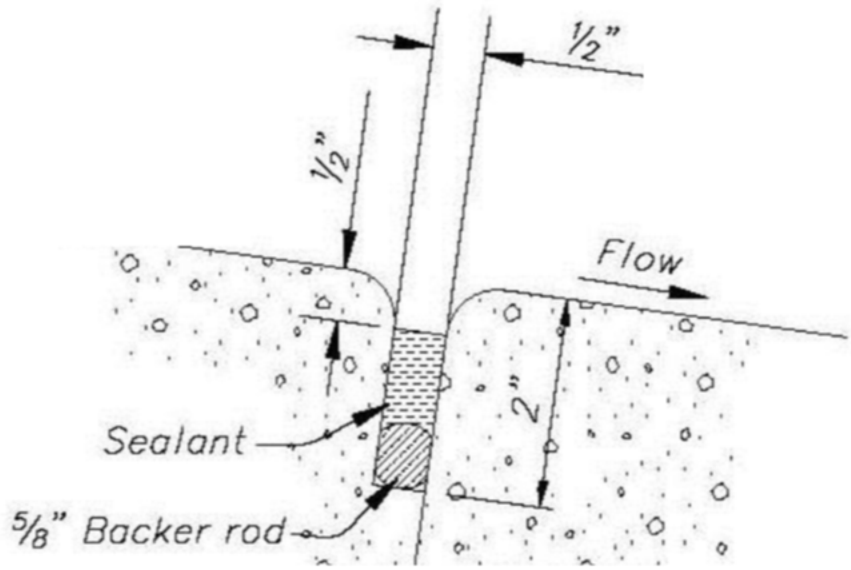

Figure 7.6: Surface block-out detail at contraction joints (after USBR, 2014[1])

USBR recommends that blockouts should not be considered where the average flow velocity exceeds 15m/s due to the risk of cavitation. In addition, “filler material (sealant) should adhere to the sides of the blockout to limit potential accumulation of water and other material.” (USBR, 2014[1]).

Only stainless-steel tying wire should be used in order to eliminate the risk of corrosion and concrete spalling due to any wire ties protruding into the concrete cover zone.

7.5.2.5 Water-stops

Water-bars made of elastomer (rubber) or PVC material should be used as water-stops at movement joints. They should be suitable for resisting the expected dynamic loading and structural displacements.

Only internal water-bars should be used at movement joints in order to cater for the risk of stagnation pressures developing which could exceed the external hydrostatic or bearing pressure.

Normally, ribbed water-bars with central ‘bulbs’ or other voids should be used at movement joints in order to accommodate expansion and/or contraction and the inherent dynamic loading and resulting structural displacements.

Where movement joints are designed in accordance with this guidance, water-bars should be able to resist a stagnation pressure corresponding to at least half the velocity head in order to allow for any construction tolerances and spalling effects (refer to section 7.5.2).

Where movement joints cannot be avoided around baffles, blocks, sills or other features placed on a spillway chute or within an energy dissipator to dissipate energy, their water-bars should be able to resist a stagnation pressure corresponding to the full velocity head.

The general guidance for locating and sizing PVC water-stops provided by USBR (2014[1]) include:

- the size (overall width) of the water-stop is also based on the hydraulic head (hydrostatic and/or stagnation pressure)
- the overall width of the water-stop should not be greater than the thickness of the concrete slab
- the dimension from the concrete face or surface to the embedded water-stop must not be less than half the width of the water-stop
- the width of the water-stop must be at least 6 times the maximum sized aggregate (MSA) used in the concrete mix design

Base-seal type water-stops could be used as additional dirt-stops on the earth-side of spillway expansion joints to prevent the infiltration of backfill material.

All water-stops should be installed in accordance with manufacturers' instructions. Manufacturers typically recommend water-stops be installed in a shallow V-shape, particularly in sloping slabs, to allow air to escape from under the arms of the water-bar when compacting the concrete to reduce risk of voids.

Any water-bar intersections should be factory welded, while butt welding could be performed on site.

Care should be taken to provide secure fixing of the water-stops during construction, typically by wiring them to sufficiently robust sections of reinforcement.

Guidance on the main design principles for water-stops is provided in section 4.8.4.

7.5.2.6 Dowels

Only plain round stainless-steel dowels with sliding sleeves and caps allowing longitudinal movement should be used at spillway contraction and expansion joints.

Dowels would normally be provided at all movement joints in order to enhance the structural response to dynamic loading and to prevent damage of the water-stops due to vertical displacements.

Dowels should be provided at all transverse joints, irrespective of whether they are keyed or 'toggled' in order to eliminate the risk of joint opening and damage to the water-stop as a result of structural response to dynamic loading or differential uplift pressure/settlement across the joint. However, they could be omitted at 'toggled' longitudinal movement joints if it could be demonstrated that the structural response to dynamic loading cannot cause damage to the water-stops and that there is no risk of adverse differential settlement.

Where no foundation keys or ‘toggles’ are provided at transverse movement joints, for example in accordance with Figure 7.1, stainless steel sleeves should be used for which the play between dowel and sleeve does not exceed 1mm. This would practically eliminate the risk of vertical offsets forming in the event of differential settlement, therefore minimising the potential for stagnation pressures developing. It would also prevent putting extra stress on the water-stop and would reduce the resistance to movement across the joint by limiting the vertical dowel misalignment due to settlement.

Dowels should be aligned parallel to the finished surface of the slab. They should be positioned approximately at mid-depth from the surface level of the slab. Where foundation keys or ‘toggled’ joints are used, dowels should be positioned approximately at mid-depth from the surface level of the part of the slab above the key or ‘toggle’ (refer to section 7.5.3).

Useful guidance for the design and installation of dowels at movement joints of ground-supported concrete slabs is provided in Technical Report 34, Concrete industrial ground floors, 4th Edition, The Concrete Society, 2016.

7.5.3. Construction arrangements and details

The construction arrangements and details are provided to clarify and illustrate the design concepts set out in this guide. Alternative arrangements and details could be used where the specific site and loading conditions justify them.

7.5.3.1 Movement joints

The movement joint details shown in Figure 7.7 below are recommended for use at transverse and longitudinal spillway joints as a function of the spillway slab thickness. These details do not include the drainage, filter and insulation details, which are provided in section 8.

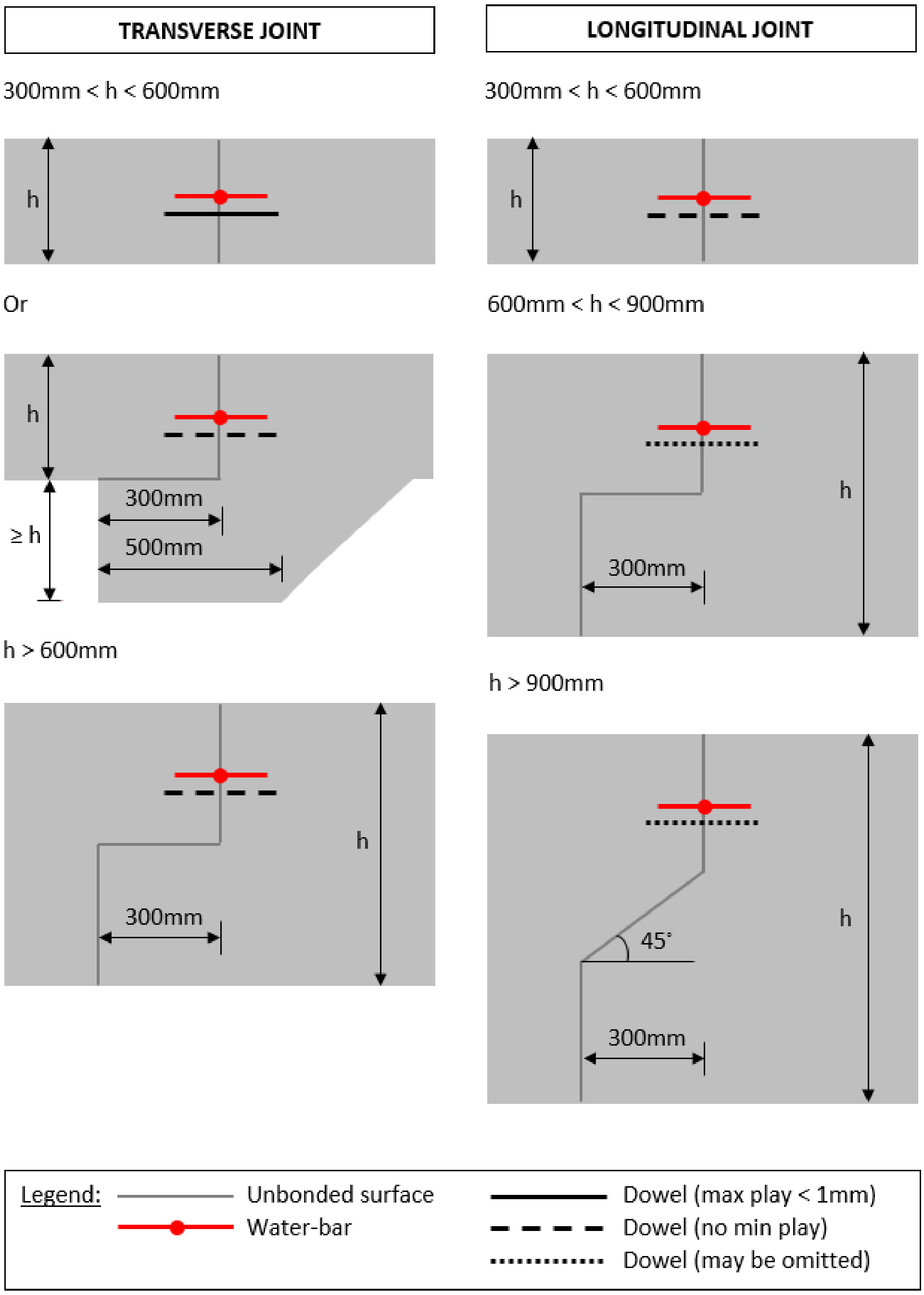

Figure 7.7: Movement joint details as a function of slab thickness

7.5.3.2 Spillway cross section

The spillway cross section shown in Figure 7.8 could be used where the spillway base slab is relatively wide, and it may not be practical to design it as a single span between the side walls (single monolith):

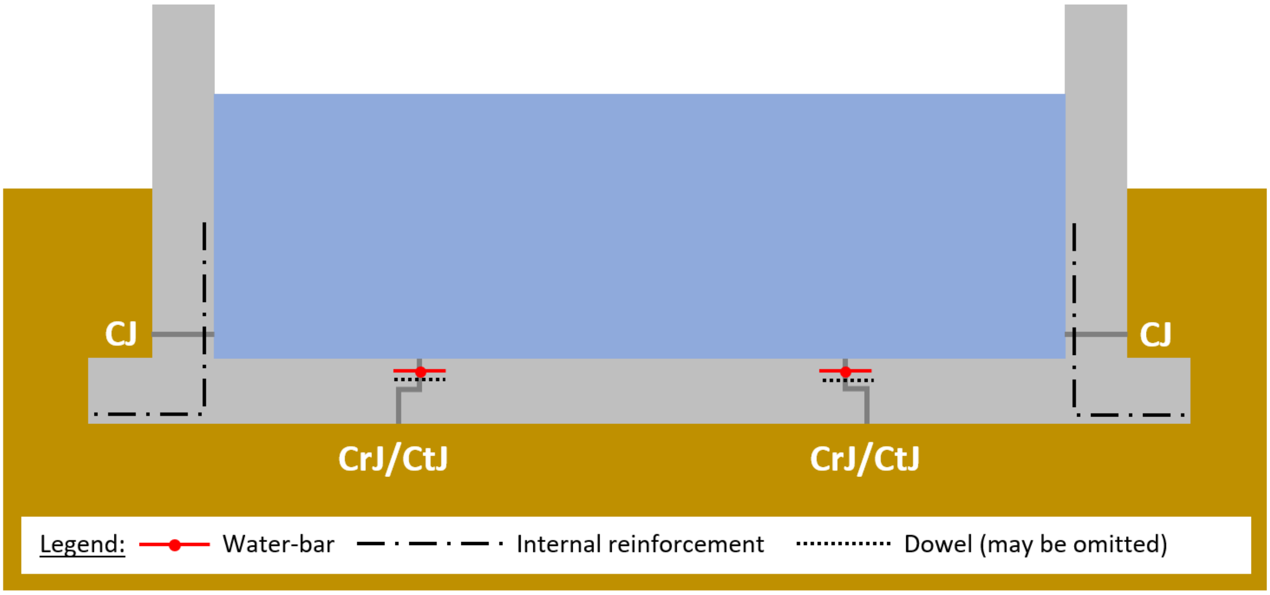

Figure 7.8: Spillway cross section

This arrangement provides the following benefits:

- The movement joints in the base would relieve tensile stresses and cracking induced by shrinkage and thermal contraction.
- With thinner slabs, the base would act as a 'Gerber' beam (cantilever span system) with the movement joints acting as 'hinges'. This would reduce the bending moments and would provide relative structural articulation to accommodate minor uneven settlement and reduce stresses within the base.
- The outstands ('T' section) would allow the mobilising of an additional weight of soil, as well as the soil shear strength, in resisting uplift. They would reduce rotation at the base of the side walls, therefore reducing the maximum bending moment. Where relatively thin structural elements are used, they would also facilitate the anchorage of the internal tensile reinforcement of the side walls for full moment transfer as recommended by USACE (2016).

With this arrangement, it has to be ensured that the side wall section is "stable against lateral sliding by foundation restraint." (Mason, 2017). This should be achieved via friction at the base or anchorage to the foundation, rather than relying on the passive resistance of the soil behind the side walls, which may cause opening of the contraction/control joints before it is mobilised.

7.5.3.3 Stilling basin long section

Flow in the stilling basin is subject to high turbulence with the maximum pressure fluctuations occurring within the first 30% of the hydraulic jump (Vasiliev and Bukreyev, 1967; Fiorotto, 1992).

In this respect, Mason (2004) recommends that “joints should be adjusted to avoid such zones, or made very secure, to avoid the injection of such high pressure to other, low-pressure zones of the basin slab.”

Therefore, where possible, and as a minimum, no movement joints should be located between the most upstream end of the hydraulic jump (when submerged at lesser flows or due to increased tailwater depth) and the end of the 30% zone as shown in Figure 7.9:

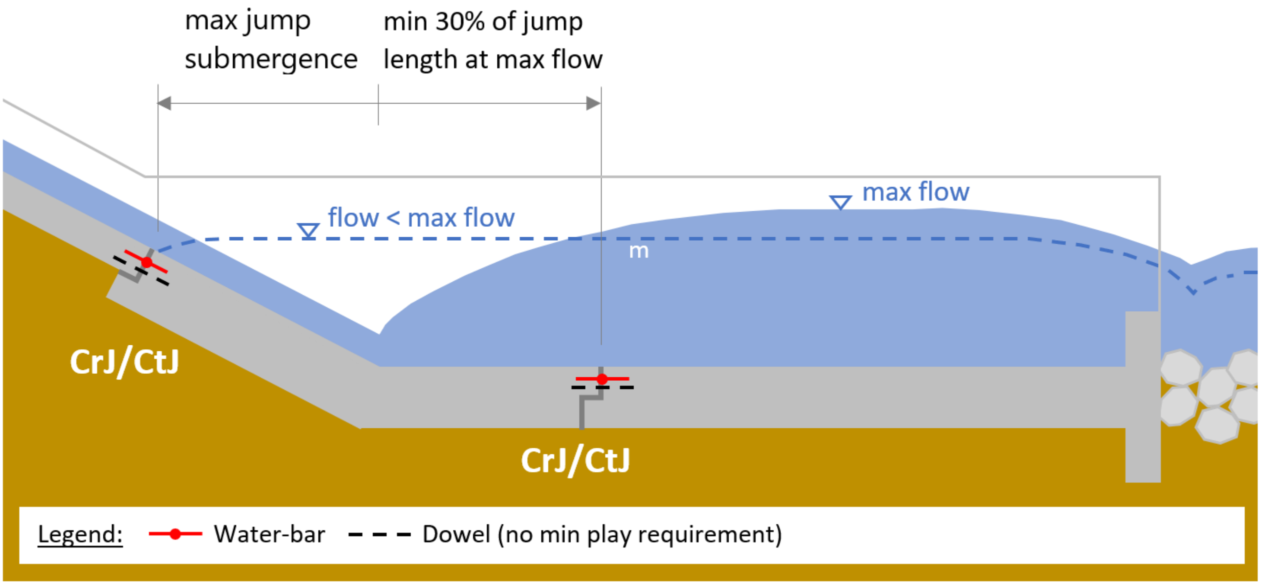

Figure 7.9: Bottom of chute and stilling basin long section

Given the significant dynamic uplift pressures developing as a result of the hydraulic jump pressure fluctuations (refer to section 8), where possible, for example in smaller stilling basins, it would be preferable to omit movement joints altogether in order to improve the resistance of the stilling basin structure to dynamic uplift.

Also, “individual slabs should be made as large as possible, owing to the fact that mean values of instantaneous pressures reduce with average area.” (Mason, 2004).

Page Break

8. Geotechnical design

This section provides general guidance relating to the main aspects of the geotechnical design of reinforced concrete open channel reservoir spillways, including the required ground investigations, foundation preparation and specific limit state design considerations.

It also provides specific guidance for the design of spillway drainage systems, seepage cut-offs and anchorage systems.

8.1. Ground investigations

The geotechnical design of reservoir spillways should be supported by adequate ground investigations, which should be planned following evaluation of the available information and documents and carried out in accordance with Eurocode 7, Part 2 (Eurocode, 1997[2]) and BS 5930:2015, Code of practice for ground investigations.

The ground investigations should provide a “description of ground conditions relevant to the proposed works and establish a basis for the assessment of the geotechnical parameters relevant for all construction stages.” (Eurocode, 1997[2]).

In accordance with Eurocode (1997[2]), “the composition and the extent of the ground investigations shall be based on the anticipated type and design of the construction, e.g. type of foundation, improvement method or retaining structure, location and depth of the construction.” Therefore, the required ground investigations will largely depend on the preliminary design locations and options considered as well as any subsequent iterations required. Similarly, the further ground investigations that may be required during the design investigation phase could be dictated by the needs of the structural, geotechnical and other design discipline needs.

8.2. Foundation preparation

The spillway foundation should generally have sufficient strength to prevent excessive settlements and resist sliding, adequate permeability and low erodibility. Soil foundations should also not be susceptible to swelling, frost action weathering and liquefaction. The latter effect could be triggered as a result of soil saturation and flow-induced vibrations.

Where the foundation present may not satisfy these requirements, various foundation preparation and ground improvement measures could be taken to improve its characteristic. These measures should be clearly specified by designers and are briefly summaried in what follows.

8.2.1. Rock foundations

Rock foundations are generally acceptable subject to adequate foundation preparation which may include shaping, dental treatment, grouting and clean-up.

Shaping of rock foundations involves achieving the intended design profile wihout any abrupt changes and/or breaks. It may also involve excavating unsuitable/weathered rock and subsequently filling any such purposely created depressions, inadvertently created irregularities or any encountered faults and/or shear zones with grout or dental concrete. Where shaping requires blasting, ‘proper procedures are essential to ensure that the permeability and strength of the rock foundation is not adversely affected’ (USBR, 2014).

Grouting could also be required in some cases in order to “establish an effective seepage barrier and to consolidate the foundation.” (USBR, 2004[1])

Cleaning rock foundations typically involves removing loose rock material using air/water jet or by hand and removing water by vacuuming, providing well points or other methods in order to enhance the strength and permeability at the contact area.

The rock foundation should be “washed or wetted before placing concrete to achieve a saturated surface dry (SSD) condition”, whereby “the foundation surface pores are saturated and free surface water and puddles have been removed from the surface of the foundation.” (USBR, 2014[1]).

Useful guidance for preparing rock foundations, including the specification of grout and dental concrete requirements, is provided by USBR (2014), USBR (2013) and USBR (2004).

8.2.2. Soil foundations

Where unsuitable soil foundation material is present, it should be either removed and replaced with suitable pervious material or engineered fill (material selected, placed and compacted in accordance with the specification for the earth works). Alternatively, it should be treated via geotechnical processes typically aimed at improving the soil strength in order to reduce settlement.

Typically, “soils that may be suitable foundation materials for a spillway but may require some additional evaluation, design, and foundation preparation or treatment are fine-grained soils (silts and clays), having low to medium compressibility.” (USBR, 2014[1]).

Soils which are susceptibe to suffosion/suffusion should either be removed and replaced with suitable material or filters should be provided in order to eliminate the risk of internal erosion.

Soils which are susceptible to liquefaction should normally be removed and replaced with an appropriate amount of gravel or engineered fill in view of the risk of liquefaction posed by the flow-induced vibrations inherent in high velocity flows.

The shaping for soil foundations typically requires that “all organic or other unsuitable materials, such as stumps, brush, sod, and large roots should be stripped and removed. Additionally, all pockets of soil significantly more compressible than the average foundation material should be removed and replaced with engineered fill. All irregularities, ruts, and washouts should be removed and replaced with engineered fill.” (USBR, 2014[1]).

The slope of excavations adjacent to embankment dams may need to be sufficiently flat (in the range of 4H:1V) in order to “ensure the backfill adjacent to the spillway can be effectively tied (compacted) into the existing embankment dam cut slopes.” (USBR, 2014[1]). Alternatively, suitable benching of the excavation could be provided. Where this is critical for ensuring the permeability of the backfill and its contact with the embankment, it should be clearly shown on the construction drawings and included in the specification for the earth works.

Where the ground slopes down towards the spillway, suitable drainage could be provided at the back of the wall to prevent groundwater build up during construction.

Cleaning soil foundations “should include removing loose or disturbed materials missed by machine excavation that will not be suitable foundation even after compaction (if needed).” (USBR, 2013). It would also involve water removal, typically via gravel sumps or well points.

8.3. Geotechnical design of reservoir spillways

The geotechnical design of reservoir spillways is an iterative process relying on the close interaction and combined input from all disciplines involved. At the preliminary stage, it goes hand-in-hand with the civil and hydraulic design to determine the optimal alignment, gradient and geometry of the spillway structures. Similarly, the detailed design is largely concerned with soil-structure interaction and drainage design and therefore involves communication with the structural, hydrogeology and hydraulic disciplines.

8.3.1. General

The geotechnical design of the spillway structure should be carried out in accordance with Eurocode (1997) and the applicable British Standards.

A conceptual site model and ground model of the geotechnical and hydrogeological conditions should be developed as part of the geotechnical design, providing adequate characterisiation of the project site. It should provide “definitive information concerning the geological setting and stratigraphy of the site, permeability and groundwater conditions, geotechnical characteristics and properties of the soil and rock materials, and the foundation design parameters and seepage control design factors” (Mack and others, 2004) for the spillway structure.

This section provides complementary non-conflicting guidance with regards to some specific design considerations relating to the most critical limit states inherent in reinforced concrete open channel reservoir spillways, including bearing resistance failure and/or excessive settlement, failure by uplift and failure by sliding at the base.

The other common limit states, which are less critical for this type of reservoir spillway have not been specifically considered. Failure due to internal erosion has not been discussed in detail either assuming that measures such as removal and replacement or filtering of susceptible soil material will be provided.

Useful guidance for the assessment of the potential for internal errosion and its remediation is provided in the ICOLD Bulletin 164 ‘Internal erosion of existing dam, levees and dikes and their foundations (2017) and in the USBR Guide ‘Best Practice in Dam and Levee Safety Risk Analysis – Chapter D-6’ (2019).

The effect of flow-induced vibrations present in reservoir spillways on the potential for soil liquefaction has also been discussed (refer to section 8.3.5).

8.3.2. Bearing resistance failure and excessive settlements

These interrelated limit states could be relevant where a spillway structure with high side walls, having the potential to run full, is founded at a shallow depth on a relatively soft man-made fill, such as the downstream shoulder of an earth-fill embankment. This could be the case where the design optimisation of the spillway chute and stilling basin installed over an earth-fill embankment gave preference to using a narrow and deep structure following the downstream slope with minimal excavation.

This could also occur where the spillway structure has been founded on artificially raised ground using engineered fill in order to accommodate an idealised hydraulic performance or to reduce design or construction cost.

Other design arrangements where these limit states may be relevant include inlet structures with a bridge upper structure, inlet weir structure creating a high eccentricity (by virtue of the weight of the inlet weir), fill placed above the natural ground behind spillway side walls adding additional vertical load due to skin friction or the presence of base slab outstands.

Where relatively thin spillway base slabs are designed, the bearing pressure could be greater under the side walls or separating walls which would increase settlement in these areas.

The bearing resistance of the foundation material should be calculated using the analytical method presented in Appendix D of Eurocode (1997) or another commonly recognised analytical method.

Where an inclination of the foundation and/or ground surface is present, it should be considered in the calculation of the bearing resistance.

The bearing resistance should be checked with both drained and undrained parameters as the condition of the foundation could change overtime.

It should be noted that in accordance with Cl. 6.6.2(16) of Eurocode (1997), "for conventional structures founded on clays, the ratio of the bearing capacity of the ground, at its initial undrained shear strength, to the applied serviceability loading should be calculated (see 2.4.8(4)). If this ratio is less than 3, calculations of settlements should always be undertaken. If the ratio is less than 2, the calculations should take account of non-linear stiffness effects in the ground."

The calculation of the bearing resistance should consider the potential decrease of the soil shear strength, including the reduction of the angle of shearing resistance and cohesion, with increase of the soil saturation ratio (Yoshida and others, 1991).

Consideration should also be given to the potential decrease of the soil shear strength under the effect of vibrations induced by flow, waves or traffic (Ermolaov and Senin, 1968). In this respect, Eurocode (1997) stipulates:

- in geotechnical design, movements and accelerations caused by vibrations and dynamic loads should be considered for inclusion as actions (Cl. 2.4.2(4))
- the possible effect of vibration on fill and collapsible soils should be considered in the calculation of settlements (Cl. 6.6.2(9))
- foundations for structures subjected to vibrations or to vibrating loads shall be designed to ensure that vibrations will not cause excessive settlements (Cl. 6.6.4(1))
- the effects of vibrations should be considered in the assessment of the overall stability of and movements in the ground (Cl. 11.3(2))

A separate discussion on the effect of flow-induced vibrations present in reservoir spillways on the potential for soil liquefaction is included in section 8.3.5.

8.3.3. Failure by uplift

The uplift failure limit state at spillways should normally consider the destabilising effect of the external hydrostatic pressure acting at the base of the structure (refer to Cl. 7.3.1.2) on the one hand and the stabilising effects of the weight of the structure, any added weight of soil (where base slab outstands are provided) and the resistance provided any anchor and friction forces on the other. In this respect, it should be noted that in accordance with Eurocode (1997), the resistance to uplift by friction (acting on the side walls) or anchor forces (acting at the base of the structure) may also be treated as a stabilising permanent vertical action.

As discussed in section 7.3.1.3, where the spillway side walls are backfilled with clay soil material, the possibility that the backfill within the zone of seasonal movements of clay soils could shrink away from the walls should be considered. Such shrinkage would partially reduce the resistance to uplift provided by the friction force between the spillway side wall and the backfill material. For this reason, it is good practice to provide the spillway base slab with outstands, therefore mobilising the internal soil friction in resisting the uplift force (refer to section 7.5.3.2).

The uplift check would normally assume that the spillway structure is empty when the external hydrostatic pressure reaches its maximum. However, in the case of stilling basins, where the maximum external hydrostatic pressure could be governed by the tailwater level, this assumption may not be quite realistic. This is because during such a design situation the stilling basin will be operating and therefore should contain a large volume of water providing stabilising effect. While this may well be the case in situations where the velocity of the incoming supercritical flow is not too high or where the hydraulic jump is significantly submerged, there are situations where the stabilising effect of the water present in the stilling basin may not be present or could be significantly reduced. Such situations include:

- isolation of the stilling basin from the tailwater by stoplogs for inspection and/or maintenance
- stilling basin sweepout. This situation could occur by design, for example, when passing the 'safety check flood'. It could also occur due to a natural exceedance or underestimation of the design flood, subsequent increase of the design flood or modelling uncertainties and errors. Where the tailwater depth is generated by the natural watercourse, such a situation could occur as a result of changes occurring within the receiving watercourse during major storm events. Any of these situations would result in a very shallow supercritical flow water depth within the stilling basin
- very high velocity of the incoming supercritical flow causing significant pressure fluctuations within the zone of the hydraulic jump roller. Recent research on this matter concluded that "in the case of sealed joints, the pressure underneath the slab is constant and the uplift force results from the pressure fluctuations acting on the slab" (Barjastehmaleki and others, 2016)

Considering the numerous factors that could potentially reduce the stabilising effect of water contained within stilling basins, as a conservative approach to assessing the uplift limit state for such structures, it is recommended that the most critical of the 3 situations below be used for design purposes:

- Maximum external ground water level and empty stilling basin.
- Maximum external ground water level or maximum tailwater level (whichever is the greatest) and supercritical flow depth persisting within the stilling basin. This situation should be considered where there is a current or anticipated long-term risk of stilling basin sweepout due to the above-mentioned or other reasons.
- Maximum external ground water level or maximum tailwater level (whichever is the greatest) and hydraulic jump forming within the stilling basin. In this situation, the negative pressure fluctuations occurring within the hydraulic jump could generate an increased uplift pressure in accordance with the design approach presented in 'Design of Stilling Basin Linings with Sealed and Unsealed Joints' (Barjastehmaleki and others, 2016).

The above approach would provide a more robust and realistic assessment of the uplift limit state than the approach adopted in the past by USBR (1987), according to which “flotation stability is computed assuming water to the elevation of the outlet channel and no water inside the basin.”

As mentioned in section 4.6.1, where all design measures recommended in this guide are taken to prevent the development of stagnation pressures, a residual stagnation pressure equivalent to 20% of the velocity head should be considered for the purposes of assessing the uplift failure limit state in order to allow for any remaining uncertainties.

8.3.4. Failure by sliding

The failure by sliding limit state should normally be assessed for selected surfaces at or near the foundation-surface interface. However, consideration should also be given to the potential for deep rotational or translational failure within the soil mass where such a potential may exist.

Both transverse and longitudinal sliding should be considered via assessment of a series of critical cross sections along the length of the spillway which may not be perpendicular to the spillway axis or parallel to each other. It should be noted that the direction of transverse sliding may switch along the length of the spillway, as it is common for the valley side to provide the destabilising action at the top of the spillway, whereas the dam may provide the destabilising towards its base.

The sliding resistance should be assessed with both drained and undrained parameters as the condition of the foundation could change overtime.

In accordance with Cl. 6.5.3 of Eurocode (1997), “the design friction angle δ_d may be assumed equal to the design value of the effective critical state angle of shearing resistance $\phi'_{cv;d}$.” However, for spillway structures it is recommended that this value be used only where foundation keys, cut-offs or other features that would promote shear within the soil rather than at the soil-structure interface are provided. In all other cases, the design friction angle could be conservatively taken as 2/3 of the effective critical state angle of shearing resistance.

Where a separating layer (polyethylene or other) has been used between the base slab and the blinding concrete in order to reduce restraint to thermal contraction and shrinkage, or to isolate the drainage provisions from adjacent concrete placements, the reduced friction at the interface between the concrete and the blinding concrete should be considered in the assessment of this limit state.

Consideration should also be given to the potential for a decrease in the soil shear strength, including the reduction of the angle of shearing resistance and cohesion, with increase of the soil saturation ratio and under the effect of vibrations induced by flow, waves or traffic (refer to Cl. 8.3.1).

In accordance with Cl. 6.5.3(7) of Eurocode (1997), “the possibility that the soil in front of the foundation may be removed by erosion or human activity shall be considered” in calculating the ground resistance in front of the structure. In this respect, Cl. 9.3.2.2 stipulates that “the level of the resisting soil should be lowered below the nominally expected level by an amount Δa, which for a cantilever wall should equal 10 % of the wall height above excavation level, limited to a maximum of 0.5 m” where normal degree of control over the level of the surface is applied.

Any effective cohesion c’ should be neglected (Eurocode, 1997).

Where foundation keys or cut-off walls are provided under the spillway, the maximum hydrostatic pressure that could develop upstream of them should be considered. Where a reliable drainage system is provided upstream of such features to control uplift, the design hydrostatic pressure should be taken to the soffit of the transverse collector drains.

8.3.5. Potential for liquefaction

The dynamic actions inherent in the high velocity flow in reservoir spillways referred to in section 7.3.2 could potentially cause liquefaction of susceptible foundation, filter or backfill soil material where conditions may exist, such that it could become partially or fully saturated.

This would have the effect of reducing the shear strength of such soil material which could cause bearing pressure or sliding failure as well as excessive settlements.

In this respect, Eurocode (1997) stipulates:

- “actions, which are applied repeatedly, and actions with variable intensity shall be identified for special consideration with regard to, e.g. continuing movements, liquefaction of soils, change of ground stiffness and strength” Cl.2.4.2(7)
- “precautions should be taken to ensure that resonance will not occur between the frequency of the dynamic load and a critical frequency in the foundation-ground system, and to ensure that liquefaction will not occur in the ground” 6.6.4(2)

Liquefaction could also cause a significant increase in the uplift pressure and horizontal earth pressures acting on the spillway structure as shown in Figure 8.1 (Pavlov, 2021):

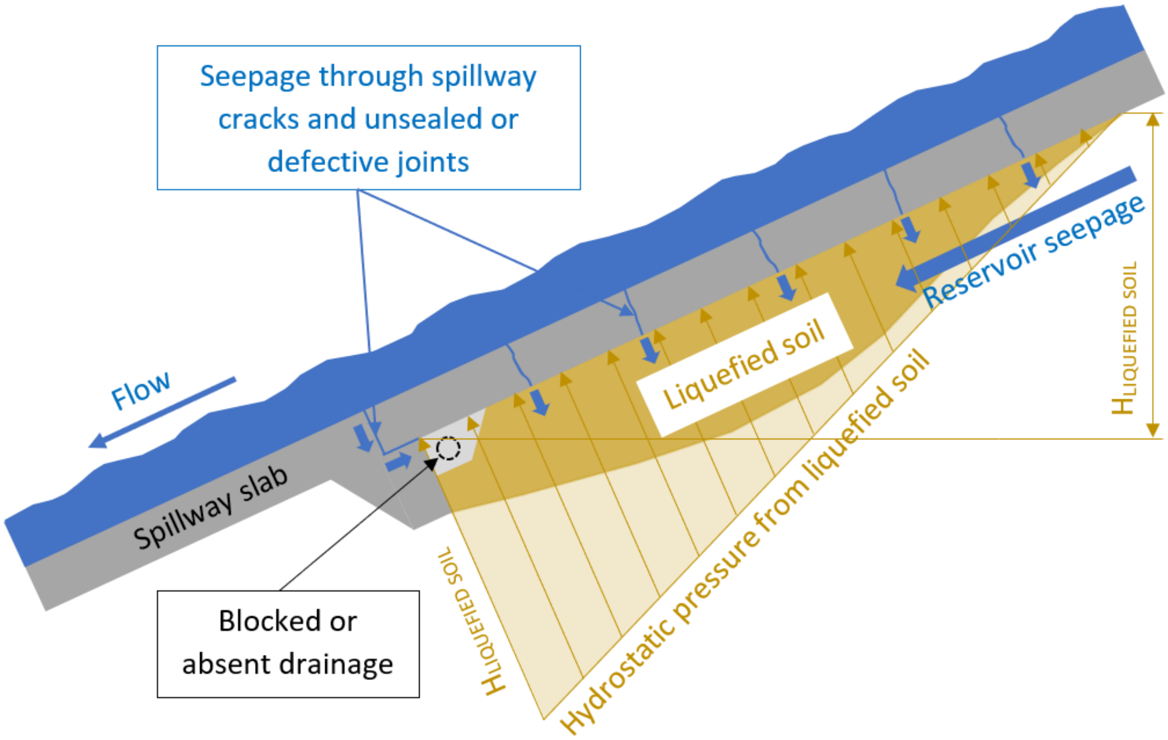

Figure 8.1: Hydrostatic loading on spillway slab due to soil liquefaction

An evaluation of the liquefaction susceptibility should therefore be carried out for all soil, drainage and filter material at the foundation or around the spillway structure considering their thickness and specified degree of compaction.

The guidance for assessment of the liquefaction potential of soils provided in Eurocode (1998[2]) could be used subject to the recommendation made in PD 6698 (2009) to consider the additional advice contained in IStructE/AFPS (2010).

Further guidance on this subject could be found in the USBR Design Standard No.13, Embankment Dams, Chapter 13: Seismic Analysis and design (2015).

In assessing the potential for liquefaction, it should be noted that the flow-induced vibrations within the energy dissipation structure are an order of magnitude greater than those in the spillway chute (refer to section 7.3.2.3).

8.4. Specific geotechnical design considerations

This section provides guidance on some specific aspects of the geotechnical design of reservoir spillways relating to the design of a drainage system, seepage cut-off and anchorage system.

8.4.1. Drainage system

8.4.1.1 General

The main purpose of providing a drainage system at reservoir spillways is to safely intercept, collect and transport seepage and surface run-off in order to control the hydrostatic loading acting on the structure as a means of reducing the capital cost of the project.

Where a drainage system is provided, it would also help mitigate other adverse effects, namely:

- prevent scour at the contact between the foundation soil and spillway concrete slab/blinding concrete due to seepage from the reservoir, ground water, surface run-off due to local precipitation or flows introduced into the foundation through crack or defective joints. This is often achieved in conjunction with seepage cut-offs to lengthen the seepage along the foundation and prevent the formation of concentrated flow paths
- prevent unfiltered seepage through cracks and defective joints which could cause concentrated leaks and internal erosion of the foundation soil material
- mitigate the risk of decrease of the foundation shear strength due to partial soil saturation
- mitigate the risk of decrease of the foundation shear strength due to flow induced vibrations and other dynamic actions
- prevent the formation of ice lenses and frost heave by draining water and keeping it below the freezing zone

Where a drainage system is provided it should be designed such that:

- it has adequate drainage capacity with adequate redundancy provided, which should be commensurate with the inherent uncertainties
- it is adequately filtered to prevent internal erosion of the foundation due to concentrated leaks, contact erosion, suffusion or suffosion and to prevent erosion of the filter material into the drainage system
- the filter has sufficient permeability and is not at risk of clogging
- any drainage outlets are not subject to excessive back-pressure such as generated at flow stagnation location or within hydraulic jumps
- any drainage pipework provided has got sufficient capacity, could be inspected and cleaned and is not at risk of freezing or blockage by drainage material
- it allows monitoring (including a system of manholes and measuring devices) and could be inspected, cleaned and maintained to ensure its reliable performance during the asset live of the spillway structure

Where the above requirements are not met, the drainage system may fail to perform its intended function and therefore should not be relied upon to provide pressure relief or any other mitigation. In this respect, Eurocode (1997) stipulates: "Unless the adequacy of the drainage system can be demonstrated and its maintenance ensured, the design ground-water table should be taken as the maximum possible level, which may be the ground surface." (Cl. 2.4.6.1(11)). In this respect, Cl. 9.4.2(1)P of Eurocode (1997) stipulates that a "maintenance program for the drainage system should be specified and the design shall allow access for this purpose."

The spillway drainage system generally consists of back of wall drainage and underdrainage.

The back of wall drainage typically consists of filtered pervious backfill, or other freely draining material, collection and outfall pipes. It generally provides the main control of uplift pressure as it is it easier to construct, inspect and maintain and therefore could have a much larger capacity than the structure underdrainage. This drainage is generally of the same type as the drainage used for other common earth and water retaining structures and therefore is not specifically discussed in this guide. Useful guidance on this subject could be found in the USBR Guide, Drainage for Dams and Associated Structures, 2004.

Underdrainage is often provided at reservoir spillways. It typically takes the form of a filtered drainage layer or drainage trench. Both systems often include collection and outfall pipes and are sometimes used in conjunction with seepage cut-offs. These 2 types of systems are discussed in more detail in what follows.

Pressure relief wells are less commonly used these days due to their inherent risks of clogging and potential to introduce water into the foundation. However, a brief discussion on the design requirements and risks involved is provided too.

Useful background information for design and analysis of various types of drainage is provided in the USBR Guide, Drainage for Dams and Associated Structures (2004).

8.4.1.2. Drainage layer

A drainage layer, sometimes referred to as a drainage 'blanket', is generally provided where the expected seepage flow is rather significant and therefore a large drainage capacity is required. However, a drainage layer could also be provided where relatively low seepage flows are expected but the design relies on the favourable effects of drainage and therefore requires a reliable drainage solution with high degree of redundancy (refer to section 8.4.1.1). For this reason, a drainage layer is generally an intrinsic part of the drainage system of practically all types of overtopping protection, including pre-cast concrete blocks, roller compacted concrete (RCC), conventional or mass concrete (FEMA, 2014). Such a drainage solution provides an additional line of defence and an increased margin of safety against instability, which is commensurate with the much higher reservoir safety risk posed by spillway structures built on top of an embankment.

A typical drainage layer feature placed beneath an embankment dam RCC overtopping protection system, featuring a 450mm thick drainage and filter layers and 200mm dia collector pipes, is shown in Figure 8.2 below (dimensions are shown in imperial units):

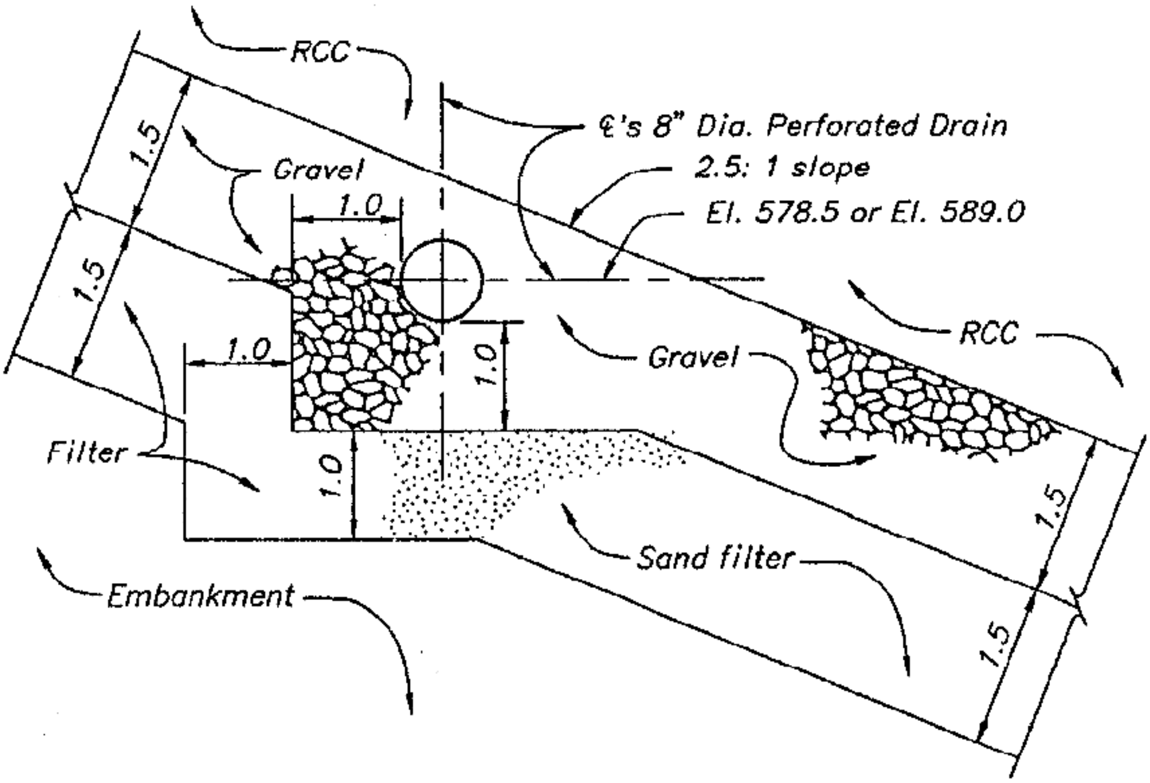

Figure 8.2: Typical drainage feature beneath an embankment dam RCC overtopping protection (FEMA, 2014)

The thickness of the drainage layer is normally dictated by flow capacity and redundancy considerations. It should also be sufficient to provide frost protection to the pipe, where additional insulation is not employed, and to mitigate the risk of ice lensing and subsequent frost heave.

Even though this arrangement is typically used for RCC overtopping protection systems, which do not include any special features, such as reinforcement or water-stops, to mitigate the risk of water being introduced to the foundation through cracks and/or joints in the RCC during overtopping flows, it could also be used where the overtopping protection takes the form of a conventional reinforced concrete spillway. In this respect, FEMA (2014) states that overtopping protection for embankment dams utilising conventional or mass concrete are also normally “constructed over a filtered drainage layer”, which along with the concrete slab, “protects the underlying embankment from high velocity flows discharging along the downstream face of the dam.”

Depending on the size of the spillway and the anticipated seepage flow, collector pipes may or may not be required. Where the drainage layer provides sufficient capacity to accommodate any potential seepage with a high degree of redundancy, and the filter has been designed to eliminate the risk of internal erosion of the foundation soil material, collector pipes may be omitted. Such an arrangement will be practically maintenance free and therefore could be relied upon for providing the benefits highlighted in section 8.4.1.1.

Where an efficient seepage cut-off has been provided at the upstream end of the spillway to mitigate the risk of seepage from the reservoir, as recommended in section 4.8.2, and all other measures recommended in this guide are taken to prevent the introduction of water from the spillway into its foundation, the drainage layer would only intercept and transport very low potential seepage flow rates. As stated previously, these could originate from the reservoir, from surface run-off due to local precipitation or from flows introduced into the foundation through cracks or defective joints.

It should be noted that, in the absence of a reliable continuous drainage layer under a spillway built on top of an earth-fill embankment, any unexpected prolonged seepage through the impermeable embankment element, or the spillway tie into it, no matter how insignificant, could over time have the potential to erode the spillway foundation at the contact with the concrete structure (Hughes, 2020). Furthermore, cracking or joint deterioration combined with inadequate spillway maintenance over the years would create unfiltered exit points for such seepage and/or could result in negative offsets, stagnation pressures and water injection into the foundation (Balmforth and others, 2020) and (Mason, 2020). This would exacerbate the erosion of the spillway foundation and could lead to a structural collapse of the spillway base slabs.

Also, the risks associated with ice lensing, frost heave and decrease of the shear strength of the foundation material due to partial soil saturation and flow-induced vibration should be duly considered.

Generally, where an earth-fill embankment contains an effective impermeable element (clay core or other), the downstream shoulder is constructed of soil material which is considered to be free-draining. Therefore, normally it would not be expected that seepage may reach the downstream face where the spillway would be constructed. However, “the lack of visible seepage on the downstream slope of an existing dam may not be sufficient to conclude that a drainage system may not be needed. It is possible that the amount of seepage that reaches the face is sufficiently small that it evaporates in the open air but could build up under” the spillway (FEMA, 2014). In addition, the construction of the spillway energy dissipation structure may, in some cases, adversely affect seepage though the embankment and raise the phreatic surface close to the downstream face.

Therefore, where a reinforced concrete spillway is constructed over an earth-fill embankment, providing a continuous filtered drainage layer underneath the spillway structure could efficiently address the inherent vulnerabilities and uncertainties by providing a reliable drainage solution. This would largely mitigate the significant risk that a potential spillway failure poses to the safety of the embankment.

The arrangement shown in Figure 8.3 or similar could be used depending on the type of joints used and the need for collector pipes (Mack, 2004):

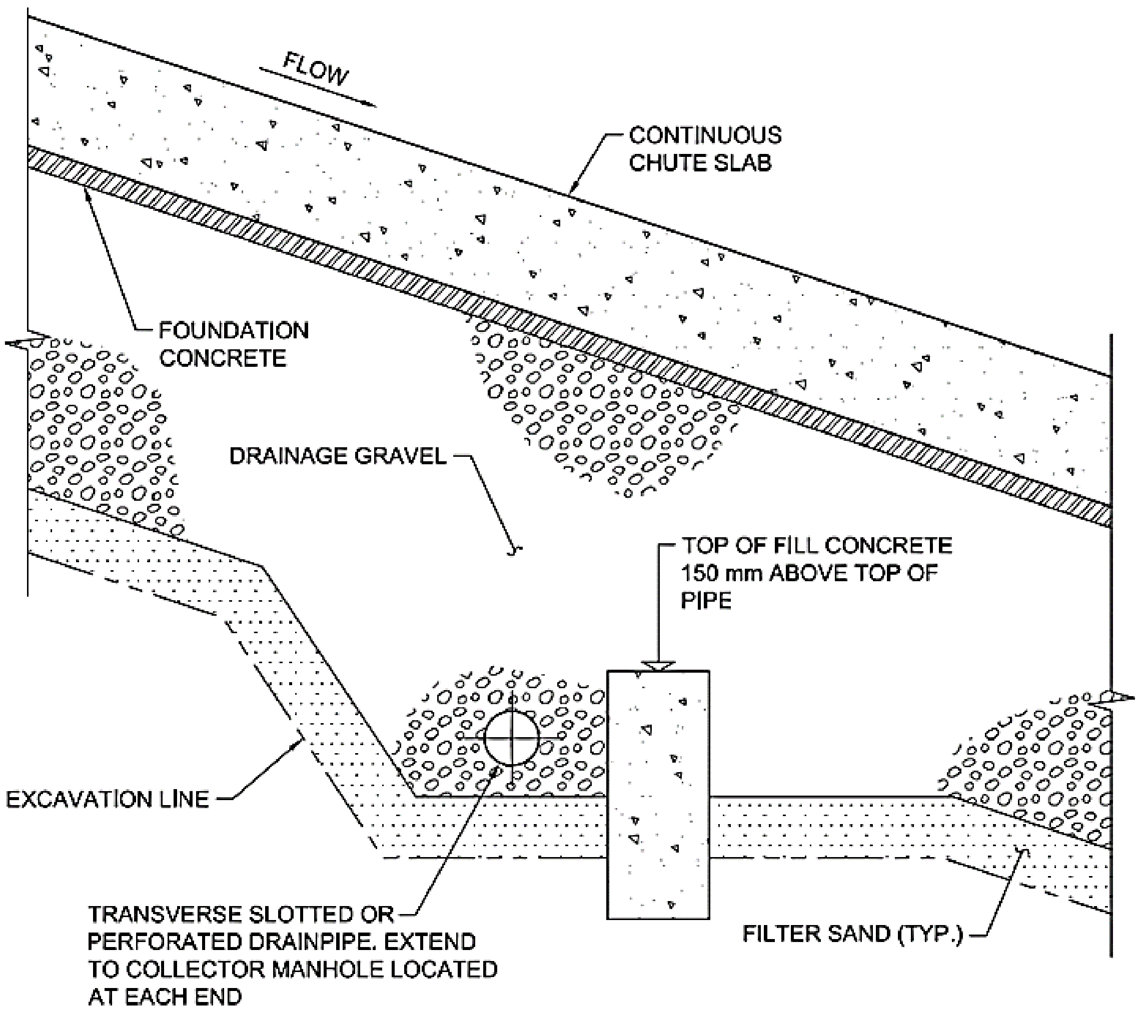

Figure 8.3: Spillway drainage layer and collector pipe arrangement (after Mack, 2014)

It should be noted that woven or nonwoven fabrics are generally not recommended for use as protective filters in locations that are both critical to safety and inaccessible for replacement (USBR, 2011). Further details for the design of drainage filters are provided in section 8.1.4.6.

The transverse collector pipe diameter should generally not be less than 200mm in diameter in order to minimise the chance of plugging and to facilitate inspection and maintenance (Mack, 2014).

Further details for the design of drainage pipework are provided in section 8.4.1.5.

The drainage provisions should be isolated from adjacent concrete placement by using geotextiles, geomembranes or other barrier materials. The selected barrier material should be suitable for the specific ground slope. “As a general rule, geotextiles and geomembranes should not be used on slopes greater than 3:1, unless they are anchored, and the overlying material can be shown to be stable against sliding.” (USBR, 2004).

8.4.1.3. Drainage trench

A drainage trench arrangement, like that featured in Figures 7.1 to 7.3, laid out in an interconnected grid pattern in plan, is typically used as an alternative to the drainage layer referred to in section 8.4.1.2 where moderate or relatively low seepage flows are anticipated to enter the spillway foundation.

With this arrangement, filtered drainage trenches are generally provided at or adjacent to the base slab joints in order to also intercept and collect any potential infiltration from the spillway channel into the foundation.

In contrast to the drainage layer solution, this drainage system restricts the movement of water along the axis of the spillway structure and therefore reduces the seepage flow rates to intercept, collect and be moved by the underdrainage system.

The main control of the seepage flow and uplift pressure acting on the spillway structure, resulting from ground water and/or surface run-off due to local precipitation, is then provided by the back of wall drainage. This is because the latter is easier to construct, inspect and maintain and therefore could provide a larger guaranteed capacity than the structure underdrainage.

The main role of the underdrainage is to intercept, collect and transport to the outfall the residual seepage flow that could reach the spillway foundation and therefore mitigate further the potential for excessive uplift pressure and other adverse effects developing as described in section 8.4.1.1.

It is therefore very important to ensure that not only the drainage pipes forming part of the back of wall drainage system but also the impervious backfill intercepting the ground water and surface run-off are not in any way connected, but are safely isolated from the structure underdrainage system. This could be achieved by separating the back of wall and underdrainage pipework and by providing a layer of impervious material “to cap the pervious backfill placed adjacent to the walls to prevent surface runoff water from infiltrating and surcharging the under-slab drainage system.” (Mack, 2014).

The trench drainage system is often combined with the use of seepage cut-offs provided at the transverse joints of the spillway structure, which further improve the control to the movement of seepage water along the axis of the spillway foundation by lengthening the seepage path and permit seepage detection.

A typical detail of a trench drainage is shown in Figure 8.4 (USBR, 2019):

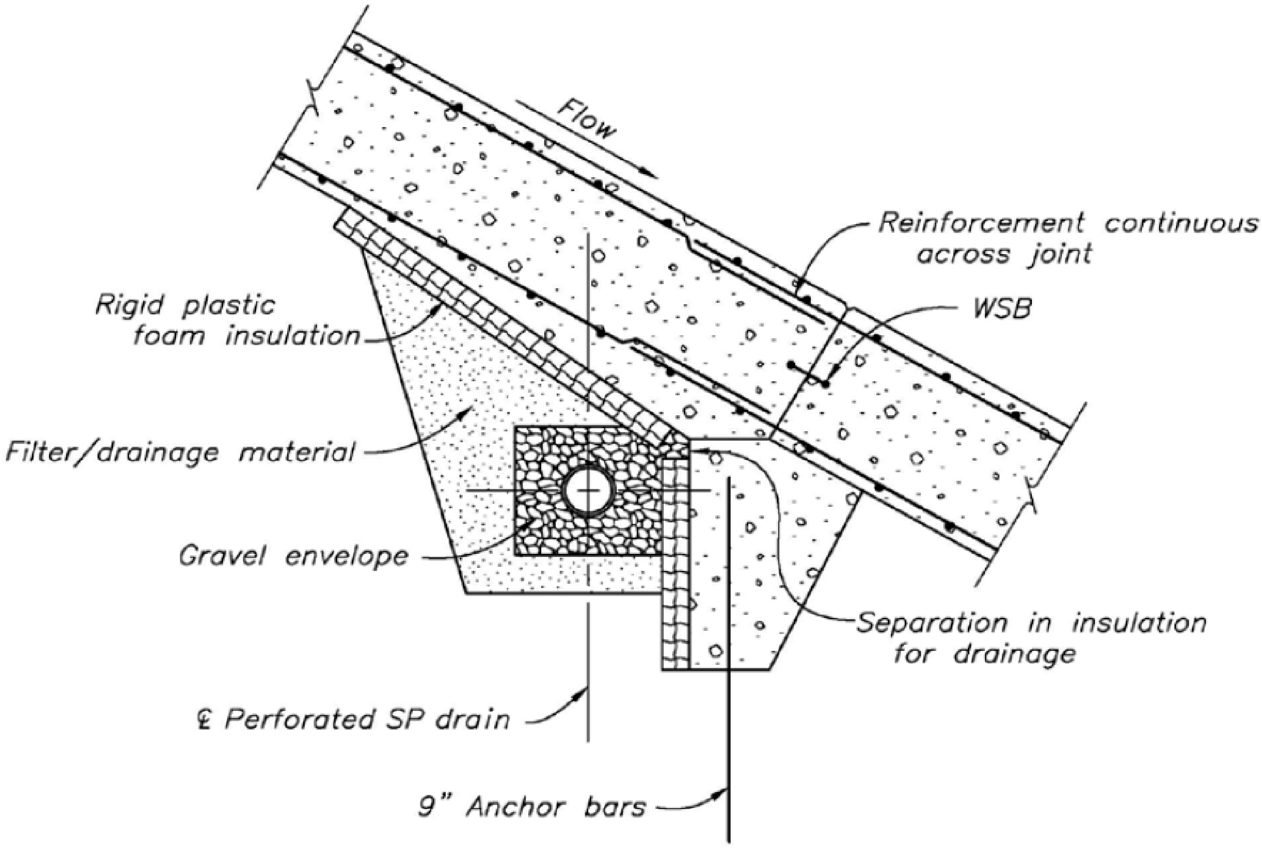

Figure 8.4: Spillway trench drainage and collector pipe arrangement (after USBR, 2019)

The above arrangement is like that shown in Figures 7.2 and 7.3 but shows distinctly the drainage gravel material around the pipe being surrounded by filter material. It is recommended that the gravel envelope surrounds the collector pipe by at least one pipe diameter on each side and underneath it. With this arrangement, consideration should be given to the maximum horizontal hydrostatic pressure acting upstream of the foundation key/cut-off wall (refer to section 8.3.4).

The restrictions on use of geotextile as an envelope or filter referred to in section 8.4.1.6 also apply for this arrangement and so does the requirement for a minimum collector pipe diameter of 200mm.

Where the collector pipe cannot be located below the frost line, suitable insulation should be provided, considering the thickness of the concrete slab and drainage/filter material.

In rock foundations, a suitable excavation method should be adopted to minimise disturbance of the foundation, as discussed in section 4.8.2.

Further details on the requirements and design of underdrain insulation are provided in section 8.4.1.7.

8.4.1.4. Pressure relief well

Pressure relief wells are generally used to relieve excess hydrostatic pressures in a confined aquifer. However, they are also sometimes installed under very large spillway inlet structures to relieve uplift pressure where they discharge into a drainage gallery. In the past, pressure relief wells were also provided to relieve uplift pressures at stilling basins. However, the latter application is not commonly used anymore due to the inherent risks of clogging and potential to introduce water under high pressure into the foundation.

Drilled hole diameters are typically 12 to 18 inches (30 to 45cm), which allows for installation of a filler pack and well casing. However, gravity flow relief wells can be installed at smaller diameters.

As with other drainage systems, filter criteria should be used in designing the 'gravel pack' around the well screen (USBR, 2004). In accordance with USACE (1992), "well-graded filters should have an annular thickness of 6 to 8 inches (15 to 20cm). Uniformly graded filters permit a lesser annular filter thickness of 4 to 6 inches (10 to 15cm)."

A typical pressure relief well arrangement is shown in Figure 8.5 (USACE, 1992):

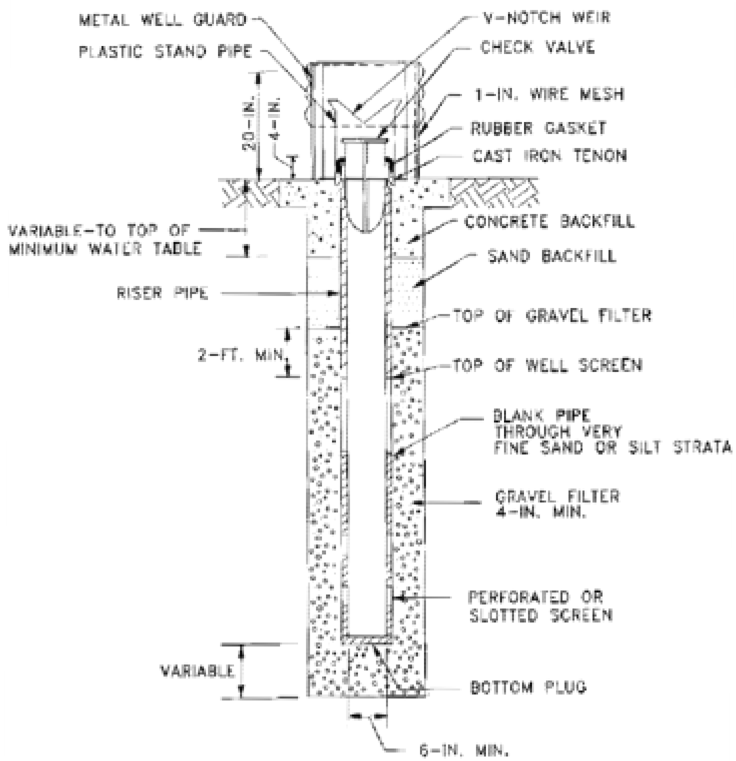

Figure 8.5: Typical pressure relief well arrangement (after USACE, 1992)

When providing pressure relieve wells It should be borne in mind that they have a "distinct disadvantage in that they require ongoing maintenance to rejuvenate their flow capacity." (USBR, 2011).

Useful guidance for the design of pressure relief wells is provided in the USBR Guide, Drainage for Dams and Associated Structures (2004) and in the USACE Engineer Manual, 1110-2-1914, Design, Construction, and Maintenance of Relief Wells (1992).

8.4.1.5. Drainage pipework design

The collector pipework of the drainage system is typically laid out in a grid pattern. Generally, “spacing of the grid in both the longitudinal (along the structure) and lateral (across the structure) directions are influenced by the amount of flow expected, anticipated efficiency over the (economic) life of the structure, etc.” (USBR, 2004).

USBR (2014) also specify, “Typically, transverse drain spacing is the same as the floor slab joint spacing. Longitudinal collector drains can be located at the outside edges when the spillway feature (such as a chute) is less than 30 feet wide. When the spillway feature is 30 feet wide or greater, intermediate longitudinal collector drains spaced between the edge longitudinal collector drains should be considered.”

All transverse collector drains should freely discharge into the longitudinal collector drains outside of the spillways structure at inspection chambers/manholes.

Drainage outlets should discharge into the outlet channel immediately downstream of the structure. They should include “a screen of some type, or rodent guards, should be placed over the exit to prevent animals from entering the pipe and plugging it.” (USBR, 2004). Alternatively, lightweight flaps could be used. No drainage outlets should discharge within the spillway channel or energy dissipation structure where high water levels and pressure fluctuations could be present, as they could be transmitted under the upstream slabs of behind the spillway walls.

The sizing of the drainage pipework should be based on a detailed seepage analysis. The analysis should consider a range of design rainfall events having the same probability of occurrence as the spillway ‘safety check flood’ and varying intensity and duration. This would allow the critical rainfall event to be identified, which maximises the design flow rates used for the sizing of the drainage pipework.

All drainage pipes should be designed for open channel flow conditions with a maximum water depth equal to 2/3 of the internal pipe diameter. This would permit free air movement and therefore prevent unstable flow conditions such as ‘slug flow’, while also allowing for potential build-up of calcium carbonate or other deposits over time.

The minimum pipe diameter used should be 200mm. This would minimise the chance of plugging and would facilitate inspection and maintenance.

All drainage pipes should be laid at a minimum gradient of 1 in 100 in order to promote self-cleansing velocity. Generally, no open drains should discharge into pipe drains, but where this may be required, the pipe drains should be laid at a gradient not flatter than 1 in 50 and “should be designed to run only half full, including the flow from the open drains.” (USBR, 1993 – Drainage manual).

The sizing of the drainage pipework should be based on the Darcy-Weisbach equation, with the friction factor calculated using the Colebrook-White equation. Tor sizing purposes, an equivalent sand roughness of 1.5mm and a kinematic viscosity of 1.67 x 10^{-6} m^2/s (at 2°C) should be used.

The drainage systems should include a system of access points (inspection chambers and manholes) to facilitate inspection, cleaning and maintenance. As a minimum, such access points should be provided adjacent to the spillway side walls, at every junction between transverse and longitudinal collector drains and at any change of direction or gradient of the longitudinal collector drains. This will provide “two access points (or cleanouts) to drain lines in order to facilitate closed circuit television (CCTV) inspection and maintenance activities.” (FEMA, 2014). Useful guidance on designing underdrain pipe systems to accommodate CCTV inspection equipment is provided by Cooper (2005).

Suitable arrangements for monitoring all the drainage flows should be provided within the inspection and outlet chambers of the drainage pipework. Any such monitoring should not have the potential to cause surcharging of the upstream drains. In order to achieve this, all pipes entering the inspection chamber or manhole should discharge from a sufficient height above the maximum water level that could be generated within the chamber as a result of the provision for flow measurement. All access points should allow sufficient air in to ensure that the drainage pipework operates under atmospheric pressure.

8.4.1.6. Filters

As discussed in section 8.4.1.1, drainage systems should be adequately filtered to prevent internal erosion of the foundation due to concentrated leaks, contact erosion, suffusion or suffosion and to prevent erosion of the filter material into the drainage system. Filters should also have sufficient permeability to pass the required drainage flows without risk of clogging.

The USBR Design Standard No. 13, Embankment dams, Chapter 5: Protective filters (2011) provides detailed guidance on the design principles and filter criteria for protective filters, including quality, flow into pipes, zone geometries, and construction considerations.

The standard also includes a detailed discussion on the shortcomings related to the use of geotextiles and states that “filters of woven or nonwoven fabrics are generally not recommended for use as protective filters” referring to studies and reports on using geotextiles for highway drainage work, indicating that geotextiles either clog or allow soil particles to pass through. This is also corroborated by USBR (2004) stating that “geotextiles are not recommended as an envelope for under-drains.”

In this respect, most practitioners in the United States and many organisations limit or forbid the use of geotextiles as filters in locations where there is no access for repair and replacement and where their performance is critical to the safety of the spillway (USBR, 2011).

Therefore, it is recommended that geotextiles or other woven or nonwoven fabrics should not be used instead of mineral filters in any underdrainage systems or in back of wall drainage systems which are critical to the safety of the spillway.

The choice of filter material should consider the susceptibility of its particles to break down during handling, transport and compaction, and natural sand materials may be preferred to crushed sand in this respect (Mack, 2014). However, it is also important that the sand material could be sufficiently compacted to resist liquefaction without this reducing its permeability.

It should be noted that the determination of filter requirements could influence design decisions relating to the location, type and size of the drainage system and therefore should be given due consideration at the early stages of the design.

8.4.1.7. Insulation

All underdrainage pipes should preferably be located below the frost line, in order to minimise the potential for their freezing. Generally, the use of insulation materials, such as rigid polystyrene foam or similar, at locations where there is no access for repair and replacement should be avoided due to the risk of deterioration of their long-term performance.

Ideally, the insulation requirements for protecting the underdrainage pipework should be met by the spillway concrete slab thickness and the thickness of the drainage layer above the drainage pipe alone.

In order to prevent ice lenses forming in the ground behind the spillway side walls, where necessary, further insulation could be provided.

The insulation requirements could be estimated using the guidance provided in the USBR guide on ‘Drainage for dams and associated structures’ (2004).

Useful guidance on the assessment and mitigation of frost action in soils could be found in the recently published ‘Frost action in soils: fundamentals and mitigation in a changing climate’, ASCE, 2020.

8.4.2. Cut-off walls

Cut-offs provided along a reinforced concrete spillway chute are typically also made of reinforced concrete, while cut-off walls at the upstream and downstream end of the spillway could be constructed using concrete, reinforced concrete, sheet piling or, less often, using other materials and techniques, including secant pile or geomembrane cut-off walls.

Sheet piling could be used for constructing cut-off walls at the upstream and downstream ends of the spillway where foundation conditions suitable for pile driving are present. The design and construction should ensure that no gaps that could potentially concentrate seepage flow and cause internal erosion are present within the sheet pile cut-off wall.

Concrete cut-offs in earth foundations should be cast where possible directly against the undisturbed walls of the excavated trench. Where this is not possible, backfilling of the trench should be carried out with suitable well compacted impermeable material.

In rock foundations, trenches for cut-offs should be created with jack hammers rather than by blasting in order to prevent cracking and destabilisation of to the rock foundation.

The depth of the cut-off wall required to control seepage and potential uplift pressure should be determined using a suitable method for seepage analysis.

Scour and/or channel degradation studies may be required to determine the required depth of the cut-off wall, as well as post-scour stability analyses of the cut-off wall (FEMA, 2014).

In order to control uplift pressures, “structure underdrains with granular backfill may be located just upstream of intermediate and downstream cut-offs.” (USBR, 2014[2]).

The USBR Design Standard No.13, Embankment dams, Chapter 16: Cut-off walls (USBR, 2014[2]) provides guidance for the design and construction of the most commonly employed types of cut-off walls currently used in embankment dam applications based on the most recent state-of-practice methodologies for cut-off wall design and construction.

8.4.3. Anchors

For the assessment of the uplift limit state of spillway structures on rock foundations, the effective weight of the spillway structure could be increased by the weight of the foundation rock to which grouted dowels, acting as passive anchors, can be tied.

The dowels typically used represent inverted L-shape reinforcement bars with diameters ranging from 25mm to 36mm, “placed in drilled foundation holes and cement grouted in place with a portion of the anchor bar equal to embedment length extending out of the foundation.” (USBR, 2014).

According to Mason (2004), “it is economically preferable to use a smaller number of large bars, because two thirds of the cost of an installed anchor bar typically comprises drilling the hole. Furthermore, a given depth of surface corrosion will have a smaller effect on large bars than on a greater number of small bars.”

The hole diameters are typically of the order of 2.5 times the reinforcement bar diameters or slightly greater to allow efficient grouting.

Generally, the holes should have a “sufficient depth to engage a mass of rock the submerged weight of which will withstand the net upward forces, assuming the mass of rock bounded by a 90-deg apex angle at the bottom with allowance for overlap from adjacent anchors.” (USACE, 2005). Typically, anchors are spaced in a 1.5 to 3.0m grid pattern (USBR, 2014).

In order to reduce costs, Mason (2004) suggests that bar depths can be staggered and states, “For example, if a 6 m depth of rock is required, only 50 percent of the bars are needed to mobilize the lower half of that depth. The other bars can be stopped at 3 m, plus an end length.”

The anchor holes should normally be inclined at a “small angle from normal to the slab so that their outlet end is downstream from their inlet end to avoid a possible increased uplift from flowing water.” (USACE, 2005).

The anchor embedment depth and spacing of anchors depend on the nature of the bedrock and the design loading. It is recommended that on-site anchor tests should be carried out to determine more accurately these parameters for the given geological conditions.

Anchors should be large enough to support the weight of the foundation to which they are attached without exceeding the yield stress of the steel (USBR, 1987). The strength design should also consider the relevant hydrodynamic actions and their potential effects. In particular, the design should consider “the resonance frequency of the system and the spectra of the fluctuating pressure in order to not overstress the steel bars.” (Barjastehmaleki, 2016). The design should also consider the potential for corrosion and deterioration of the bond strength at the anchor-grout interface due to fatigue effects resulting from prolonged exposure to flow-induced vibrations.

It is recommended that the design allows the anchorage system to withstand uplift with an ample factor of safety of 2 to 3, depending on the uncertainty and variability of the geological conditions, while considering a residual stagnation pressure equivalent to 20% of the velocity head in order to allow for any remaining uncertainties in accordance with section 8.3.3.

As an alternative to grouted rock dowels, micropiles could be used. They should be designed, constructed and tested in accordance with BS EN 14199:2015, Execution of special geotechnical works - Micropiles (2015).

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9. Alternative spillway systems

9.1. General

Lower cost alternatives to the conventional reinforced concrete spillway systems could be used where the specific site conditions and/or the low downstream risk involved may justify it.

Typically used alternative spillway systems discussed in this section include: reinforced grass, stepped-block, gabion, RCC and rock ramp systems.

Such alternative systems should only be used for auxiliary or emergency spillways where low frequency of operation and relatively low unit discharges are to be accommodated.

When designing alternative spillway systems, it should be ensured that the inherent increased risk of damage associated with their use is fully understood and efficiently managed. In this respect, some states in the USA prohibit the use of overtopping protection for embankment dams while other place regulatory restrictions on their use (FEMA, 2014).

9.2. Reinforced-grass spillways

The erosion resistance of grassed surfaces subject to high velocity flow could be increased by using geotextile or concrete grass reinforcement systems.

The design of grass reinforcement systems should be carried out in accordance with the following guidance, while considering the relevant proprietary performance test and design data as well as the prescribed installation procedures specific to a manufacturer’s product.

9.2.1. Geotextile grass reinforcement systems

The design of geotextile grass reinforcement systems should be carried out in accordance with the CIRIA Report 116, Design of reinforced grass waterways (Hewlett and others, 1987).

The CIRIA Report 116 provides guidance on good practice by setting out the procedure for the planning, design and management of steep grassed waterways, together with the engineering principles and considerations which apply.

The report discusses the application of both plain and reinforced grass. It highlights first the engineering and environmental functions which plain grass may perform in a grassed waterway in preventing erosion of the soil surface. Then it presents the enhancement of these functions that could be achieved by using geotextile and concrete grass reinforcement. It does this by recommending limiting values, in terms of the hydraulic loading parameters of maximum velocity and flow duration.

In particular, the report indicates that plain grass with ‘good cover’ could provide erosion resistance for flow velocities as high as 4.5m/s, as long as the flow duration does not exceed one hour. For longer durations, the plain grass resistance to erosion reduces progressively the limiting maximum velocity of plain grass with ‘good cover’, which drops to 2m/s for a duration of 50 hours.

The design approach is based on the results of research projects, including a major field trials programme to investigate the performance of prototype reinforced grass waterways.

It is duly acknowledged in the report that “not all the pertinent hydraulic, geotechnical and botanical processes can be described analytically”, and the risk of damage associated with grassed waterways is “higher than with conventional forms of construction.” It is therefore suggested that the guide is not a substitute for experience and the design of such structures requires good engineering judgement and should call upon expert advice where necessary.

The report indicates the limitations present at the time and states that “the applications of reinforced grass covered by the report relate to waterways subject to unidirectional flow, having good grass growth and a subsoil of relatively low permeability” and that “flow events should be no longer than about two days duration, following which grass recovery and subsoil drainage must be able to take place.”

It is also highlighted that reinforced grass systems should “only be used in situations where the frequency of occurrence of flow in the waterway is low.” This is because “grass cannot tolerate submergence or root waterlogging for more than a few days without beginning to die.”

The report provides guidance on the hydraulic, geotechnical and botanical design considerations, the detailing and specification requirements, as well as the requirements for inspection and future maintenance.

9.2.2. Concrete grass reinforcement systems

The design of concrete grass reinforcement systems should consider the provisions of the previously mentioned CIRIA Report 116 and the FEMA P-1015 Technical Manual: Overtopping protection of dams (FEMA, 2014).

The FEMA manual emphasises that the specific concrete product to be used should have been tested under the “flow conditions for which the product is expected to perform, giving due consideration to subgrade and drainage provisions.”

In this respect, the manual highlights that “most testing has been performed and prototype installations constructed with uniform channel widths and parallel walls, whether vertical or trapezoidal.” It then goes on to recommend that “if the spillway walls converge additional physical or numerical modelling should be performed to assure flow velocities and directions are not exceeding tested design limits.”

It should be noted that the CIRIA Report 116 suggests that “all concrete systems provide immediate erosion protection against the effects of rainfall or surface run-off following installation.” In this respect, the FEMA manual states that “some varieties of blocks rely on a vegetative cover grown in soil placed into open areas of the blocks or over the top of the blocks to improve performance.” It also stipulates that “if vegetation is called for as part of the design and integral to the system performance, then it should be maintained to the level called for by the specifications.”

The FEMA manual highlights the specific vulnerabilities and failure modes associated with concrete grass reinforcement systems and states that “it is the designer’s responsibility to submit the required engineering design details that will show adequate proof of no failure.”

It should be noted that, where high velocities are expected, continuously reinforced cable-tied articulating concrete block (ACB) systems are inherently safer to use than individual block systems. This is due to the cabling providing “additional lateral restraint between blocks against movement” and it can “delay the progressive failure of the armour layer” (CIRIA, 1987) if one of the blocks is dislodged. In this respect, “only cable-tied and wedge-shape blocks (refer to section 9.3 below) have been used as ACB overtopping protection on embankment dams in the US to date.” (FEMA, 2014).

9.3. Stepped-block spillways

This spillway system relies on the beneficial effect of overlapping of the concrete elements to achieve hydrodynamic stability while enhancing energy dissipation. It also provides flexibility to accommodate minor differential settlements and allows replacement of the concrete elements if required.

Generally, this system is placed “over a smooth subgrade with a geotextile and/or a bedding or drainage layer between the subgrade and the block system.” (FEMA, 2014).

The design and construction of such systems should be carried out in accordance with the specific requirements for the type of system and product used.

It is important that only “products that have been tested under the flow conditions for which they are expected to perform” should be considered with ”due consideration to subgrade and drainage provisions.” (FEMA, 2014).

Particular care should be taken to ensure that the strength of the concrete used is sufficiently high and the detailing of the concrete elements and their interface with adjacent structures is suitable to prevent damage to corners of the blocks during deferential settlements and other movements.

Useful guidance for the design of stepped-block (wedge blocks) spillways is provided in the CIRIA Special Publication 142, Design of stepped-block spillways (1997). This is based on the original concept of utilising wedge-shaped stepped elements for the protection of erodible surfaces developed in the USSR in the early 1970s.

An example of a modern wedge-block auxiliary spillway at Ogden Dam, UK is shown in Figure 4.2.

Alternative wedge-shaped block systems have been developed since then, including the Armorwedge system and the more recently studied ACUÑA block system (Caballero, F. and others, 2018).

Further guidance for the design and construction of stepped-block systems is provided in FEMA (2014).

9.4. Roller compacted concrete spillways

Roller compacted concrete (RCC) spillways combine the advantages of simplicity and ease of construction with strength and durability.

Unlike the conventional reinforced concrete spillway system, RCC spillways do not include any joints and water-stops or anchorage. However, they are normally provided with underdrainage to control seepage and relief uplift pressures. The “most common method used to control seepage for an RCC spillway or overtopping protection is a drainage layer placed beneath the RCC.” (FEMA, 2014).

RCC spillways are suitable for a wide range of velocities and depths and present very good resistance to abrasion and damage by debris impact (FEMA, 2014).

RCC spillways are often covered with soil and grass to improve their visual appearance, with some added benefits, including improved concrete curing conditions and reduced potential for freeze-thaw damage. However, this comes at the expense of increased maintenance and downstream environmental impact following spillway operation, potential erosion of the soil cover due to rainfall or underground seepage and restricted access for visual inspection.

Useful guidance for design and construction of RCC spillways is provided in the Design manual for RCC spillways and overtopping protection (PCA, 2002) and in FEMA (2014).

9.5. Other spillway systems

Useful guidance for the design of other alternative spillway systems and a description of their inherent vulnerabilities and risks is provided in the FEMA Technical manual: overtopping protection (2014).

Useful guidance for rock ramp and rock weir design is provided by USBR (2007) and USBR (2016).

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10. Instrumentation

The instrumentation typically used at reservoir spillways is to monitor the performance of the drainage system and allow measurement of structural displacements. It normally includes V-notch weirs for flow measurement, vibrating wire piezometers to monitor uplift pressures, measurement pins and joint and crack meters to measure displacements.

10.1. Drainage system performance

The drainage system performance is typically monitored via seepage measurement (V-notch) weirs.

Generally, the design should ensure that such weirs achieve free discharge and that sufficiently quiescent approach conditions are present to allow accurate flow measurement. While the design of the V-notch weir chamber does not necessarily have to comply with ISO 1438:2017, Hydrometry - Open channel flow measurement using thin-plate weirs, the guidance provided in it should be used to achieve suitable flow measurement conditions.

Where drainage pipes have been designed to discharge through walls from height, they should be extended sufficiently past any concrete or other walls and safe access should be provided to allow volumetric measurement of the seepage flow using a calibrated bucket.

Where a significant amount of seepage is anticipated and the structural stability against uplift is largely controlled by the drainage system, installing vibrating wire piezometer transducers, or similar instruments, at regular intervals along the centreline of the spillway channel should be considered. The transducers should be installed upstream of the drainage trench filter in order to record the maximum uplift pressure should the filter or drainage pipe became blocked. Since the transducers “will not be accessible for calibration or replacement, multiple transducers should be considered to provide redundancy” and “to account for the possibility of malfunction.” ‘Redundant measurements are also useful for verifying and evaluating unusual readings.” (FERC, 2020).

Any such instrumentation should be installed and operated in accordance with the manufacturer’s instructions.

It should be noted that there may be a potential for the frequency of the output signal of vibrating wire piezometers to interfere with the frequency of the flow-induced vibrations produced during overflow events.

In order to monitor the turbidity of seepage water, turbidimeters could be installed. They could provide an early indication of potential erosion of foundation soil material due to failure of the filter system.

10.2. Structural displacements

The instrumentation used for measuring structural displacements, such as spillway slab deflections, displacements across expansion and contraction joints and crack widths, typically includes measurement pins and vibrating wire or other joint and crack meters.

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11. Operation, inspection and maintenance

The design should give due consideration to the anticipated mode of operation, inspection and maintenance of the spillway structure. In particular, it should consider the spillway accessibility, means of isolation and other opertaional requirements during extreme fllod events and other emergency situations that could affect the prevention, control or mitigation of potential uncontrolled escapes of water. In general, service spillways should preferably not be constructed over embankments (refer to section 4.2.1) due to the increased risk posed by them and the resulting more rigorous requirements for their inspection and maintenance.

Stoplogs could be provided at the inlet structure in order to facilitate isolation of the spillway chute and energy dissipator for inspection and maintenance purposes where reservoir draw-down below the weir level may be problematic. Similarly, stoplogs could be provided to isolate the energy dissipator where persistently high tailwater level may be present, while taking into consideration the resulting uplift force (refer to section 8.3.3).

The design of the spillway inlet structure should take into account the potential for floating debris to be generated within the catchment and to be transported to the spillway structure. The design should also normally provide clear openings of the spillway bays, and any possible covered spillway channel structures, which are wide enough and tall enough to allow free passage of floating debris. The approach channel should achieve velocities which are sufficiently low to prevent the debris from being drawn down towards the spillway threshold, thereby reducing the discharge capacity. Recent guidance on the evaluation and management of the risk of blockage of reservoir outlet structures by floating debris is provided in the ICOLD Bulletin 176, Blockage of reservoir outlets by floating debris (ICOLD, 2021).

In accordance with Mack and others (2004) “the chute slope should normally not be steeper than 3H:1V since steeper slopes make construction difficult and more expensive, increase the potential for downhill creep to occur, and result in difficult access for inspection and maintenance.”

The design should consider the potential risks posed to the spillway operation by snow and ice. Useful guidance on this subject can be found in ICOLD (2016).

The energy dissipator should normally be designed to be freely draining and to remain dry when not in operation in order to ensure it can be regularly inspected even though it is acknowledged that this may not always be practical. The regular discharge of compensation or scour flows into it should generally be avoided if this has the potential to prevent free access and reduce the effectiveness of routine examinations. Stilling basins, which remain full, could also attract children and swimmers and therefore pose a risk to public safety.

In this respect, the design of the spillway structures should give due consideration to all health and safety hazards to the spillway operators and the public, especially to the most vulnerable groups of children, elderly and disabled people.

Such hazards could be due to the presence of:

- deep channels, creating a fall from height hazard
- steep slopes, creating a slip hazard
- enclosed conduits, creating a hazard of becoming trapped and drowning
- fast flow creating a slip and drowning hazard

To manage these hazards effectively, it should be ensured that members of the public are prevented from accessing the spillway structures as much as possible and that the spillway flow is not routed through public areas (refer to Figure 11.1):

Figure 11.1 Risk to public safety posed by spillway structures (Source: Glen Brady). a) Undrained stilling basin and its wing walls used as diving points. b) Exposed spillway structure with steep slopes within a heavily used public area. c) Spillway flow route passing through a children's playground and well used park area. d) Public pathway used a spillway crest structure

Suitable access arrangements for inspection and maintenance should be provided to all areas of the spillway structure and its drainage system in agreement with the operations team while ensuring that all health and safety and public safety risk are duly mitigated.

Where practical, permanent safe access should be provided. Any such permanent features should not impact adversely on the hydraulic performance of the spillway or create any health and safety hazards for operators or the public. Where providing permanent access may not be practical, suitable design arrangements should be made for allowing temporary access such as via rope access, scaffold platforms or other means.

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12. Decommissioning and replacement

The design should consider the anticipated conditions and constraints associated with the decommissioning of the spillway structure at the end of its asset life, and any required replacements of elements of the structure. In doing so, it should identify and mitigate, as much as is reasonably practicable, any foreseeable construction, health and safety and environmental impact and risks posed by these activities.

Factors and impacts to be considered should include but not be limited to:

- decommissioning and replacement of sheet piles or other embedded earth retaining structures and their impact on the dam and/or adjacent structures
- demolition of thick concrete/reinforced concrete elements in the vicinity of the dam
- replacement of back of wall drainage and backfill
- replacement of cover plates where provided at expansion joints
- health and safety and environmental impacts of decommissioning and replacement activities

The design should clearly when and how various elements of the spillway structure should be replaced during the period of operation and how all structures should be eventually decommissioned. This would allow the client to understand the risks and future costs associated with these activities and to manage them more efficiently.

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