UNIVERSIDAD POLITÉCNICA DE MADRID
ESCUELA TÉCNICA SUPERIOR DE INGENIEROS DE CAMINOS, CANALES Y PUERTOS
serpa Dam Safety Research

Overtopping Protection of Embankment Dams

Technical Monograph

Rafael Morán
Miguel Ángel Toledo
Fco. Javier Caballero

Department of Civil Engineering: Hydraulics, Energy and Environment

Mc Graw Hill | AULAMAGNA PROYECTO CLAVE

Overtopping Protection of Embankment Dams
Technical Monograph

Overtopping Protection of Embankment Dams

Technical Monograph

Rafael Morán, Miguel Ángel Toledo
and Fco. Javier Caballero

McGraw Hill | AULAMAGNA
PROYECTO CLAVE

Overtopping Protection of Embankment Dams
Technical Monograph

First edition: 2026

ISBN: 9791387985271
ISBN eBook: 9791387985899
Legal deposit: SE 463-2026

Printed in Spain

Contents

Section 3.
Hard Protections

Preface

This monograph presents a comprehensive synthesis of protection methodologies for embankment dams subjected to overtopping scenarios. Drawing upon empirical data, numerical modeling, and case studies from international practice, the work examines both conventional and innovative armor systems. The technical framework encompasses hydrodynamic analysis, geotechnical considerations, and performance evaluation criteria. Special attention is given to the physical mechanisms governing erosion resistance and structural stability under extreme flow conditions. This compendium is intended as a reference for practitioners and researchers engaged in the design, analysis, and long-term management of embankment dam protections.

Acknowledgements

We wish to express our sincere gratitude to all those who have supported the SERPA research group and contributed to the development of this work. We gratefully acknowledge the financial and institutional support provided by the Confederación Hidrográfica del Ebro, ACUAES, Comunidad General de Regantes del Canal de Bardenas, Dirección General de Desarrollo Rural de la Junta de Castilla y León, PREHORQUISA, CEDEX, SPANCOLD, and European Working Group on Overflow and Overtopping Erosion (EURCOLD). We thank all the researchers of the SERPA Research Group for their collaboration and continuous dedication to the various research projects that have shaped this monograph. We especially thank Javier Peraita, Ricardo Monteiro-Alves, and Miguel Ángel Fernández for their valuable and significant contributions. We extend our deepest appreciation to José Luis Sevilla, René Gómez, Teresa Cavero, Mario Carreras, Joaquín Arroyo, Francisco Sagaste, Jean-Robert Courivaud, Mark W. Morris, Eugenio Oñate, Luis Garrote, William Fiedler, Thomas Hepler, Brian Crookston, Tony Wahl, Raymond Baker, Kathleen Frizell, Warren Frizell, and Luis Balairón for their sustained support and their unwavering commitment to advancing knowledge in dam safety and overtopping protection technologies. We also honor the memory of Luis Ruano, whose sustained financial support made possible the continued advancement of the SERPA Research Group's investigations in dam safety and protection technologies. Their collective expertise, support, and shared commitment to these endeavors have been essential to bringing this comprehensive synthesis to completion. Finally, we would like to thank Adela Morán for her wise advice on the layout work of this book.

Sponsoring Company

ACIS, INNOVACIÓN MÁS INGENIERÍA, S.L.

Section 1

Overtopping and Protection Systems Overview

Aim and Scope

Recent evidence of the impact of climate change on global flood patterns (Blöschl et al., 2019; Moreno-Rodenas, Mantilla-Jones, & Valero, 2025; Muda, Kamal, Norkhairi, & Ismail, 2025; Soriano, Mediero, & Garijo, 2020) is prompting the technical community to develop responses that enhance the safety of civil infrastructure (ICOLD (International Commission on Large Dams), 2025). Additionally, infrastructure aging and growing downstream exposure mean that even moderate floods can now have catastrophic impacts. Many dams are approaching or exceeding their design life, and their spillways, outlet works, and monitoring systems often lack the capacity or redundancy required for contemporary risks. Beyond prevention and mitigation measures, there is an increasing demand for engineering solutions that strengthen the structural resilience of dams and levees, particularly in critical infrastructures where potential failures could lead to fatalities and substantial economic losses.

Latest research has addressed this objective (Hafilah Wan Ariffin et al., 2025; Monteiro-Alves et al., 2025; Moran, Toledo, Peraita, & Pellegrino, 2021; Pether, Marsh, & Cartwright, 2009; Toledo, M. Á, Morán, & Oñate, 2015; Wahl & Mortensen, 2025), however, significant gaps remain regarding both the fundamental physics governing erosion caused by dam and levee overtopping flow, and the development of cost-effective engineering solutions to protect these structures in ways that are acceptable to society.

The objective of this monograph is fundamentally practical: it seeks to introduce dam protection technologies currently in use among professionals in the technical community. Its aim is to provide readers with information about the functionality and application possibilities of these technologies, as well as references, guidelines, and manuals available for further study of each solution.

This monograph is organized into technical notes grouped into three main sections. The first section presents an overview of dam safety, focusing on the issue of overtopping, which is recognized as the most frequent cause of embankment dam failure. This section also defines the concept of overtopping protection and classifies the various systems into soft and hard protection. The second section describes soft protection measures, outlining general design criteria, including their advantages and disadvantages, and supplying important references such as manuals, publications, and catalogues for further consultation as needed. The third section mirrors this approach, focusing instead on hard protection systems.

Dam Safety Overview

By way of introduction, it is pertinent to briefly review the concept of *dam safety*, as the purpose of protection is fundamentally tied to it. According to the meaning of the word, safety is defined as the *quality by which something is safe or free from risk*. However, it is widely recognized in the field of infrastructure that zero risk does not exist; therefore, safety must be understood as a concept related to a level of risk that society is willing to accept according to its socio-economic standards.

According to the *International Commission on Large Dams* (ICOLD), *dam safety* is defined as the margin that separates the actual conditions present in a dam from those that would lead to its deterioration or failure (CNEGP, 2005). This definition highlights that evaluating the level of safety requires a thorough understanding of both the dam's current condition, typically assessed through data interpretation from monitoring systems, and the deviations in its behavior from what is considered normal operation. These deviations may signal the initiation of processes that could cause damage or deterioration to the dam. The analysis of such deviations is central to safety assessments and is typically linked to emergency threshold values as established by National dam regulations.

The fundamental purpose of dam protection systems is to enhance dam safety or, equivalently, to reduce the risk of dam failure when facing specific threats. While dams generally have a very low probability of failure, any improvement in safety must be economically proportionate to the benefit achieved, so that the investment is justifiable and comprehensible for society. As with other safety features—for example, the protection that car airbags offer at a relatively low cost—certain technological solutions can deliver significant safety gains within reasonable economic limits. In contrast, more functionally effective protection systems may exist, but their costs would be prohibitive for most users, as in the hypothetical case of equipping all cars with the protective measures found in military tanks.

Therefore, the evaluation of dam safety requires a diagnostic process, typically conducted by a specialist with field experience and, preferably, supported by tools that quantitatively assess the following aspects:

1. The dam's response to potential stresses it might experience—assuming correct operation—can be estimated using predictive models. The dam's response is recorded through visual inspections and all the measuring instruments in its monitoring system. Consequently, the predictive model should provide accurate estimates of the expected values for each data series under a given set of operating conditions.
2. The deviations between recorded and expected values, given the dam's normal behavior, are analyzed by comparing the reference values—those anticipated under theoretically correct operation as provided by the predictive model—with those actually observed. These deviations are classified according to pre-established emergency thresholds.

The safety assessment must then result in a formal decision regarding the dam's safety status, determining the scenario in accordance with dam safety regulations. For example, under Spanish regulations (MITECO, 2021), if the dam is in a normal situation, the standard operating rules apply. If the status is classified as an emergency, the Emergency Plan is enacted, with actions determined by specific risk scenarios:

- Scenario 0: Requires heightened surveillance only.
- Scenario 1: There is a risk of serious damage, but it is considered solvable with available resources.
- Scenario 2: There is a serious risk of failure, with no certainty that available means can resolve it.
- Scenario 3: There is a very high or imminent risk of failure. Measures must be taken to enable evacuation of populations in flood-prone areas and address the material damage expected downstream.

Risk Analysis

In the previous section, the concept of risk was introduced as part of the definition of dam safety. The most basic definition of risk (R) expresses it as the product of three factors: the probability P(c) that a given loading condition or event (c) will occur that could lead to failure, the conditional probability P(f|c) that failure (f) will happen if that condition arises, and the consequences or damage C(c|f) that would result in this case (CNEGP, 2005). This relationship can be written as follows:

$$R = P(c) \cdot P(f|c) \cdot C(c|f) \qquad \text{Eq. 1}$$

Based on this definition, three types of risk can be identified:

a. **Total risk**: This is the sum of the failure risk (as defined by Eq. 1) and the risk present even when the dam operates as intended (non-failure risk). Total risk includes the risk associated with dam failure as well as the risk from normal dam operation. For example, downstream flood damage can occur even without structural failure of the dam.
b. **Incremental risk**: This is the risk that results exclusively from dam failure and is given by subtracting the non-failure risk from the failure risk. Incremental risk can be significantly reduced by safety improvements such as overtopping protection systems and other mitigation measures.
c. **Potential risk**: This refers to the maximum possible damage in the event of an accident, regardless of its likelihood. This concept is frequently used for dam classification, informing the safety requirements and regulatory oversight for each structure.

As seen, risk is an inherent aspect of all construction projects and is especially critical in major infrastructure such as dams. This necessity underpins the importance of risk management, which seeks to maintain safety standards that meet societal expectations within available economic resources. Hence, Risk Analysis has become the internationally accepted process for prioritizing decisions and actions that target risk reduction in dam safety management. This methodology systematizes the analysis of safety; however, its main limitation lies in the challenge of quantifying the random variables included in the risk equation.

Some primary sources of uncertainty that affect risk quantification include:

1. The level of knowledge about the current dam condition, often hindered by inadequate monitoring and limits in analyzing collected data.
2. The unpredictability of future scenarios and loading conditions that could lead to dam failure.
3. The difficulty in objectively assigning failure probabilities for various scenarios.
4. The complexity in quantifying the consequences of failure, considering the diversity of affected assets—ranging from economic and social to environmental and even potential loss of life.

Despite these challenges, risk-based methodology is actively used in dam safety decision-making. The U.S. Bureau of Reclamation, for example, establishes decision criteria using graphical tools such as the FN diagram (Frequency-Number diagram). The FN diagram visually represents the relationship between the probability (frequency) of dam failure and the potential number of fatalities resulting from such an event (Fig. 1). The FN diagram serves as a key reference, using the probability of failure and the potential loss of life as central variables for decision criteria (Fiedler, 2016).

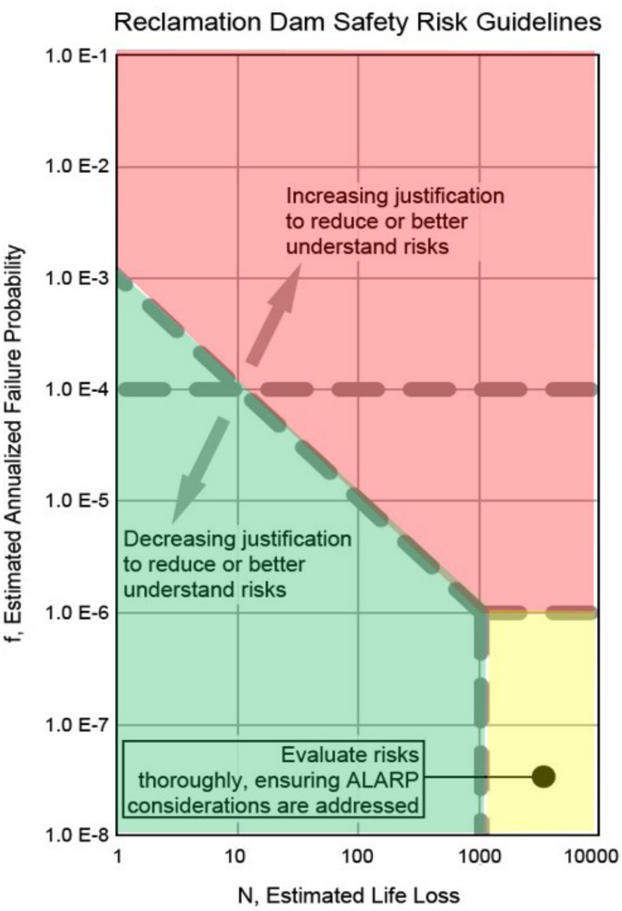

Fig. 1. Adapted from the US Bureau of Reclamation FN diagram (Fiedler, 2016).

In Fig. 1, the FN diagram employs color-coded zones to guide decisions on dam safety improvements. The red area highlights combinations of failure probability and potential loss of life where actions to enhance dam safety should be prioritized, indicating immediate or urgent need for risk reduction. If a dam's risk estimate falls within the yellow area, this signals the necessity for a more detailed, case-specific safety assessment to determine whether mitigation is justified. Conversely, dams with values in the green zone are considered to have acceptable safety, and further intervention is typically unnecessary beyond routine monitoring and maintenance.

If risk analysis, or the established safety management criteria, indicates the need for preventive actions to enhance dam safety, such measures can target either a reduction in the probability of failure or a mitigation of its potential consequences. For example, in the case of failures associated with overtopping, one preventive approach is to implement protection systems specifically designed to limit or prevent damage to the dam. When it comes to mitigation, the most common strategies involve preparing and maintaining robust emergency plans, including effective measures for alerting at-risk populations and facilitating timely evacuation if necessary.

Overtopping

Overtopping refers to the phenomenon that occurs when the water level in a reservoir rises above the crest of the dam, resulting in water flowing over the dam and down its downstream face or slope. Overtopping is the most frequent cause of failure in embankment dams, which are generally much more vulnerable to this phenomenon compared to concrete or masonry dams. According to statistics published by the International Commission on Large Dams (ICOLD) in Bulletin No. 99 (ICOLD, 1995), overtopping was the main cause of failure for 31% of embankment dams and a secondary cause in an additional 18% of cases. (code 2.3.8. in Fig. 2).

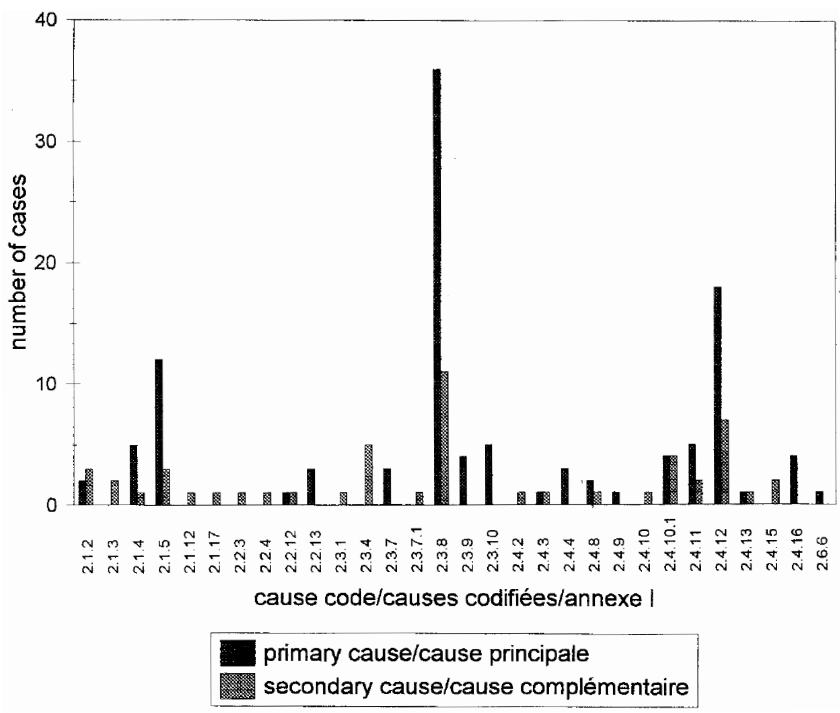

Fig. 2. Causes of embankment dam failures according to ICOLD bulletin no. 99 (ICOLD, 1995). Overtopping appears as the main cause (code 2.3.8) followed by internal erosion of the dam body (2.4.12) and the foundation (2.1.5)

Therefore, the importance of overtopping in the safety of embankment dams is clear, especially given that these dams constitute about 80% of the world's large dam inventory. Some of the most common causes leading to overtopping include:

a. Insufficient reservoir flood attenuation capacity
b. Lack of adequate protection along the dam crest
c. Insufficient spillway capacity or spillway malfunction

d. Earthquakes causing settlement or deformation of the dam body and the generation of seismic waves
e. Excessive settlement of the dam structure
f. Wave overtopping resulting from landslides into the reservoir
g. Operational failures in reservoir management, such as lack of coordination with hydrological information systems or absence of seasonal safety margins.

In most cases, dam overtopping is primarily caused by uncertainty in the estimation of design flood events or by malfunction of the dam's spillway and outlet structures. These issues typically result in insufficient discharge capacity, eventually leading to water surcharging over the dam crest.

The extent of damage resulting from overtopping in embankment dams depends mainly on:

- The unit overflow discharge, defined as the flow per linear meter of the dam crest (typically expressed in $m^3/s/m$ or m^2/s), represents the intensity of overtopping and is highly influential in determining its effects.
- The duration of the overtopping event, since appreciable damage generally requires a certain period of hydraulic loading, which depends on the specific characteristics of the dam.
- The dam height, as this factor governs the potential energy available to drive erosive and damaging processes.
- The reservoir volume, which, in the event of failure, influences the breach hydrograph and the magnitude of downstream impact.
- The zoning of the embankment. Among the various subcategories of embankment dams, the breach development pattern can vary substantially—for instance, the failure mechanism of a homogeneous earthfill dam differs markedly from that of a rockfill dam with an impervious core or a concrete-faced rockfill dam.

It is important to note that all dams can withstand a certain overtopping flow for a limited period—referred to as the incubation period. Once a specific combination of overtopping intensity and duration is exceeded, damage begins to develop within the dam structure. As soon as the integrity of the dam's impermeable element is compromised, water from the reservoir starts to escape, producing significant outflows that evolve over time according to the so-called breach hydrograph. The downstream propagation of this hydrograph can lead to catastrophic flooding.

The impact of a dam failure depends critically on the breaching process. As can be expected, a slow, gradual breach produces substantially different consequences than a sudden, rapid failure. Consequently, the breach development mechanism significantly influences both the resulting flood hydrograph and the window of time available to implement downstream mitigation measures. These measures are generally described in the Emergency Action Plans, which are mandatory for large dams. For example, in Spain, the National Regulation (Real Decreto 264/2021, de 13 de abril, por el que se aprueba el Reglamento sobre Seguridad de Presas y sus Embalses, 2021) establishes the obligation to implement such plans for dams and reservoirs with greatest risk potential. These plans set out procedures for risk assessment, alerting and evacuating downstream populations, and implementing contingency actions, thus ensuring effective response capabilities in the event of failure or emergency. These measures are generally described in emergency action plans, which are usually mandatory for large dams in accordance with current Spanish regulations.

Unfortunately, the breach formation process remains insufficiently understood, resulting in significant uncertainty when predicting flood-prone areas and estimating the arrival time of a flood wave at downstream locations. This lack of clarity complicates the accurate assessment of potential damages associated with dam breaches.

Measures to reduce the risk of overtopping-induced failure in embankment dams can be broadly classified into two categories. The first includes measures aimed at lowering the probability of dam failure—either by reducing the likelihood of an initiating event (addressing term P(c) in Eq. 1) or by reducing the probability of failure given that the event occurs (addressing term P(f|c) in Eq. 1). Relevant examples of these measures include:

a. Deployment of advanced hydrological information systems to optimize flood management.
b. Adoption of seasonal drawdown measures to increase the reservoir's flood attenuation capacity—although these may impact routine operations by reducing available storage for normal use.
c. Conventional protection measures, such as raising the dam crest or enhancing the capacity of service spillways using standard engineering approaches.
d. Unconventional protection solutions, including:
    i. Increasing discharge capacity through cost-effective measures such as emergency spillways, fuse-plug spillways, shoulder protections, labyrinth or piano key spillways, and similar upgrades.
    ii. Creation of temporary flood attenuation barriers using auxiliary elements, such as inflatable dams.

On the other hand, there exists a separate group of measures focused on reducing the consequences (term C(c|f) in Eq. 1) associated with potential dam failure caused by overtopping; these include:

a. Classification of dams according to their potential risk, along with the development of operating rules and emergency action plans. This allows prioritization of interventions for the dams at greatest risk, and the establishment of procedures and criteria to be followed in the event of overtopping.
b. Implementation of emergency action plans, which involves ensuring the necessary resources are available to execute the plan's guidelines, as well as public awareness activities so that at-risk populations are informed about the hazards and know how to respond appropriately should such an event occur. Typically, this requires training programs for civilians and periodic emergency drills.

From this point forward, the analysis will focus on risk mitigation strategies that aim to reduce the probability of dam failure (P(f|c)) in overtopping scenarios, specifically through the use of protection systems applied to the downstream slope.

Overtopping Protection Overview

In general, dam overtopping protection refers to construction technologies designed to reduce or delay damage to the body of an existing dam—either partially or entirely—when subjected to crest overflow. Protection systems typically consist of surface armoring that covers the downstream slope and toe of the dam, safeguarding the downstream shoulder from the high-velocity flows passing over the dam crest.

The erosive impact of overtopping flows on the soil materials that form the dam body—materials with significantly lower cohesion than concrete or masonry—primarily manifests as particle transport, mass sliding, and external erosion, including surface erosion and headcutting. In cases where flow permeates internally—typically in clean rockfill dams—the resulting turbulent through-flow can also induce internal erosion, transporting particles from within the dam body. These mechanisms are the principal failure modes responsible for overtopping-induced damage in embankment dams. They often act concurrently and interactively; the relative significance of each mode during a breach event remains the subject of ongoing research and investigation (Alves, Toledo, & Morán, 2016; Du et al., 2025; Jandora & Říha, 2008; Larese et al., 2015; Li et al., 2021; Monteiro-Alves et al., 2025; Song, Zhao, Zhao, Fu, & Hu, 2024; Toledo, Miguel Á, Campos, Lara, & Cobo, 2015). The aim of overtopping protection systems is to prevent these failure mechanisms from developing by enabling the controlled evacuation of flows that exceed spillway capacity over the dam body, thereby limiting potential damage to acceptable levels.

Protections installed on embankment dams function as structural reinforcements, operating only during extreme flood events with very low probabilities of occurrence—typically, when the service spillway is overwhelmed or in similar rare scenarios. As a result, these systems are rarely, if ever, activated. This rarity justifies the need to carefully balance project costs against benefits, with the main benefit being a reduction in the probability of failure associated with extraordinary events. Current practice in the technical community emphasizes such cost-benefit considerations when evaluating overtopping protection alternatives (FEMA, 2014; Hepler, Fiedler, Vermeyen, Dewey, & Wahl, 2012; Lempérière, Vigny, & Deroo, 2012; Lempérière & Vigny, 2004). The aim is to rationalize the costs of measures intended for rare use, such as auxiliary spillways and extreme event protection systems. For overtopping protections, this approach enables more projects to be implemented, thereby raising overall safety levels within available resources. Consequently, one valued feature of these systems is their cost-efficiency relative to conventional solutions like service spillway enlargement or dam crest raising. The goal is to optimize the costs of works with very infrequent use, such as auxiliary spillways or protection systems for extreme events. In the case of overtopping protections, this approach allows a greater number of projects to be undertaken, increasing overall dam safety with the resources available.

The protective measures may either cover the entire downstream face or be limited to a specific section of the dam, achieved by lowering the crest to allow water to flow through a protected downstream area. In most cases, the protection of embankment dams is typically divided into the following zones:

a. The inlet area, usually located on the crest of the dam. The main objective in this zone is to ensure continuity between the protection system and the downstream lining, while preventing negative pressures caused by the detachment of the water flow at the vertical transition.

Fig. 3. Inlet area of the roller compacted concrete (HCR) protection of the Leyden Dam (USA). Protection on the left of the photograph and vegetation upstream face on the right

b. The lining of the downstream slope's external surface. This is the area where the selected protection system is applied. As noted above, either the entire slope or a designated section for water circulation may be lined as an outlet channel. In this zone, particular emphasis must be placed on the joints between protective surfaces, the drainage system, and the proper adaptation of the chosen protection method to the behavior of the dam on which it is installed.

Fig. 4. Gabion lining of the downstream slope of the West Cornfield Dam (USA). (Photo: Tom Hepler)

c. The dam toe. It usually creates a distinct break point where energy dissipation occurs, either in the valley area or at the contact between the dam's toe and the slopes. At this stage, a wide range of design alternatives can be considered, but the common objective is to mitigate the effects of downstream flow reattachment, thereby preventing damage caused by mismatches at the dam's toe.

Fig. 5. Flip bucket for the wedge-shaped block spillway of the Barriga dam (Spain).

Exceptionally, protection systems have been implemented in service spillways located over the dam itself, particularly in ponds, small dams, or dams with low potential risk, especially in countries such as the United States, the United Kingdom, Portugal, and Spain. The main drawback in these cases is that failure of the spillway protection is critical for the dam, as it can cause severe damage potentially leading to complete failure. For this reason, such projects must be undertaken with great caution and under the supervision of engineers experienced in this type of work.

Although various protection methods have existed for decades, the principal global technical reference is the Technical Manual: Overtopping Protection for Dams, developed by the U.S. Department of Homeland Security's Federal Emergency Management Agency (FEMA). This comprehensive manual is recognized for providing consensus-based guidance on designing and implementing overtopping protection measures for dams worldwide (FEMA, 2014). This manual is the first official technical guide to include design recommendations for various types of overtopping protection, drawing on both application experience and established safety criteria. The FEMA manual specifically recommends adopting these protection measures for infrequent discharges, particularly when conventional solutions to increase spillway capacity would require prohibitively high investment, or when other justifying reasons are present. Additionally, the manual allows for the use of such protection technologies in service spillways for dams classified as having low potential risk

It should be noted that the operating limits specified in this manual for each type of protection are not determined solely by technical resistance criteria, but rather by practical experience from previous implementations that have performed satisfactorily. Consequently, the recommended operating limits represent an envelope of maximums already applied successfully in real-world cases. While engineers should treat these limits as general guidelines, there may be specific situations where exceeding them is justified. In such cases, projects must be supported by detailed technical studies to validate the proposed solution.

Based on their resistance to erosion, protections can be grouped into two main categories: *Hard protections* and *Soft protections* (Morán, R., 2015). Hard protections consist of those systems where a complete or nearly complete separation is achieved by introducing highly cohesive, erosion-resistant elements between the overflowing water and the body of the dam, enabling the management of relatively high design unit flows.

Hard protections include systems such as conventional mass concrete or roller-compacted concrete, continuous reinforced concrete slabs, asphalt concrete walls, artificial turf reinforced with mortar, cement-stabilized soil, lime-treated soil, and prefabricated wedge-shaped blocks. The remaining systems are classified as soft protections, which include natural vegetation, geomembranes, gabions, rip-rap, randomly placed or hand-placed rock, rockfill toes, reinforced rockfill, and articulated precast blocks.

For all types of protections, the primary design considerations include the following factors:

- Type of dam and characteristics of the downstream shoulder. The suitability of certain protections depends on whether the dam is homogeneous or zoned, the permeability and erodibility of the core material, expected settlements, presence of interstitial pressures, arrangement of drainage systems, and the extent of auxiliary facilities at the dam's toe.
- Classification by potential risk, which gauges the consequence of dam failure. Higher-risk dams demand more conservative design criteria.
- Hydraulics of the flow passing over the protection, since different protections result in varied flow patterns—such as smooth slab flow, stepped flow, or roughness-induced turbulence typical of rip-rap and gabions.
- Design unit flow rate, a key physical parameter alongside dam height. The maximum unit flow a system can resist is a critical measure of its erosion resistance.
- Fluid velocity, stagnation points, and shear stresses induced by flow. Many protections have defined operational limits based on safe combinations of flow depth and shear stress, restricting their use to specific ranges in practice.

Table 1 and Table 2 provide an overview of different types of protection systems, along with a summary of the recommended application ranges for embankment dam overtopping protections, as outlined in the FEMA guidelines:

Table 1. Soft protection systems for embankment dams.
Application ranges of use according to FEMA

Type of protection	Dam height (m)	Unit Flow-rate ($m^3/s/m$)	Water depth (m)	Velocity (m/s)
Natural Vegetation	8 - 15	0.6 – 2.2	0.3 – 1.2	3
Rip-rap	15	0.9 – 2.2	0.6 – 1.2	-
Reinforced Rockfill	43	14	3 – 4.3	-
Gabions	8	2.8 – 3.7	1.4	7 - 9
Articulated Concrete Blocks	12	2.8	1.3	8

Table 2. Hard protection systems for embankment dams.
Application ranges of use according to FEMA

Type of protection	Dam height (m)	Unit Flow-rate ($m^3/s/m$)	Water depth (m)	Velocity (m/s)
Synthetic Turf	12 - 15	3.0	1.5	6
Wedge-shaped Blocks	15 - 18	3.9	1.7	14
Roller Compacted Concrete	30 - 60	29 – 32	6	6 - 9
Reinforced Concrete Slab	45 - 60	22 – 26	6	24
Artificial Turf	12 - 15	3.0	1.5	6

Section 2

Soft Protections

This section introduces soft protections, defined as methods that do not fully separate the flow from the material forming the backrest. Consequently, the level of protection provided by these systems is lower than that of hard protections. However, their comparatively low cost may justify their use, as they can prevent dam failure under moderate flows or delay failure long enough to significantly reduce downstream consequences if overflow exceeds their design capacity.

This category includes vegetative protections, geomembranes, riprap, toe berms, reinforced rockfill, gabions, and articulated concrete blocks. For each type, references and links are provided to further resources detailing calculation and design considerations that go beyond the scope of this monograph.

Natural Vegetation

Plant protections are suitable for low unit discharges (0.6–2.2 m^2/s according to FEMA criteria) and short overflow durations (Fig. 6). For instance, natural grass revegetation with moderate vegetative cover can withstand flow velocities of up to 4 m/s during a one-hour overtopping event, whereas this threshold decreases to 1.5 m/s for events lasting 50 hours.

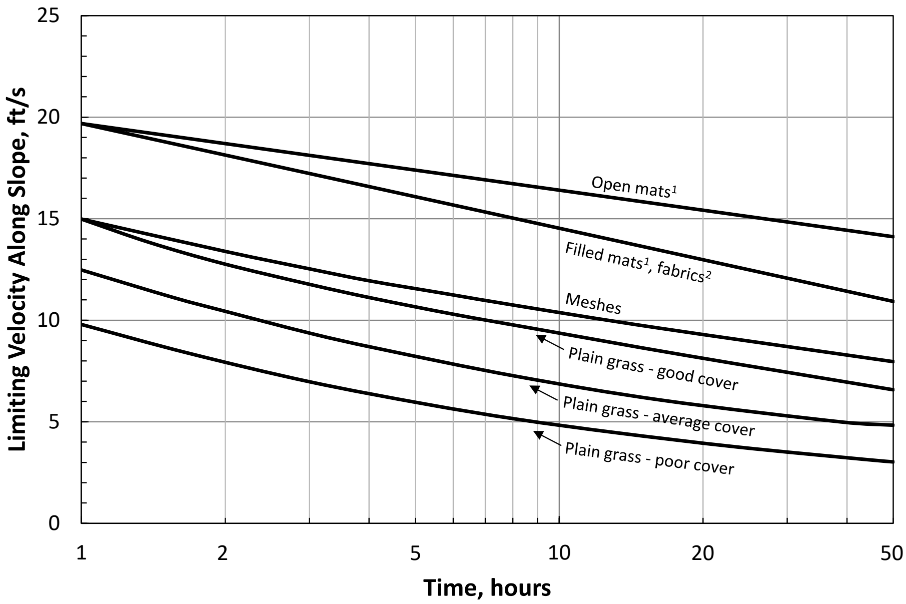

Fig. 6. Relationship between the duration of the overflow and the velocity (in feet per second) resisted by different types of plant protection (Hewlett, HWM, Boorman, & Bramley, 1987)

Therefore, this technology provides a low level of protection and is suitable primarily for small dams subject to low unit discharges, where the main objective is to delay, to some extent, the onset of dam damage. As noted in Section 1, increasing the time to failure is highly beneficial as it provides a greater window for warning and evacuating downstream populations, thereby reducing the associated risks. The protection system consists of planting grass or turf on the downstream slope of the dam (Fig. 7). The revegetation is established on a layer of topsoil, which can, in some cases, be reinforced with various types of geomembranes—significantly increasing resistance to erosion.

Fig. 7. Virginia Kendall Dam, Ohio, following three hours of overtopping with approximately 30 cm of hydraulic head above the crest. The slope of the dam was 2.5 (FEMA, 2014).

The protective effect is primarily achieved by reducing water velocity, thanks to the resistance offered by plant stems. Additionally, vegetation enhances the reinforcement of the slope's topsoil due to the stitching effect created by plant roots. This effect is further increased when turf is established over a geomembrane, a geogrid, or a layer of articulated prefabricated blocks (commonly referred to internationally as ACBs, or Articulated Concrete Blocks) with apertures that permit plant growth (Fig. 8). In cases where turf is placed over geomembranes, maximum protection is reached only after full vegetation establishment; prior to this, resistance is minimal. Artificial protection elements provide additional resistance to erosion but significantly increase overall costs.

Fig. 8. Revegetation on articulated blocks (Photo: Clayton Fawcett. Contech Engineered Solutions)

Artificially reinforced protections are typically patented by manufacturers, who provide detailed installation procedures and erosion resistance specifications in their technical manuals. For example, a recent patent covers reinforcement mats (https://nagreen.com/erosion-control-products/RollMax/VMax), which provides erosion resistance even before full vegetation cover is established. Laboratory testing has demonstrated that this system can withstand flow velocities of up to 6 m/s.

The calculation procedures for determining the duration of protection offered by natural vegetation, with or without reinforcement, are detailed in the FEMA manual (Chapter 6.2). This methodology involves estimating the velocity of water flow along the slope and determining the expected duration of protection (Fig. 6), based on the condition of the vegetative cover and any reinforcements. The calculation of flow velocity is influenced by factors such as vegetation density, average plant height, surface friction, and the developed shear stress. It is important to consider the effects of discontinuities and the plasticity index of the slope material when evaluating the erosion resistance of this protection system. The equations below (provided in Imperial units) are used to estimate flow depth and velocities:

$$q = \frac{1.486}{n} \cdot D^{5/3} \cdot S^{1/2} \qquad \text{Eq. 2}$$

$$n = e^{IC \cdot \left(0.0133 \cdot \ln(VR)^2 - 0.0954 \cdot \ln(VR) + 0.297\right) - 4.16} \qquad \text{Eq. 3}$$

where:

q, overtopping flow rate (ft^3/s)

n, Manning number (obtained according to Eq. 3)

D, water depth (ft)

S, slope of the energy line, in uniform flow is equal to the slope of the water surface and the slope of the channel bed

IC, delay index defined according to Eq. 4.

$$IC = 2.5 \cdot \left(h \cdot \sqrt{M}\right)^{1/3} \qquad \text{Eq. 4}$$

h, representative height of the grass (ft)

M, average vegetation density (number of plants/ft^2)

V, speed (ft/s)

R, hydraulic radius (ft). In wide channels one can assume R = D and therefore V·R = q

Once the water depth is known, the velocity is immediately obtained and, from it, it is possible to estimate the duration of the protection using Fig. 6.

One of the principal references for this type of protection is the CIRIA manual on plant protection (Hewlett, HWM, Boorman, & Bramley, 1987). This publication includes calculation methods and extensive design details, and it is recommended for designers seeking to size a vegetative protection system.

Research on this protection system has been performed recently, including grass revetments in sea dikes subjected to wave overtopping (Koelewijn et al., 2022; Mozer, Almström, Olsson, & Schüttrumpf, 2025; van der Vegt, 2024).

The main advantages of this technology are its low initial cost and the aesthetic appeal of the finished slope. However, the most significant disadvantages are its limited level of protection in the absence of additional reinforcement—often only delaying dam failure—as well as its high dependency on climatic conditions and plant resilience, making it unsuitable for arid or low-rainfall regions. The effectiveness of vegetative protection requires a uniform grass cover along the slope, as bare patches or uneven growth create weak spots prone to erosion. This can result in high maintenance costs (irrigation, mowing, clearing), even though installation is relatively inexpensive. Finally, vegetative protection is generally ineffective under hydraulic jumps, so supplemental measures are required at the dam's toe.

Geosynthetics

This chapter covers protection systems in which the primary reinforcement element consists of waterproof polymer-based materials that can be configured in various forms. Based on their configuration, three principal types can be identified: *geomembranes*, *geocells*, and *geocontainers*.

a. Geomembranes. These systems consist of impermeable membranes manufactured from polymer compounds such as LDPE, HDPE, PP, EPDM, or PVC. They typically act as barriers separating the dam's soil material from the water flow area and may include an added fill layer of soil-like material. Documented applications of this protection system in dams are limited. In the United States, a notable example includes its use in the spillway of Cottonwood Dam No. 5. (Fig. 9), in Colorado (Timblin, 1988). The outcome was unfavorable; the system was ultimately removed and subsequently replaced with a different design.

Fig. 9. Installation of the protective sheet at Cottonwood Dam No. 5 (Timblin, 1988).

b. Geocells. These systems are composed of polyethylene strips, typically 10 to 20 cm thick, interconnected in a honeycomb pattern that can be filled with various materials such as soil, topsoil, or concrete. This cellular structure allows geocells to be anchored to the dam body, increasing shear resistance under overlying water flow. Both protection and anchorage systems are generally patented by each manufacturer, with dedicated installation and usage specifications. Laboratory tests have shown that concrete-filled geocells can withstand flows up to 2.7 m^3/s in a 1.22 m-wide channel on a 2:1 slope, reaching velocities as high as 10.9 m/s without damage (Fig. 10). However, there are currently no known references documenting the application of geocells specifically for dam protection. When filled with topsoil, geocells support revegetation, providing a visually and ecologically favorable finish, albeit at the expense of reduced erosion resistance compared to mortar-filled alternatives.


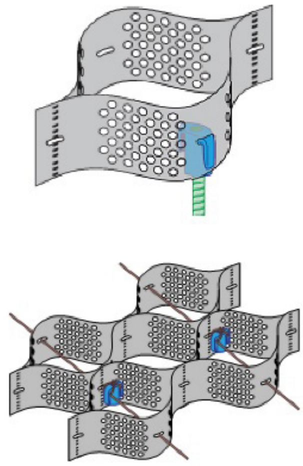

Fig. 10. Left: Preparation of the Geoweb-type
eocell test on the Colorado State University canal.
Right: diagram of the arrangement of the geocells and their ground anchoring system
(http://www.prestogeo.com/wp-content/uploads/2016/10/GWCH-Geoweb-Concrete-Infill-CSU-Research-Summary.pdf)

c. Geocontainers and geobags. These systems utilize membranes that can be filled with materials—typically cement mortar—to enhance their stability (Akter, 2016; Elias & Shirlal, 2022; Thompson, A., Oberhagemann, & She, 2020). In such applications, the geomembrane acts as a sacrificial formwork and may be further strengthened with high-tensile cables. In-situ filling enables the system to closely conform to the surface being protected (see Fig. 11). The resulting macro-roughness of the filled membrane reduces local water velocities. The overall outcome closely resembles the performance of stitched meshes of articulated concrete blocks, as further discussed in the last chapter of this section.

Fig. 11. Example of installing mortar-filled geocontainers in a chute (https://synthetex.com/hydrotex/uniform-section/)

The chief advantages of these systems are economics, coupled with their ability to adapt to potential ground settlements beneath the protected surface. However, several drawbacks exist. Primarily, these products were originally developed for other applications—most often canal or channel protection—and are generally patented by manufacturers. As a result, installation must always follow the specific guidelines provided by each manufacturer for the selected model. Due to these limitations, such systems are predominantly suitable only for protection on gentle slopes with moderate unit flow rates, unless mortar infill is utilized, which increases overall costs. In summary, most geomembranes are not appropriate for high unit flows or for the typical slopes encountered in dam design. Consequently, FEMA advises against the use of these systems on dams exceeding 7.5 meters in height.

Riprap and Rockfill

This category encompasses all protection systems formed from riprap and rockfill materials without artificial reinforcements, such as metal elements (referred to as ‘reinforced rockfill’ which is discussed in a dedicated section of this section), or the addition of mortar to partially fill the voids. The latter technique—mortar infilling—is well known and applied for reinforcement of bridge piers and channel revetments but remains unexplored for dam applications.

Within the riprap and rockfill protections, several sub typologies can be distinguished:

a. Dumped riprap. This system consists of a rockfill layer placed along the dam slope, forming an external stone armor sized to withstand erosion from a specific overtopping flow. Most design approaches are grounded in physical modeling, where the overtopping water is assumed to flow parallel to the dam slope (Fig. 12).

Fig. 12. Installation of a riprap protection in Upper Stoneville Dam, USA.
(Photo: Thomas Hepler)

b. Placed riprap. This surface protection method involves individually positioning large stones on a granular transition layer, which provides uniform support. Stones are selected according to predefined size and aspect ratio criteria and are oriented in a specific manner relative to the slope (Fig. 13). Studies of this protection type have focused on permeable riprap shoulders, making them applicable in dams where water flows as shallow, skimming overtopping and as internal ‘through-flow’ that exits at the downstream toe.

Fig. 13. Installation of a breakwater protection placed in Svartevatn Dam, Norway.
(Photo: Priska H. Hiller)

c. Rockfill toe berms. The use of rockfill toe berms as overtopping protection is still under development and their application is currently limited to dams with highly permeable downstream shoulders. The primary purpose is to stabilize the downstream shoulder and prevent mass slope failures that can occur if internal water flow (percolation) develops during overtopping events (Fig. 14). Such percolation, also referred to as through-flow, generates interstitial pressures that compromise slope stability unless additional stabilization is provided. This protection measure typically consists of a compacted rockfill berm at the downstream toe of the dam, which may be supplemented by an external riprap or additional rockfill to prevent particle dragging.

Fig. 14. Laboratory testing of a rockfill toe berm protection under through-flow conditions.

The protective material is typically defined by parameters related to the stone's size, such as its average volume, diameter, or mass. Since this material is generally uniform, using these measurements accurately characterizes most of the stones it comprises. The stone diameter can be specified in different ways: as the diameter of a sphere having an equivalent volume (Ds); as the sieve diameter that allows 50% of the sample to pass through (D_{50}); or as a nominal diameter (D_n). (Bunte & Abt, 2001) according to Eq. 5:

$$D_n = \sqrt[3]{a \cdot b \cdot c} \qquad \text{Eq. 5}$$

where:
a, b, and c are the largest, intermediate, and smallest dimensions of the stone, respectively.

In protective riprap tests with flow parallel to the slope, the *Stone Froude Number* (F_s) is employed. This dimensionless parameter, widely used by researchers in the field, enables direct comparison of results from different studies. The Stone Froude Number is defined as shown in Eq. 6:

$$F_s = \frac{q}{\sqrt{g \cdot D_n}} \qquad \text{Eq. 6}$$

Where q denotes the unit discharge flowing over the protection. When the unit discharge over the protection corresponds to the flow rate that initiates structural failure of the revetment (q_c), the resulting Froude number is referred to as the critical Stone Froude number ($F_{s,c}$), which characterizes the erosion resistance of the protection (Eq. 7).

$$F_{s,c} = \frac{q_c}{\sqrt{g \cdot D_n}} \qquad \text{Eq. 7}$$

a. Dumped riprap

This chapter includes protections consisting of riprap placed on the dam shoulder, typically over a granular support layer that acts as a transition between the sizes of the dam material and the protective layer. This type of riprap has been widely used to shield the upstream slope of dams from waves generated within the reservoir.

Riprap works by decreasing the velocity of overflow as water passes over its surface. Flow may occur through the voids between stones (percolation) or above the stones (skimming flow) when the layer is fully saturated. The velocity reduction within the percolation zone is notably greater than in the skimming flow zone. An interface exists between these zones, where the flow pattern is still not fully understood. This protection specifically mitigates surface erosion caused by water circulation parallel to the slope. It is important to note that this form of protection does not effectively resist mass sliding of a permeable shoulder subject to percolation and the development of interstitial water pressures resulting from through-flow. In such cases, additional stabilization measures, such as surface reinforcement or adjustment of the downstream slope to account for interstitial pressure effects, are required. Research indicates that, for materials of equal size, riprap composed of rounded stones provides a lower degree of protection (approximately 40% less in terms of unit flow) than that formed by angular, quarried or crushed stone.

Consequently, dumped riprap is most effective under moderate flow conditions and gentle slopes, scenarios which are uncommon in dam design except for very low-height structures. Relevant examples include the Khasab Dam in Oman and the Upper Stoneville Dam (Fig. 15) in the United States.

Fig. 15. Aerial view of the Upper Stoneville Dam (USA)

The studies conducted for sizing riprap protection have primarily focused on determining the type and size of material based on the unit discharge of the overflow and the slope downstream of the dam. The general design criteria are as follows:

- Material: Rock should preferably be angular, sourced from a quarry or crushing process, and have a uniform gradation (poorly graded). Studies have shown that well-graded materials offer a lower level of protection and are prone to segregation when placed on the slope.
- On slopes steeper than 4H:1V, which are common in dam embankments, it is generally required that most of the flow passes through the voids in the layer, significantly restricting the allowable protective unit flow.
- In the United States, various studies have been carried out to determine the appropriate size of the rockfill. Typically, the sieve size through which 50% of the stones pass (referred to as the average size, D_{50}) is adopted. FEMA, in its protection manual, recommends sizing using the Temple and Irwin chart (Fig. 16), which is based on research of Abt and Johnson, Robinson, Mishra, and Frizell. This chart, expressed in Imperial units, consolidates data from multiple studies, each applicable to different slope conditions, depending on the slope used during testing.

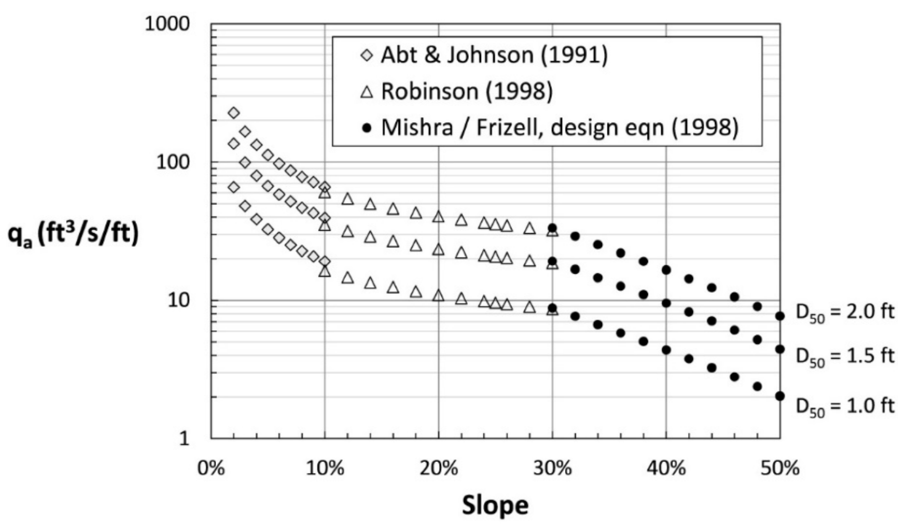

Fig. 16. Average size of riprap protections (FEMA, 2014)

- The minimum recommended thickness for the riprap layer ranges from 2 to 4 times the selected average stone size (D_{50}), depending on the design requirements and site conditions.
- The protection is generally placed over a granular transition layer that ensures adequate support. This layer must function as a filter relative to the dam shoulder material (E), in the same manner as the riprap (R) must with respect to the support layer material (B). If there is a significant difference in particle sizes between consecutive layers, several intermediate layers should be provided. The filter design criteria recommended by FEMA for the support layer—applicable when the shoulder material has a plasticity index less than 7—are given in Eq. 8 and Eq. 9.

$$\mathrm{D}_{15\mathrm{B}} < 5{\cdot}\mathrm{D}_{85\mathrm{E}} \qquad \text{Eq. 8}$$

$$\mathrm{D}_{85\mathrm{B}} > \frac{\mathrm{D}_{15\mathrm{R}}}{5} \qquad \text{Eq. 9}$$

- Likewise, the thicknesses recommended by FEMA for the support layer are those indicated in Table 3.

Table 3. Recommended thickness for the support layer based on the thickness of the riprap layer (FEMA)

Thickness of Riprap (cm)	Thickness of the support layer (cm)
30 - 60	23
60 - 90	30
More than 90	38

- In addition to the studies and design criteria developed in the United States, there are other formulas arising from research conducted in Europe, mainly in Nordic countries, that enable the determination of protective riprap sizes (Abt, Thornton, Scholl, & Bender, 2013; Haws & Erickson, 2020; Khan & Ahmad, 2011a; Knauss, 1979; NVE, 2012; Peirson, Figlus, Pells, & Cox, 2008). Some of these formulas are presented below (Eq.10 and Eq. 11).

$$\mathrm{D}_{min} = S^{0.43}{\cdot}q_{\mathrm{c}}^{0.78} \ \ (\mathrm{NVE,\ 2012}) \qquad \text{Eq. 10}$$

$$\mathrm{F}_{s,c} = 1.9 + 0.8{\cdot}\phi - 3{\cdot}\mathrm{sen}\alpha \ \ (\mathrm{Knauss,\ 1979}) \qquad \text{Eq. 11}$$

Where D_{min} is the minimum stone size to be placed; S (tan α) is the slope of the surface to be protected; $F_{s,c}$ is the critical Froude number of the stone, as defined in Eq. 7; α is the angle between the surface and the horizontal; and F is the material compactness coefficient, which can be considered as 0.625 for dumped riprap and 1.125 for placed riprap.

The main advantage of riprap protection is its low cost when suitable material is available near the dam site, as well as its adaptability to any deformations of the dam body. Riprap can also be effective against severe leaks resulting from cracking or internal erosion of the impermeable core, as was the case at Pineview Dam, where this type of protection was installed as a preventive measure (see Fig. 17).

Fig. 17. View of the Pineview Dam (USA) (Google Maps)

However, riprap protection presents several disadvantages that limit its practical application. For instance, costs rise significantly as slope gradients become steeper or as unit flows increase, due to the larger stone sizes required under these conditions. In fact, in certain cases, it may be more cost-effective to flatten the slope by adding material, thereby reducing the required stone size for protection. Additionally, this solution is subject to greater uncertainties compared to other protection systems, largely owing to the inherent heterogeneity of the material used.

b. Placed riprap

Protections using placed riprap have been primarily developed in Nordic countries, where several projects have already been implemented on existing dams based on their respective design criteria (Ravindra, Ganesh Hiriyanna Rao, Sigtryggsdottir, & Lia, 2018; Ravindra, Ganesh Hiriyanna Rao, 2020). Naturally, this type of protection is more expensive than dumped riprap, as it requires prior selection and grading of material as well as a longer installation time. The placement of the outermost layer is performed mechanically, with each stone set individually, which calls for significant experience and skill from the operators. Although this protection method already has proven use cases, it remains a novel approach and is still considered to be in the development phase.

The protection typically consists of one or more layers of smaller supporting riprap, over which the top layer of larger stones is laid, each stone being individually placed by the operator. To determine the average stone size, the Dornack formula (Eq. 12) is used, which is based on experimental results for slopes ranging from 1.5 (S=0.67) to 10 (S=0.1).

$$\mathrm{F_{s,c}} = \left(0.649\cdot \mathrm{tg}^{-0.6}\alpha + 1.082\cdot \mathrm{tg}^{0.4}\alpha\right)^{5/4}\cdot\sqrt{\left(\frac{\rho_s}{\rho}-1\right)\cdot\cos\alpha} \qquad \text{Eq. 12}$$

Where $F_{s,c}$ is the Froude number of the stone defined in Eq. 7, α is the angle between the surface of the slope being protected (tanα = S) and the horizontal plane, ρ_s denotes the specific weight of the stone, and ρ represents the specific weight of water.

In addition to stone size, the shape and placement of each stone are critical to the effectiveness of this type of protection, with the construction process itself serving as the key aspect of the method.

For optimal performance, selected stones must be placed with their longest axis (A) oriented at a specific angle (β) relative to the plane formed by the outer surface of the slope to be protected (Fig. 18). Theoretically, the most effective angle β is 90° (Lia, Vartdal, Skoglund, & Campos, 2013) however, in practice, the construction process suggests limiting this value to around 60°. With this configuration, protection tests have been conducted at nearly prototype scale on slopes of 1.5 H:V (S=0.67), and protection levels have been attained which, when expressed in terms of the variable $F_{s,c}$, exceed 5 (Hiller, 2017) —in contrast to values ranging from 0.5 to 1 typical for dumped riprap.

In conclusion, based on the research results obtained thus far, it appears evident that the level of erosion protection afforded by placed riprap is significantly higher than that provided by dumped riprap; nonetheless, further research on this technology is needed. One aspect requiring additional study concerns the ability of this protection type (and the traditional riprap) to ensure slope stability against sliding of the embankment when pore water pressures caused by percolating throughflow are present.

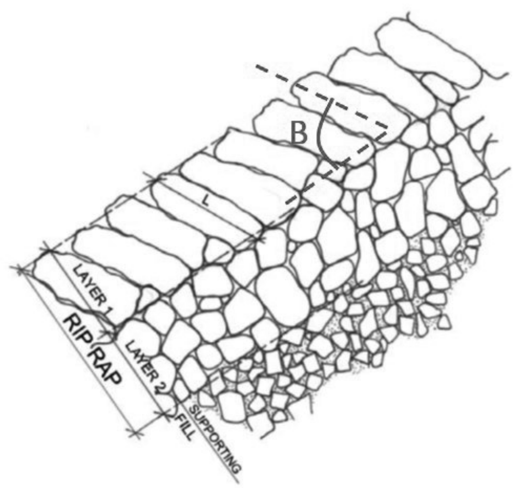

Fig. 18. Diagram showing the arrangement of placed riprap according to a typical cross-section, adapted from (Lia, Vartdal, Skoglund, & Campos, 2013)

The main advantage is the increased resistance to erosion compared to dumped riprap, estimated at roughly 30% higher—and potentially more—when stones are properly placed. It can also serve as a cost-effective solution relative to other alternatives when suitable material is available nearby, even though its cost exceeds that of conventional riprap. Like dumped riprap, placed riprap protection can also be effective against extreme seepage caused by cracking or internal erosion of the impermeable core, provided the overall dam-protection system maintains stability against sliding.

Disadvantages of this protection include the dependence on the operator's skill during stone placement, as well as the higher cost compared to dumped riprap. As previously noted, the effectiveness of this method in stabilizing the slope against sliding—particularly where the protection is permeable enough for overflow water to percolate—cannot be guaranteed unless the outer slope is modified accordingly. Therefore, the design of such solutions must justify the inclusion of additional measures to prevent this failure mechanism.

c. Rockfill toe berms

Rockfill toe berms have traditionally been used in dam engineering to increase the overall stability of the downstream shoulder at the downstream toe of the dam body. The purpose of these toes is to introduce a highly permeable material (usually dumped rockfill) with significant shear strength, thereby enhancing the stability of the downstream shoulder. In the past two decades, additional riprap toe protections have been constructed on rockfill dams to mitigate potential damage resulting from accidental percolation, which may occur if the impermeable component of the dam loses its watertightness. These measures involve placing a layer of rockfill material at the dam toe, parallel to the downstream slope, up to a specified elevation, with stone sizes sufficient to withstand the forces from certain percolation flows.

Such interventions have been applied mainly to existing dams, particularly in Sweden and Norway (see Fig. 19), following national safety standards that specify minimum unit through-flow rate that dams must be able to resist. This approach focuses on improving the toe's drainage and mechanical resistance to ensure dam safety under conditions of increased seepage flow due to internal erosion (Kiplesund, Ravindra, Rokstad, & Sigtryggsdóttir, 2021; Ravindra, G. H., Sigtryggsdottir, Asbølmo, & Lia, 2019; Ravindra, Ganesh HR, Sigtryggsdóttir, & Høydal, 2019).

Fig. 19. Construction of the rockfill toe berm at Suorva Dam in Sweden (Nilsson & Rönnqvist, 2004)

In an overtopping scenario involving a dam with a high-permeability downstream shoulder, water infiltrates the shoulder (through-flow), resulting in partial saturation of the lower region and discharge at the downstream toe. As the flow rate increases, the shoulder may become fully saturated, leading to greater surface flow outside the slope in a direction largely parallel to it. Toe protections in the form of riprap are designed to prevent damage to the downstream shoulder during the initial stages of overtopping.

To address this, a design methodology—based on experimental laboratory research—has been developed for sizing riprap toes that will stabilize the shoulder for a specified percolation flow, considering three primary failure mechanisms: particle dragging, internal erosion, and sliding. If the shoulder becomes fully saturated and water flows on the surface from the crest downward, toe protection is insufficient and a broader area must be protected, extending across the entire downstream slope.

The principal application limits for rockfill toe designs are summarized below:

- The design flow rate threshold is reached at complete saturation of the shoulder.
- The dam shoulder to be protected must consist of high-quality, non-cohesive, quasi-isotropic stone—not over-stratified during compaction.
- The foundation must exhibit sufficient scour resistance throughout the percolation duration. Otherwise, additional protective measures will be needed for the foundation or adjacent downstream areas.

Protection against sliding with toe support consists of a highly permeable riprap material placed at the dam toe, as illustrated in Fig. 20. The toe is characterized by its height (H), berm width (B), and slope (N_b) and is formed with riprap material of known specific gravity, internal friction angle, and permeability.

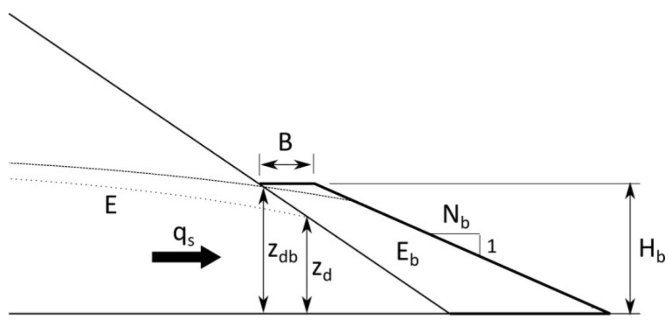

Fig. 20. Diagram of protection with rockfill toe berm at the dam toe

The material properties of the rockfill toe berm must be suitable for preventing internal erosion. As a result, the toe material must function as an effective filter for the shoulder material; otherwise, it will be necessary to install transition layers between the two. To avoid surface erosion caused by scouring of stones located along the outer edge of the protection zone, the outermost layer must be sized according to the design unit flow and the slope determined for mass sliding resistance. To achieve this, refer to the formulations presented in this chapter concerning dumped riprap and placed riprap.

The essential data required to define the geometry of the rockfill to berm are:

- Design unit through-flow: the percolation unit flow (q_s) that the protection must withstand without damaging the dam.
- Slope of the protected dam section (N).
- Properties of the available dam and riprap shoulder materials for protection:

Coefficients (a,b) of the nonlinear seepage equation (Eq. 13) for the dam (E) and protective material (E_b) relating hydraulic gradient (i) to the average seepage velocity (v)

$$\mathrm{i} = \mathrm{a} \cdot \mathrm{v} + \mathrm{b} \cdot \mathrm{v}^2 \qquad \text{Eq. 13}$$

A proposed design procedure (Morán, Rafael, Toledo, Larese, & Monteiro-Alves, 2019), validated under laboratory conditions, involves first calculating the seepage for the design flow through the dam shoulder without the toe. This calculation provides the height of the saturation line at its intersection with the shoulder slope (z_d). The berm width of the toe (B) should be set at the minimum value compatible with the compaction process used in construction. The toe slope (N_b) must ensure stability under fully saturated conditions; for this purpose, the Toledo's formula (Eq. 14) can be applied, which defines the slope of the protection as the inverse of the tangent of angle α (Toledo, M. A., 1997):

$$F = \frac{1}{\gamma_{Eb,sat}} \cdot \left(\gamma_{Eb,sat} - \frac{\beta \cdot \gamma_w}{\cos^2\alpha} \right) \cdot \frac{\tan\varphi_{Eb}}{\tan\alpha} \qquad \text{Eq. 14}$$

Where:
F: required safety factor against sliding

$$\beta = -0.32 \cdot N_b + 1.52 \cdot N_b - 0.77; \ (1.5 < Nb < 2) \qquad \text{Eq. 15}$$

β = 1 with $N_b > 2$
N_b: slope (H:V) of the toe berm
$\gamma_{Eb,sat}$: saturated specific gravity of the rockfill at the toe berm
γ_w: specific gravity of water
φ_{Eb}: friction angle of the rockfill at the toe berm
α: angle of the downstream slope of the rockfill toe berm with the horizontal (tg α = $1/N_b$)

Once the slope has been determined, the height of the toe (H_b) must be estimated using a coefficient A that exceeds the previously defined variable z_d. An initial estimate for coefficient A can be obtained by solving Eq. 16:

$$\frac{K_{dE} \cdot K_{dEb} \cdot (B + Z_d \cdot N_b \cdot A)}{Z_{dEb} \cdot Z_d \cdot N \cdot A + K_{dE} \cdot (B + Z_d \cdot (N_b - N) \cdot A)} \cdot A^2 = K_{dE} \cdot \frac{B + z_d \cdot N_b \cdot A}{N \cdot z_d} \qquad \text{Eq. 16}$$

All values in Eq. 16 are known except for A. The variable K_{dE} is the inverse of the permeability at maximum gradient for the dam material (E), as defined in Eq. 17. K_{dEb} is similarly defined but applies to the material of the protective toe (E_b). The variables a and b are the nonlinear permeability coefficients defined in Eq. 13, with subscripts E or Eb indicating the material to which they pertain.

$$K_{dE} = \frac{1/N}{\frac{-a_E + \sqrt{a_E^2 + 4 \cdot \frac{b_E}{N}}}{2 \cdot b_E}} = \frac{2 \cdot b_E}{N \cdot \left(-a_E + \sqrt{a_E^2 + 4 \cdot \frac{b_E}{N}} \right)} \qquad \text{Eq. 17}$$

Once coefficient A has been determined, the height of the toe (Hb) can be calculated, thereby geometrically defining the typical toe section. Finally, a verification calculation must be performed, consisting of a numerical simulation of seepage under the design unit flow through the dam-toe system.

One advantage of this type of protection is its low potential cost, since material for the toe construction is often available near the dam if the original design included rockfill. The use of dumped rockfill further enhances construction simplicity. Functionally, this solution provides effective protection not only against overtopping but also against core cracking, internal erosion of the wall, or foundation failures—rupture mechanisms that account for a substantial proportion of failures in embankment dams. Additionally, as a drainage solution, it prevents the development of pore pressure within the dam compared to impermeable protections.

The main drawbacks of toe-type protections for mass sliding resistance include limited experience with real case applications and large-scale testing, which currently restricts evidence for their performance at prototype scale. From the dam impact perspective, this solution may interfere with pre-existing structures such as bottom outlets or spillways, potentially affecting final costs. Moreover, toe protections are recommended for moderate design flows, with decreasing effectiveness for higher flows. Another challenge arises if anisotropy in the permeability of the shoulder rockfill is significant, as this may compromise overall structural effectiveness.

Reinforced Rockfill

As discussed in the section on riprap and rockfill protections, there are established equations for sizing the stone required to prevent erosion. When combined with robust design against mass sliding, these solutions can withstand the various failure mechanisms associated with overtopping events. There are also alternative strategies that use stone or riprap as the base protective material, aiming to mitigate the inherent lack of cohesion in such materials. Examples include reinforced riprap protections and gabion protections—the latter will be addressed in the following section of this technical monograph.

Beyond these, a broader set of techniques—collectively known as “reinforced fill”—can be used to enhance earth fill materials (including cohesive soils) for dam construction. One example from this category is the Armored Earth technique (Reinforced Earth), which is more commonly used in other civil engineering sectors. While this type of reinforced structure is not addressed in detail here, this chapter presents two reinforcement techniques applicable to rockfill protections. The first involves grouting mortar at the external surface of the rockfill, while the second consists of steel armouring reinforcement, as described in ICOLD Bulletin No. 89, which includes a dedicated chapter on reinforced rockfill dams. (ICOLD, 1993).

a. Mortar Grouting Reinforcement

The technique of reinforcement through mortar addition has been successfully implemented in other areas of civil engineering—such as riverbank protection, bridge piers, and highway embankments—under the designation “*Partially Grouted Mortar,*” as developed by Lagasse et al. (Lagasse, Clopper, & Arneson, 2008). Based on this methodology, various studies have explored approaches to enhance the erosion resistance of rockfill that may be exposed to external erosive forces (Abboud & Coe, 2019; Escarameia, 1998; Khan & Ahmad, 2011b; Marr et al., 2014; Yandem, 2015).

According to Lagasse et al. the mortar fill must meet the following characteristics:

- It must preserve at least between 50% and 65% of the void volume of the riprap to maintain its draining capacity.
- It must ensure the bonding of each stone with several adjacent stones.
- The D_{50} of the riprap must be between 20 and 40 cm.
- The thickness of the protection must be at least twice the D_{50}.
- The uniformity coefficient must range between 1.5 and 2.5.

The Federal Waterway Engineering and Research Institute of Germany (Federal Waterway Engineering and Research Institute (BAW), 1990) provides the following reference mix for one cubic meter of mortar:

- Ordinary Portland cement: 442 to 454 kg

- Sand, dry: 704 to 716 kg
- Very fine crushed gravel, dry (6.35 mm): 704 to 716 kg
- Water: 251 to 269 kg
- Entrained air: 5 to 7%

As previously noted, this methodology has been widely and successfully employed in civil engineering; however, practical experience regarding its application for the protection of rockfill dams and levees remains limited. Despite its cost-effectiveness for safeguarding existing dams against overtopping or increasing the resilience of rockfill protections discussed in Chapter 3, there is still a lack of comprehensive knowledge derived from real-world implementations.

b. Steel Armoring Reinforcement

Reinforced rockfill with steel armoring involves strengthening the outer surface of the downstream slope with a mesh of steel bars anchored into the dam body. This method is typically used to stabilize dam slopes that have downstream rockfill shoulders. The primary objective is to prevent movement of the stones forming the outermost layer of the slope when the shoulder is exposed to unusually high overflow or through-flow discharges—either from overtopping or due to a loss of watertightness in the dam's impermeable component. The protective system must be capable of withstanding filtration drag forces, impacts from floating debris, and potential surface sliding; therefore, reinforcements are generally constructed using large-diameter steel bars.

Design criteria for this approach are empirical and based on practical experience from actual projects. Nearly all documented cases of this protection technique are found in Australia and South Africa. The most notable example is the Pit No. 7 Afterbay Dam (Fig. 21), which is 11 meters high, has a slope ratio of 2.25, and has frequently operated under unit flows near 1 m^2/s, with occasional flows reaching up to 14.2 m^2/s.

Fig. 21. View of the downstream slope of the Pit No. 7 Afterbay dam (FEMA, 2014)

Additional examples of overtopping events on reinforced rockfill dams include the *Gogoon* (FEMA, 2014) and the dam of *Des Arc Bayou Site No. 3,* which stands 40 meters high with a downstream slope of 1.80; however, operational data on overtopping are not available for the latter. ICOLD Bulletin No. 89 provides a list of 50 Australian dams incorporating reinforced protections, with 5 reported failure cases as highlighted in the FEMA manual.

The components of these protections are as follows:

- <u>Outer mesh</u>. This consists of a framework of anchoring bars combined with a surface mesh (see Fig. 22). Typically, the support bars are arranged so that horizontal bars are placed beneath those parallel to the line of maximum slope, which are positioned above. These anchoring bars generally have diameters ranging from 20 to 36 mm, averaging around 24 mm. The center-to-center spacing between the bars on the slope surface is usually around 1.4 m horizontally and 1.5 m vertically, depending on the slope and the mesh size, which itself is determined by the dimensions of the underlying rockfill. Directly above the clamping bars is the surface mesh, composed of wires or bars with diameters between 3 and 20 mm, with an average diameter of 5 mm. Openings in the surface mesh may vary from 5 × 5 cm to 1.2 × 1.2 m, with an average of 1 × 1 m. The protection should extend higher than the outlet point where the seepage water exits to the outside.

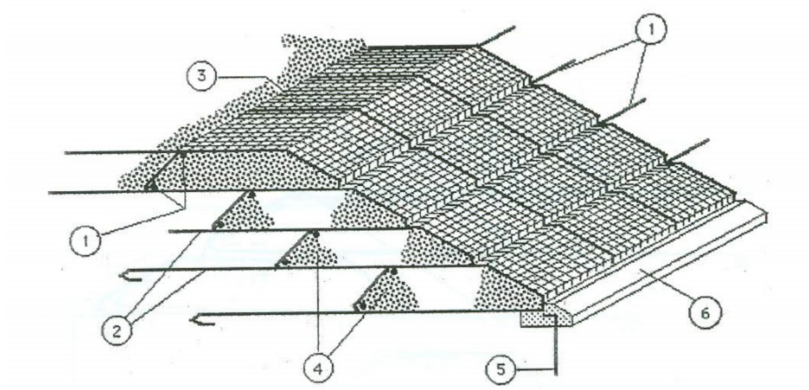

Fig. 22. Fika Patso Dam Reinforced Breakwater Protection System (ICOLD, 1993)

- Stitching and anchoring system to the dam. The anchors secure the steel mesh by maintaining close contact with the slope's outer surface and preventing displacement due to the outward movement of stones prone to being dragged or sliding downstream. This system typically consists of steel bars inserted into the dam body, with diameters ranging from 20 to 36 mm (average 24 mm), and spaced transversely at approximately 1 meter intervals using similarly sized bars. For effective performance, the anchoring bars must be embedded deeply enough—typically around 12 meters on average, with a minimum of 6 meters and a maximum of 40 meters—to extend beyond potential slip surfaces that could develop if the anchor were absent. The vertical spacing of these anchors is generally between 0.75 and 3 meters (average 1 meter), while horizontal spacing ranges from 0.25 to 2.40 meters (average 1.35 meters). In some designs, the crest protection is completed with a concrete slab acting as both a spillway and an anchoring structure for the upper portion of the outer mesh. (Fig. 23).

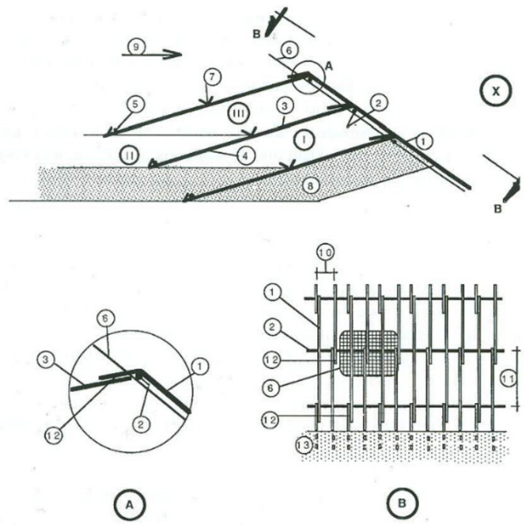

Fig. 23. Proposed construction details for reinforced rockfill (ICOLD, 1993)

- Foundation Anchoring. At the lowest part of the slope, near the dam's toe, the largest stones and the most robust reinforcement must be used, as this area is subject to the highest hydraulic gradients. It is also essential to ensure a secure connection between the slope protection at the dam's toe and the foundation, since this is where the most significant hydraulic gradients are encountered. This is typically addressed by installing a concrete block at the toe, which is anchored into the foundation. ICOLD Bulletin 89 provides several example solutions from previous projects that can serve as valuable references for designers.

Among the advantages of reinforced rockfill are the numerous documented cases of successful implementation in real dams that have experienced significant overtopping events. Furthermore, ICOLD has published a bulletin providing design criteria, which serves as a valuable reference for project planning and installation. This protection method is relatively inexpensive for new dam construction.

On the downside, some failures have been recorded due to insufficient stability against mass sliding, and the protection system can suffer from degradation related to reinforcement corrosion. Another disadvantage is the difficulty of applying this method to existing dams, as installation is typically integrated into the original construction process and is less effective if retrofitted. From an aesthetic perspective, reinforced rockfill is not particularly appealing unless additional surface treatments are applied to conceal the underlying reinforcements.

Gabions

As described in the previous chapter, gabion protection—like reinforced rockfill—is intended to artificially increase the low erosion resistance characteristic of rockfill due to its lack of cohesion between particles. Unlike reinforced rockfill, which achieves this through construction techniques executed in situ on the dam shoulder material, gabion protections are modular, relying on prefabricated elements.

Gabions are structures consisting of parallelepiped baskets made of steel wire mesh, typically filled with stones of relatively uniform size. The mesh serves to confine the stones and prevent their displacement. In some designs, the baskets are internally compartmentalized with diaphragms (see Fig. 24) to reinforce the structure and further limit movement of the stones within, especially when water passes through. Using these internal reinforcements is recommended when the design unit flow exceeds 1.4 m^2/s.

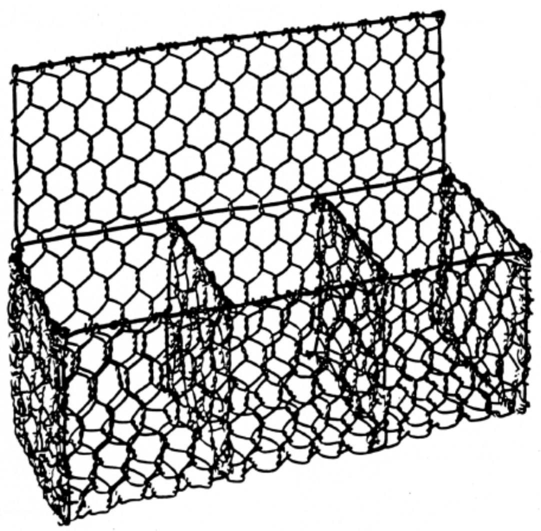

Fig. 24. Outline of mesh forming a gabion basket with internal diaphragms (Berney et al., 2001)

Additional measures to stabilize the gabion protection system may include joining the faces of adjacent gabions and reinforcing the basket edges with metal mesh, particularly in areas most exposed to hydraulic forces. In some cases, concrete surface coatings are applied to improve resistance to impact from debris; however, it is important to consider the potential effects of such coatings on the overall drainage capacity of the system.

There are documented applications of gabion protection systems in small dams (see Fig. 25) and in sediment retention structures within torrential channels, such as those implemented at the Zamoly Dam (Hungary), Pacific Pines Dam (Australia), and West Cornfield Dam (USA).

Fig. 25. Gabion protections at Pacific Pines (left) and West Cornfield (right) Dams (FEMA, 2014)

The protective capacity of gabion systems is typically verified through testing conducted by manufacturers for their various models; therefore, engineers should rely on manufacturer manuals and recommendations during design and specification. In addition to these guidelines, both FEMA and FAO provide supplementary recommendations for the use and design of gabion structures in hydraulic and dam engineering applications (Berney et al., 2001). Specifically, FEMA references experimental trials conducted by the *Bureau of Reclamation*, which confirm that gabion systems perform effectively in laboratory settings at unit flows up to 3.7 m^2/s and velocities of 9 m/s for slopes of 4H:1V and 6H:1V—slopes considerably flatter than those typically found in embankment dams. For steeper slopes of 3H:1V and 2H:1V, which are more representative of embankment dams, tests have achieved maximum velocities of 5.8 m/s and unit flows of 2.5 m^2/s. In these cases, only minor damage was observed, such as cage deformations caused by downstream displacement of stones and some localized damage at the dam crest. Additionally, it was demonstrated that the size and shape of the stones used in gabions do not significantly impact hydraulic behavior, whether under skimming or nappe flows.

Crest damage can be reduced by anchoring the gabions at the top of the dam—either by extending the protection upstream or by constructing an anchor trench. If even these minor breakdowns are unacceptable, FEMA recommends restricting the unit flow to 0.9 m^2/s.

For optimal placement and internal material stability, it is recommended that the gabion protection system be installed on a granular filter layer or on a geotextile meeting proper filter criteria with respect to the downstream shoulder material.

With respect to energy dissipation, there are specific criteria for basin design in gabion spillways (Peyras, Royet, & Degoutte, 1991) which indicate that staggered configuration of gabion protection are more effective. In some cases, the dissipation structure at the dam toe can itself be built from gabions—as in the West Cornfield Dam (Fig. 26) resulting in significant cost savings compared to cast-in-place energy dissipation devices.

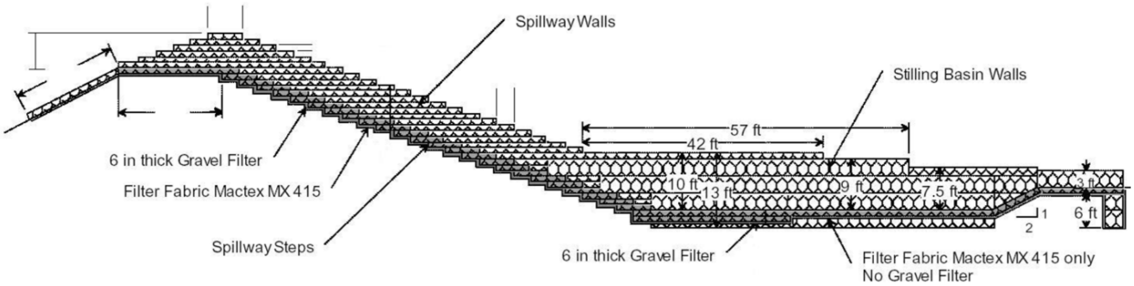

Fig. 26. Cross section of West Cornfield Dam Protection (FEMA, 2014)

The dimensions and length of a potential stilling basin located at the dam toe can be estimated using the FEMA methodology, which incorporates the studies of Peyras. This approach utilizes the dimensionless parameter D to perform the calculations (Eq. 18):

$$D = \frac{q^2}{g \cdot H^3} \quad \text{Eq. 18}$$

Here, *q* represents the unit discharge, *H* is the elevation difference between the basin and the reservoir water surface, and *g* denotes the acceleration due to gravity. Using this dimensionless variable, the sizing chart can be consulted (Fig. 27) to estimate the initial water depth (d_r) at the basin inlet.

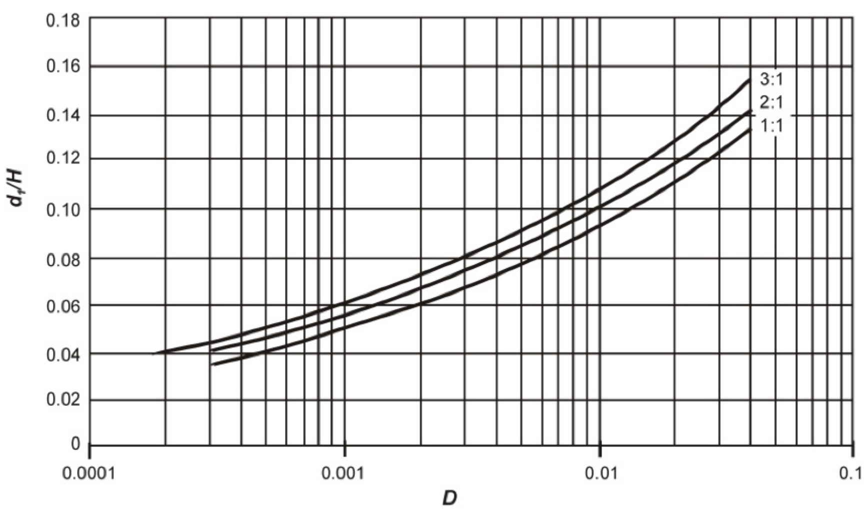

Fig. 27. Non-dimensional water depth at the inlet of a stepped gabion spillway (FEMA, 2014)

Once the water depth at the inlet has been determined, the corresponding conjugate depth can be calculated. For a slope of 1, the recommended basin length is approximately six times the conjugated water depth. According to existing studies, gentler slopes require a longer basin; thus, for a slope of 3, it is advisable to increase the basin length by 15% compared to that specified for a slope of 1.

Recent publications on this protection system (Kobayashi, Kobayashi, Takahashi, & Morii, 2024; Lee, D. et al., 2024; Lee, Y. H. et al., 2024) are available for engineers seeking information on its practical applications.

The main advantages include low cost compared to alternatives such as riprap, as gabions provide a higher degree of protection for the same volume of material while allowing the use of smaller stone sizes. The modular design of these elements facilitates onsite installation and helps reduce construction time.

However, several disadvantages should be considered. Gabions can be susceptible to corrosion, vandalism, and impacts from floating debris. Clogging by fine sediments may adversely affect drainage performance. Additionally, aesthetic concerns may limit their use in certain environments or projects.

Articulated Concrete Blocks (ACB)

This term encompasses all modular systems formed by prefabricated concrete blocks that maintain interconnection between elements, often enabling rapid installation on dam slopes. These are typically described as ACB systems, referencing the acronym for 'Articulated Concrete Blocks'. ACBs emerged as an industrial alternative to conventional erosion protection systems and have become widely used for the protection of channels, canal linings, and drainage structures (NRCS, 2007). However, only certain ACB typologies commonly used in channel protection are suitable for dam overtopping applications, due to the distinct hydraulic conditions involved. In dams, flow characteristics—including shear forces, velocities, and flow orientation—differ significantly from those encountered in channelized systems. The earliest documented installations of ACBs for dam protection were at Blue Ridge Parkway, where three small dams ranging from 8.5 to 12 meters in height were armored to withstand unit discharges between 0.65 and 2.8 m^2/s.

Typically, the blocks used in ACB systems are fabricated from mass concrete or fiber-reinforced concrete, with steel reinforcement being uncommon. These protection systems share the ability to conform to moderate settlement and deformation of the underlying substrate. The diversity of models and patents is significant, as manufacturers offer a range of products tailored to specific applications. Certain ACB designs incorporate sufficient spacing between blocks to enable partial vegetation growth, thereby enhancing the aesthetic integration of the protection. Depending on the method of interconnection among prefabricated units, ACB systems can be further classified into distinct categories:

1. Interlocking. Systems in which each concrete block features recesses and protrusions that enable lateral interlocking between adjacent units (Fig. 28). Its installation requires manual placement of each unit, and its level of protection is lower than that of other ACB systems due to weak points that form at the joint protrusions. It is not commonly used for dam protection.

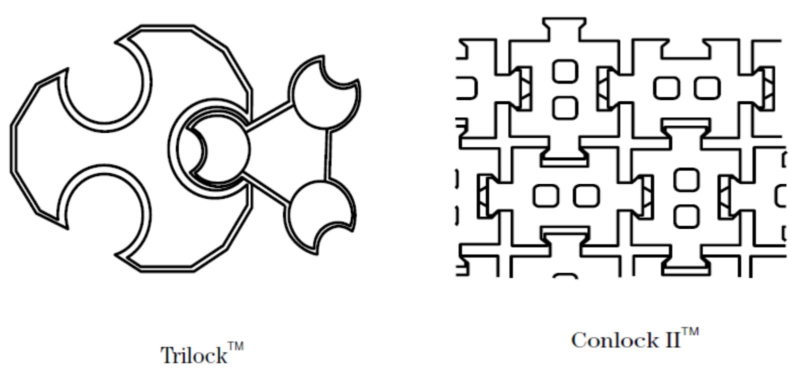

Fig. 28. Examples of interlocking ACB systems (NRCS, 2007)

2. <u>Cable-tied</u>. In this type, the prefabricated blocks are connected with ropes or cables, which facilitate their transport by crane to the installation site (Fig. 29). Within cable-tied systems, there is a distinctive type of protection using geogrids filled with mortar, whose operating principle closely resembles that of cable-tied prefabricated blocks. This typology will be addressed in this monograph under the section dedicated to geomembranes.

Fig. 29. Transporting Armorflex Block Blankets™ to the cofferdam of the El Portugués Dam (DRAGADOS)

3. <u>Overlapping</u>. These systems consist of prefabricated blocks that partially overlap each other (Fig. 30). The best-known product is the wedge-shaped block, which will be discussed in the Hard Protections section, as it achieves near-complete separation between the dam shoulder material and the overflowing water.

Fig. 30. Installation of wedge-shaped blocks in the spillway of the Barriga Dam

Currently, there are documented applications of this type of protection, including the cofferdam of the El Portugués Dam (Puerto Rico), which used a cable-tied Armorflex™ system, as well as the

Strahl Lake and Richmond Hill Mine Dams in the United States. In the case of the El Portugués Cofferdam, there were several significant overtopping events with hydraulic loads on the crest of up to approximately 1 meter. Although these caused considerable damage to the protection, they successfully prevented the cofferdam from failing.

As noted, manufacturers typically have their own installation procedures and resistance specifications for their protection systems. Both must be supported by numerical or experimental studies validating their effectiveness. Nevertheless, some general design considerations applicable to protections with ACBs can be outlined:

- Dam Crest. Experimental studies (Frizell, K. H., Mefford, Dodge, & Vermeyen, 1991; Powledge et al., 1989) have shown that failures can occur due to negative pressures developing downstream of the crest, caused by flow separation at the detachment point. To mitigate this issue, it is advisable to anchor the blocks and smooth transitions in the surface geometry, thereby preventing the formation of suction zones at slope changes between the horizontal section and the downstream face.
- Chute. The block model, its dimensions, and the type of connection between blocks are selected based on the anticipated hydrodynamic loads; reference should be made to the manufacturer's catalog for system specifications. While there are various sizing and calculation methods for channel and riverbank protections (NRCS, 2007) these are generally not applicable to this protection type. Manufacturers typically conduct experimental tests on their marketed models to certify the protection levels achieved. Such tests often do not account for the stabilizing effects provided by wires and interconnections between modules, resulting in generally conservative findings. Occasionally, at transverse joints between cable-tied block layers, a continuous concrete cord is cast to ensure protection continuity. Nevertheless, the predominant failure mode for these systems is usually not related to block or joint strength, but rather to erosion and washout of the substrate on which the blocks are installed (Fig. 31).

Fig. 31. Protection status with stitched CBAs of the No Name dam in the USA (Schweiger, Shaffer, & Nadeau, 2016)

ACB systems are generally installed on a leveled surface with at least one support layer and, in some cases, additional transition layers that serve as filters and drains. These layers must be designed according to the properties of the underlying material. Their proper performance is essential to ensure the internal stability of the structure when flow moves over the blocks. Occasionally, a geotextile is used in place of a granular filter. When geotextiles or similar membranes are positioned beneath the blocks, it is important to verify that they do not create a preferential sliding surface incapable of resisting the shear caused by flow over the blocks (FD in Fig. 32). Instances of failure have been reported due to block mats sliding along the interface with geotextile sheets or geogrids. In such cases, supplementary fastening or anchoring measures must be implemented to avoid movement.

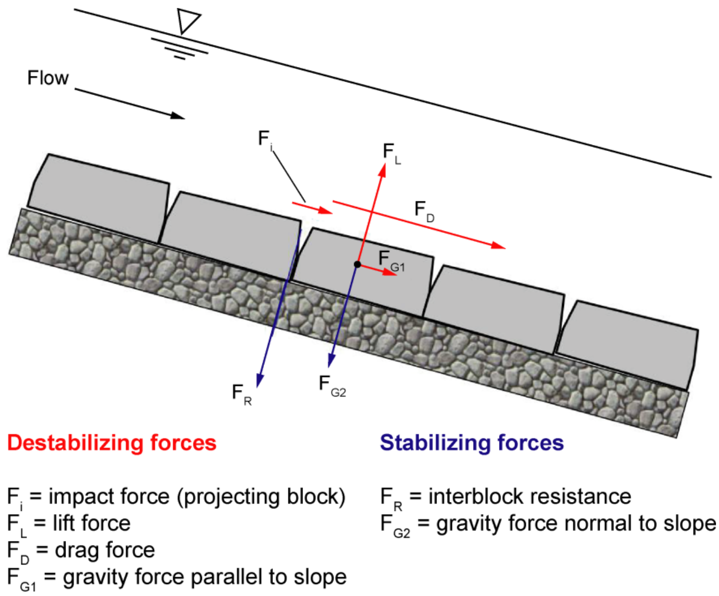

Fig. 32. Diagram of forces acting in an ACB protection system (FEMA, 2014)

- Dam Toe. At this stage, two key aspects must be addressed. First, it is essential to ensure that both the drainage flow generated by the protection system and that from the internal drainage of the dam body can be properly discharged. Second, measures must be taken to prevent the energy dissipation in this zone from causing further damage. This type of protection is highly susceptible to direct flow impacts and to pressure fluctuations from intense turbulence, such as those present near hydraulic jumps. Consequently, FEMA typically recommends that the termination of ACB systems includes an engineered anchor trench. If protection extends downstream from the dam's toe, the use of heavier blocks in this area is advised to resist the higher turbulence encountered. Specifically, for interlocking block systems, their installation near the toe of the dam should be avoided if hydraulic jumps are expected.

As previously noted, a major advantage of this type of system—particularly those stitched with cables—is the ability to rapidly install block mats, which can be delivered directly from the truck to the installation site by crane. Interlocking blocks, in contrast, are supplied individually and positioned one by one, thus reducing this benefit. Nevertheless, they remain more economical and quicker to install than in-situ concrete protections. Another advantage is the existence of official ASTM specifications (D6684, D7276-08, D7277-08) addressing their use in overspill protection. However, as stated in installation guides (AYRES Associates, 2001; NCMA, 2010; NRCS, 2007), such standards are not applicable in overtopping protection cases.

It is also noteworthy that these systems have documented histories of real-world applications and experimental trials. The flexibility of ACB systems to accommodate dam movements represents an operational improvement over rigid, reinforced, or mass concrete protections.

Among the drawbacks, usage is limited to regions with local manufacturers that have experimentally validated models. Furthermore, installation guides and sizing criteria are also manufacturer-dependent and may not align with the specific requirements of each project. This means that available models withstand specific conditions and cannot be used if the case exceeds those limits; for higher-performance models, new testing may be necessary, which can sometimes be impractical at prototype scale. The unit discharges that ACB systems can endure—except for overlapping block types—are moderate compared to other prefabricated solutions. Nevertheless, their cost-effectiveness and speed of installation make them competitive both for small dam protections and temporary works where a reduced protection level, and acceptance of moderate damage (provided total dam failure is avoided), is permissible—as seen in the cofferdam of El Portugués Dam. In this context, several publications (Melville, Van Ballegooy, & Van Ballegooy, 2006; Paul Schweiger & Darin Shaffer, 2013; Schweiger, Shaffer, & Nadeau, 2016) analyze failures and breakdowns in ACB systems, providing valuable insight into their limitations.

Section 3

Hard Protections

This section describes hard protection systems, which create a complete barrier between the high velocity flow and the granular material of the downstream slope of a dam. Generally, hard protection systems are more effective than soft ones, though they often require higher investment. This category includes cemented soils, artificial turf reinforced with mortar, open stone asphalt coatings, wedge-shaped blocks, mass concrete, and reinforced concrete slabs. For wedge-shaped blocks and mass concrete protections using RCC, a stepped configuration is normally used. The stepped bottom enhances energy dissipation. The design of these protections relies on principles common to stepped spillway technology.

After Section 3, the monograph provides references and links for further reading on calculation and design. These resources offer more in-depth coverage than what is presented in this monograph.

Cemented and Lime-Treated Soils

This chapter addresses protective systems composed of natural soils stabilized with cementitious binders—such as Portland cement or lime—to improve cohesion and durability. After treatment, the soil exhibits overtopping-erosion resistance (Bennabi, Herrier, & Lesueur, 2016; Herrier, G. et al., 2018), as observed in documented applications such as the Pannecière Cofferdam (France) (Mouy, 2013), Loumbila Dam (Burkina Faso) (Nombre, 2017), and Friant Kern canal banks (California, USA) (Herrier, Gontran, Berger, & Bonelli, 2012).

The case of the Pannecière cofferdam is particularly significant because it withstood an overflow at a rate of 0.04 m^3/s for 8 hours, with flow velocities reaching 5–6 m/s. The dam itself was 17 m high and protected externally with a layer of lime-treated soil. The base material consisted of granite sand with 14% fines (<80 µm) and a permeability of $k = 6.2 \cdot 10^{-5}$ m/s. The binder was mainly clinker, used at a 4% content. Currently, the ICOLD Committee on Cemented Material Dams is preparing a new bulletin detailing this construction technology, including information on mixture compositions and a portfolio of case studies. The bulletin will feature a dedicated section on resistance to external erosion.

Cement soil is produced by mixing soil with water and a specified cement content, typically ranging from 4% to 10%. This mixture increases overall cohesion and thus augments erosion resistance (Hansen, 2002; Pheng, Hibi, Hori, & Kohgo, 2019). In the United States, cement soil has frequently been used as an alternative to riprap to protect upstream dam faces from wave action, as documented in projects such as the Merrit, Cheney, and Ute dams. However, there are currently few published examples regarding its use for overtopping protection (Kadrmas & Huzjak, 2016). Early tests in the United States were conducted on the spillway of Broad Canyon Dam. The Alvin J. Wirtz Dam in Texas is considered the main reference for soil-cement protection against overtopping flows (Fig. 33).

Fig. 33. View from downstream of the Alvin J. Wirtz dam with its slope protected with cement soil (Photo: Larry D. Moore, CC BY-SA 4.0, https://commons.wikimedia.org/w/index.php?curid=46264825)

For Alvin J. Wirtz Dam, the construction material included 136 kg/m^3 of cement and an equal amount of fly ash. Consequently, the total binder content required to achieve the target compressive strength of 140 kp/cm^2 was 272 kg/m^3. The maximum aggregate size used was 6.35 mm, and the overall volume of concrete placed amounted to approximately 120,000 m^3.

The installation technique (Fig. 34) closely resembles that of roller-compacted concrete (RCC), but it incorporates cost-reducing simplifications, including the omission of side formwork and the lack of horizontal joint treatment between lifts.

Fig. 34. Installation of cement soil protection (Photo: Korey J. Kadrmas and Robert J. Huzjak)

Over the past decade, research has continued into the use of cement-soil as a construction material for spillways, such as in the emergency spillway of the Frank Jaeger Dam. Scale-model hydraulic tests were conducted with various cement-soil mixtures, analyzing their performance under water velocities approaching 10 m/s and sustained flows of up to 2.4 m^2/s for 6 hours. These tests revealed points where surface aggregate loss led to hole formation, but the protective layer itself did not suffer severe damage. The results indicated that initial signs of erosion appeared at velocities between 6–7 m/s, with the extent of damage dependent on the duration of overtopping. These findings underscore the limited durability of this solution, which should be considered when selecting spillway protections. For this reason, the FEMA manual (FEMA, 2014) advises against using cement-soil for embankment dam overtopping protection and recommends mass concrete or RCC alternatives instead.

The main advantages of cement-soil protection are rapid installation and cost savings compared to mass or reinforced concrete. The thick layer enhances stability against uplift and reduces surface deterioration due to freeze-thaw cycles. However, frequent or prolonged discharges increase surface damage, and there are additional concerns about material durability under weathering and freeze-thaw effects. Questions also remain about interlayer bonding with this construction method, though complete impermeability is not necessary since the protection does not communicate with the reservoir water load. In summary, current evidence shows that the economic benefits of cement-soil solutions are outweighed by their inferior performance relative to RCC protection.

Synthetic Turf Reinforced with Mortar

This solution was recently introduced in the United States under the Hydroturf® patent. As a newly developed technology, operational experience in dam overtopping scenarios remains limited at present. Nevertheless, the manufacturer has conducted an extensive campaign of experimental tests, confirming its resistance to high-velocity water flows (Cooley & Thorton, 2016; FEMA, 2014; Thompson, W. R. & Leyh, 2025; Watershed Geosynthetics, 2025). The system requires on-site installation and consists of a structured geomembrane placed over sequential layers of geotextile, artificial turf, and mortar infill (Fig. 35). The manufacturer certifies the lifespan of this protection system at between 50 and 100 years (ASTM G147, G7 standards). The on-site installation process involves the following phases:

1. Spreading and grading the backing layer with soil-like material.
2. Placing waterproof geomembrane sheets and sealing the edges to ensure installation continuity.
3. Installing geotextile and artificial turf over the geomembrane.
4. Applying the infill binder over the geotextile and artificial turf, smoothing it to control infill thickness and allow grass filaments to protrude.
5. Hydrating the binder to initiate setting.

Detailed information regarding installation procedures and system specifications can be found on the manufacturer's website (http://watershedgeo.com/hydroturf/).

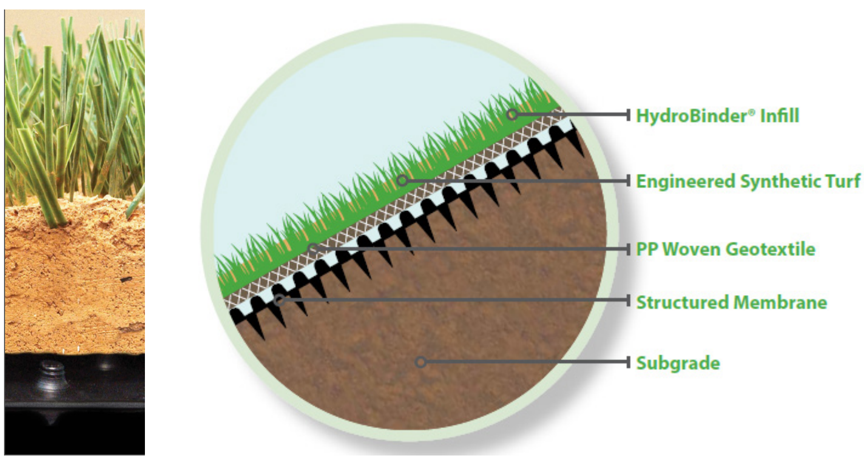

Fig. 35. Photo and schematic of the Hydroturf® protection (Ben Cooley. Watershed Geosynthetics)

The HydroTurf® system has been successfully tested in the Colorado State University test channel, applied over a highly erodible silty sand layer. The experiments demonstrated reliable protection performance on a 2:1 slope with velocities of 8.8 m/s and unit flows of 4.4 m^2/s, equivalent to an overflow height of 1.7 m. These tests reached the facility's operational limits without recording any failure or material loss from the support layer. In addition to continuous flow trials, prototype-scale functional tests were also conducted under conditions of hydraulic overhang and extreme events such as rock impacts and exposure to fire. The system's resistance to sabotage and vandalism was confirmed during these evaluations.

Key advantages of this protection system include rapid, low-impact installation and significantly lower costs compared to traditional hard armor solutions. The finished surface also achieves a visually appealing, natural grass-like appearance. The primary limitation is its proprietary status, which necessitates specialized installers and complicates implementation in areas lacking qualified personnel. HydroTurf® has been implemented for protection for small dams such as Richmond Square Dam, Victory Lakes Dam, and Walsh Ranch Lake Spillway. Further validation in real-world scenarios is pending, with aspects such as possible anchoring at the dam body, drainage detail, and construction solutions at the crest and toe of the dam remaining to be addressed.

Bituminous Coatings: *Open Stone Asphalt*

The bituminous coatings of Open Stone Asphalt (OSA) are designed based on research conducted at the Flanders Hydraulics laboratory and the University of Karlsruhe, as demonstrated by experimental test campaigns in prototype conditions (Bieberstein, Quieber, & Wörsching, 2004; Elskens, 1995). This protective coating consists of a blend of geotextile fibers and a bituminous material.

The bituminous material itself is composed of a mixture mainly containing 80% aggregates and 20% sand mastic. The sand mastic includes filler, sand, and bitumen, and can optionally contain fibers for enhanced properties. Additionally, the outermost layer of this coating can have a layer of soil applied on top to enable surface revegetation, which significantly enhances the visual appearance of the coating, as shown in Fig. 36.

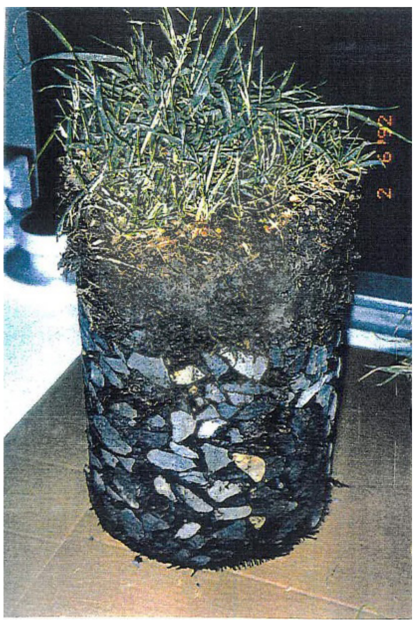

Fig. 36. Specimen of an Open Stone Asphalt type protection system (Elskens, 1995)

The dosing of materials is intended to create a semi-permeable lining that provides some drainage capacity from the inside to the outside under standard operating conditions. References for dams are still limited because design manuals restrict the use of these linings to slopes steeper than 3H:1V and maximum unit discharges of 0.8 m^2/s. Research has produced a sizing chart based on the slope (expressed as the inverse slope), the unit discharge over the slope, and the angle of friction at the interface between the protection and the soil-type material (see Fig. 37).

However, there are several documented cases of this technology being applied to dams, such as the Monchzell Dam in Germany and the Blackmoorfoot, Bodmin Town Leat, Loch Ericht and Wychall Flood Storage Reservoir in the United Kingdom (Penman, A., Hinks, J., & Wilkes, D., 2024).

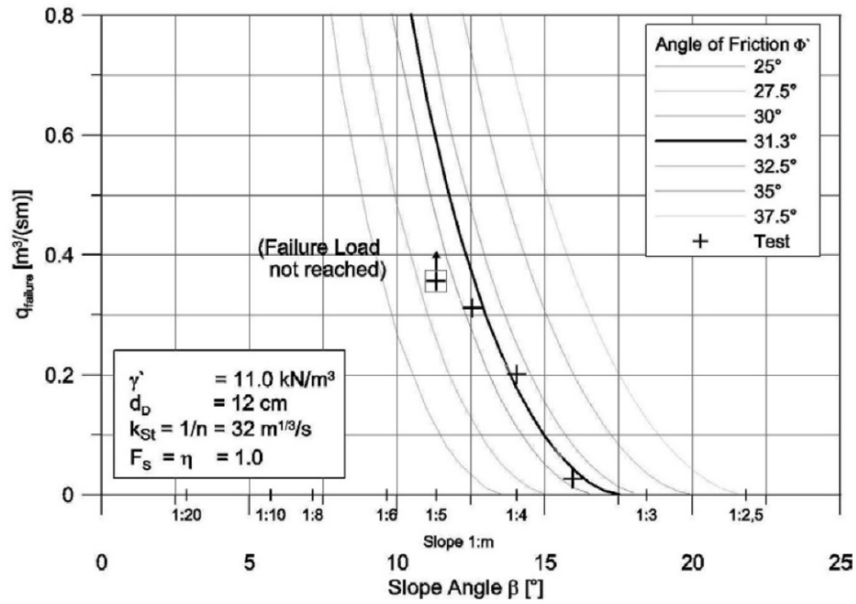

Fig. 37. Design chart of Open Stone Asphalt Protection
(Bieberstein, Quieber, & Wörsching, 2004)

This technology remains under development, with ongoing prototype-scale testing in the United States on steeper slopes and higher unit flow rates, primarily targeting wave-induced overflow in dikes. Its principal advantages include lower cost compared to other rigid protection solutions and the high-quality finish attainable after revegetation. The bituminous composition imparts a degree of adaptability to settlements that may occur in the underlying shoulder. Additionally, as a semi-permeable system, it minimizes the formation of sub-lining pressures that could destabilize the structure. The vegetative layer offers further protection, enhancing material durability against weathering.

Currently, new industry-sponsored research is being conducted to refine the technology for applications in overflowing protection for embankment dams, suggesting the potential for further advancement in the near future (https://www.hesselberg-hydro.com/materials/open-stone-asphalt-mattresses/).

Nevertheless, its use in dams is limited due to a lack of precedent, as existing design standards do not accommodate the typical slopes found in earthfill dams. Moreover, the protection capacities certified thus far remain moderate. The solution is protected under a Belgian patent (patent code WO2011067744A1, held by Deme Environmental Contractors).

Wedge-shaped Blocks

Wedge-shaped block (WSB) protections consist of a series of modular mass concrete elements that are installed in consecutive rows with overlapping joints (Fig. 38, left). When placed atop one another, these elements form a stepped lining along the discharge channel (Fig. 38, right).

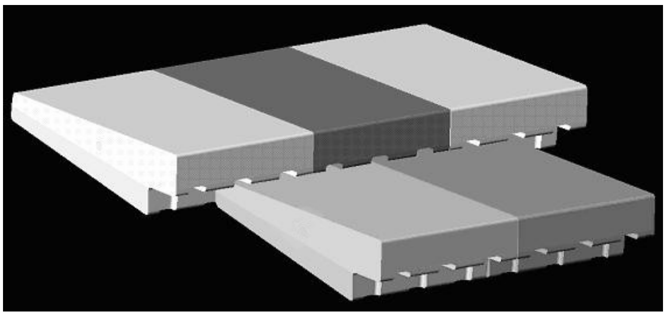

Fig. 38. Left: Configuration of the overlap between WSBs. Right: Barriga Dam spillway (Burgos, Spain)

The core principle of this technology lies in its stabilizing effect, which is achieved by the pressure distribution that develops as water moves at high velocity across the blocks (Fig. 39). The skimming flow over the stepped surface generates negative pressures immediately downstream of each block's riser. These pressures are transferred to the interface between the block and the underlying support via holes located in the riser of every block, resulting in suction that facilitates drainage of seepage flow through the block joints. Additionally, this process exerts a stabilizing influence on the block itself by reducing the positive pressure on its underside. Laboratory and experimental research have confirmed the hydraulic stability of this solution (Caballero, Francisco J., Salazar, San Mauro, & Toledo, 2015).

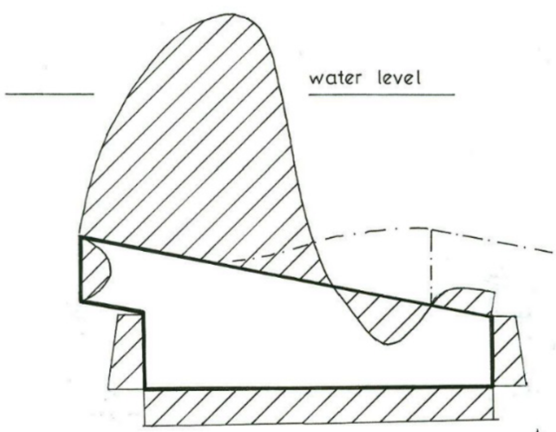

Fig. 39. Scheme of the distribution of average pressures in a wedge-shaped block (Baker 1992)

The original concept of this technology and its first applications were developed in the 1960s at the Institute of Civil Engineering in Moscow, Russia, by Professors Gordienko and Pravdivets (Morán, R., 2016). In Russia, the first scale models (1:100, 1:50, and 1:6) were constructed, and various prototype tests—such as those conducted on the Dnieper River (Pravdivets & Slissky, 1981)—were performed under extreme operational conditions, with hydraulic heads of 35 meters and unit discharges of 60 m^2/s.

The technology was further advanced by Professor Baker's group at the University of Salford in the United Kingdom, who carried out new studies (Baker & Gardiner, 1994, 1995) that included building a prototype at Brushes Clough Dam (Greater Manchester), where a unit flow of 2.2 m^2/s was tested on a slope of 3H:1V. Following this work, the Construction Industry Research and Information Association (CIRIA) published a design manual (Hewlett, H., Baker, May, & Pravdivets, 1997), which remains the most comprehensive technical guide for installation to date.

Further research was undertaken by the U.S. Bureau of Reclamation, involving experimental trials in the test channel at Colorado State University in Fort Collins(Slovensky, 1993), with a 2H:1V slope and maximum unit flows exceeding 4 m^2/s, yielding successful results. These efforts resulted in the creation of the *Armorwedge*™ block patent in the United States, which has since expired, rendering the use of this product freely available. Concurrently, the Instituto Superior Técnico in Lisbon, along with Portugal's National Laboratory of Civil Engineering, continues research on the technology, producing publications and doctoral theses that have made significant contributions to the field (Relvas, A. J., 2008; Relvas, A. T. & Pinheiro, 2008).

The prototype installations referenced in the CIRIA design manual include Bolshevik, Klinbeldin, Maslovo, Sosnovski, the Dnieper Hydroelectric Power Plant, Dneister, Kolyma, Transbaikal, and Jelyevsky in the former Soviet Union, as well as Jiangshe Wanan in China. Regarding installations in operational dams, notable examples are the spillway of the Barriga Dam (Spain), the emergency spillways of the Wadi Sahalnawt Dam (Oman) and Friendship Village Dam (USA), and the spillways at Bruton and Ogden Dams in the United Kingdom. In most cases, the system has been applied to construct emergency spillways by lowering the crest of the dam body at the selected site for the spillway, providing continuity with a block-lined discharge channel, and completing the downstream section with an energy dissipation device within the channel. Typically, the discharge channel maintains the slope of the dam shoulder, aligning the spillway axis with the normal axis of the dam body.

When used as slope protection—allowing discharge across the entire downstream surface—additional protection is required at the dam's toe to prevent potential damage from the direct impact of overflow waters on the ground.

Wedge-shaped block protections have been installed on both low-permeability and highly permeable foundation materials. The characteristics of the supporting material, particularly its permeability, are critical in designing the block support and drainage layers but are less influential when selecting or sizing the blocks themselves.

Below is a brief summary of the design criteria for different spillway components (inlet structure, chute, and energy dissipator) as published in technical guides such as CIRIA and FEMA, as well as those applied in existing spillways or dam protections. The primary consideration is that potential failure of this protection type is generally due to poor installation or inadequate design of the support layers, energy dissipation device, or spillway inlet. In summary, provided the blocks are properly installed on a geotechnically stable support base, the system will perform effectively under the certified design flow rates.

Inlet structure

The inlet structure of the protection system (or spillway) is typically located at the crest of the dam, within a lowered section specifically designed for this purpose. It is usually constructed as a reinforced concrete slab that provides continuity with the dam's impermeable element (Fig. 40). However, there are instances where the inlet is formed by articulated prefabricated blocks, such as at Bruton Dam.

At Barriga Dam, the inlet's plan geometry is defined by an elliptical transition that connect to the straight section of the trapezoidal feed channel. The bottom slab is horizontal, while the trapezoidal channel's side slopes are 2H:1V. Both upstream and downstream end of the bottom are curved with smooth transitions to match the dam's slope.

The inlet structure typically serves to anchor the first row of blocks in the discharge channel by overlapping them, thereby preventing vertical movement. The vertical joint design must be smooth to avoid direct jet impact on the block channel during discharge.

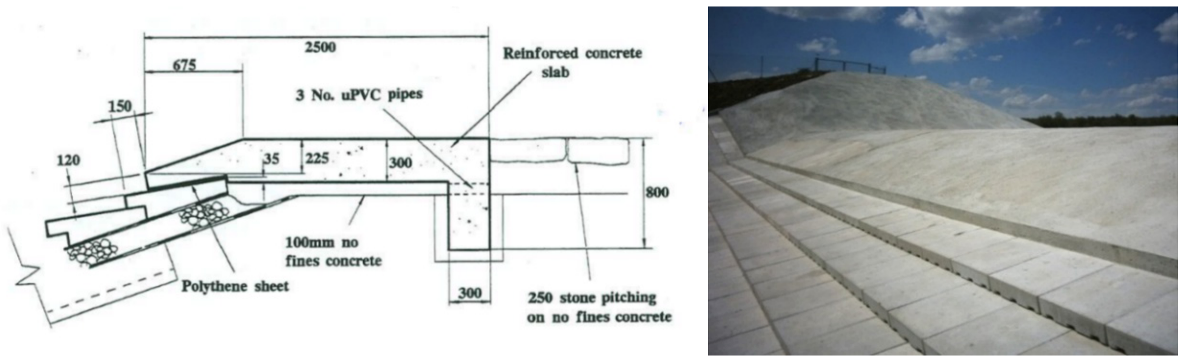

Fig. 40. Left: Cross-section of the inlet structure of the Brushes Clough Dam. Right: view from downstream of the inlet structure of the Barriga Dam.

In some cases, the flow over the dam crest occurs directly on the spillway floor, functioning as a ford (as seen in the Barriga and Klinbeldin Dams), while in other cases, the passage is supported by intermediate piers (such as at Ogden Dam).

Chute

The chute serves as the protected area of the downstream surface of the dam (or spillway) where the WSBs are installed. These blocks are arranged in overlapping rows, placed in a staggered pattern to prevent continuity of the longitudinal joints and, consequently, reduce the concentration of leakage flows along these joints. To accomplish this, half-blocks can be used in alternating rows to offset the side joints.

Typically, the chute features a trapezoidal cross-section, allowing blocks to be installed both on the supporting layer of the downstream slope and its sloped sides. This configuration minimizes the use of rigid elements, enabling the lining to adapt better to the settlements typical of embankment dams.

The initial design decision involves selecting the type and size of block based on the specific characteristics of the project. Currently, there are two main methodologies for this purpose:

1. Sizing according to the CIRIA manual (Hewlett, H., Baker, May, & Pravdivets, 1997). This methodology is general in nature, drawing on the original experiments conducted in the former

Soviet Union and supplemented by additional trials and research at the University of Salford in the 1990s. The manual provides design charts that allow for determination of the average block thickness based on the unit discharge and the longitudinal slope of the spillway, covering slopes from 2.5H:1V to 10H:1V. Once the required thickness is identified, blocks are manufactured and tailored to fit the specific project requirements.

2. Sizing according to the specifications of manufacturers' patents. In this approach, manufacturers conduct tests to validate the strength of their blocks and propose their own design methodologies. Examples include the Armorwedge™ type blocks (U.S. Patent No. 5,544,973, which is now expired) and the Acuña type block (currently covered by Spanish Patent ES 2595852 B2, Fig. 41), both of which are based on more recent research and offer improvements over earlier designs (Caballero, Francisco J., Salazar, San Mauro, & Toledo, 2015; Caballero, Francisco Javier, Toledo, Moran, & San Mauro, 2021; Caballero, Francisco Javier, Toledo, Moran, & Peraita, 2023). In both cases, experimental testing has been conducted on 2H:1V slopes, permitting their use on slopes steeper than those covered by the CIRIA guidelines. Typically, sizing is not custom-designed, but rather involves selecting from the range of models provided by the manufacturer—using the experimental verifications supplied regarding block resistance under specific conditions (such as channel slope, unit discharge, flow velocity, shear stress, etc.).

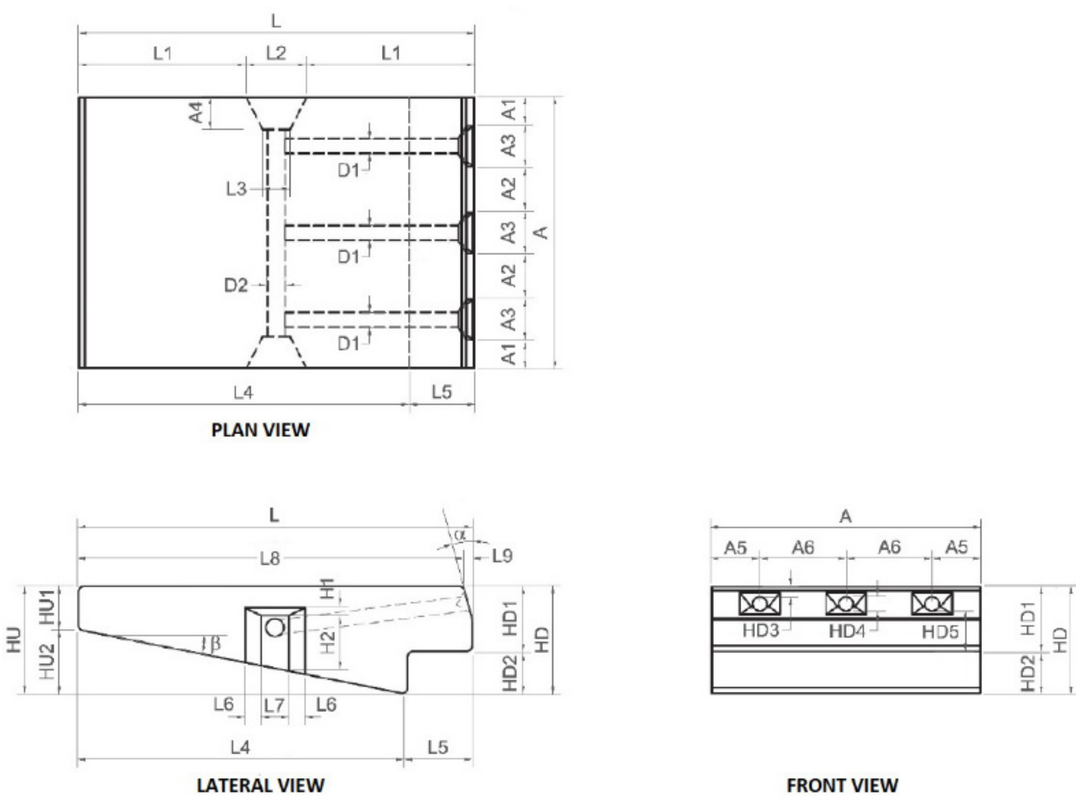

Fig. 41. Plan, lateral and front views of ACUÑA block

If, during design, the calculated stresses are lower than those from the manufacturer's testing, the chosen model is considered suitable as a protection system. In situations where project demands exceed the standard test conditions, hydraulic modeling by Froude similarity has been used to tailor the design using a specific commercial block (Morán, R., Toledo, Sevilla, & García, 2008; Morán, Rafael & Toledo, 2014).

In addition to purely hydraulic criteria, other factors are important when selecting and sizing the block model. Considerations such as resistance to vandalism, durability under freeze–thaw or wet–dry cycles, and impact resistance against debris may necessitate adjustments to block dimensions. In certain cases, modifying the concrete mix by incorporating fibers or reinforcements can enhance the impact resistance of the blocks. Another design option for added safety is to provide a transverse hole through each block, allowing a cable to be threaded along each row (Fig. 42, left), thus securing the blocks together in the event of instability affecting one of the elements. When positioned at the workpiece's center of gravity, this hole can also facilitate lifting and placement of the blocks by crane (Fig. 42, right).

Fig. 42. Barriga Dam. Left: Laying the cable through the holes of the WSBs. Right: Transporting the WSBs to the site by mobile cranes using steel bars inserted into the cross hole.

The junctions between different sections of the channel, as well as the channel's edges, are critical zones that require special consideration. Specifically, these include: the joints connecting the slab and the slopes of the discharge channel; the interface between the inlet structure and the discharge channel (upper section); and the connection between the discharge channel and the energy dissipation device (lower section). For the junction between the slab and the slope in a trapezoidal discharge channel, earlier designs have featured prefabricated curbs of uniform cross-section along the channel (Fig. 43, left), or a reinforced concrete beam with joints every 3 meters that extends the stepped layout across both planes (Fig. 43, right).

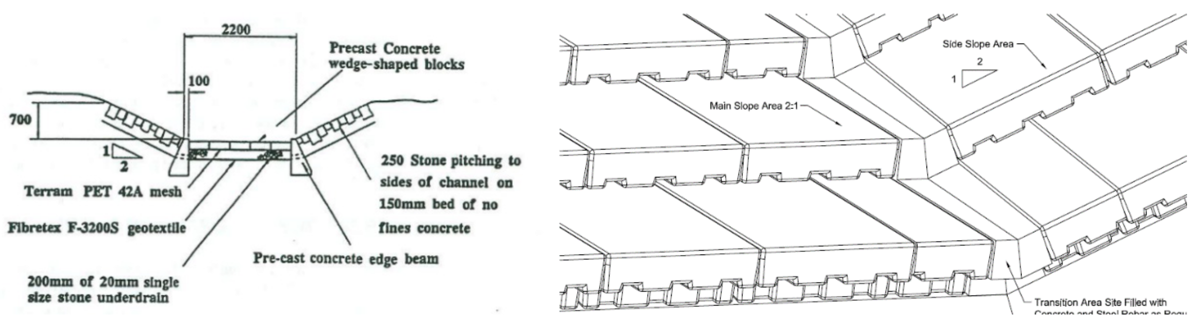

Fig. 43. Left: Spillway channel cross-section of the Brushes Clough Dam. Right: Slab-slope joint at the spillway of the Barriga Dam

Regarding the junction between the inlet structure and the chute, as mentioned earlier, its purpose is to ensure that the upstream row of blocks is supported at its upper edge by the concrete slab forming the inlet structure. From this point, the stepped chute begins, with the appropriate alignment at the joint to prevent nappe detachment (Fig. 40, right).

At the downstream end, the joint between the chute and the energy dissipation device is critical to the stability of the assembly, as it supports the bottom row of blocks, upon which the successive upstream rows are constructed. Therefore, a solid solution must be designed to provide firm support against the dissipation structure that returns water to the channel. Various solutions are possible, depending on the type of energy dissipator used. Support may be provided by a reinforced concrete structure at the dam's toe, replicating the stepped blocks (Fig. 44, left), or by a bearing on the plunge pool base at the toe of the dam (Fig. 44, right), among others.

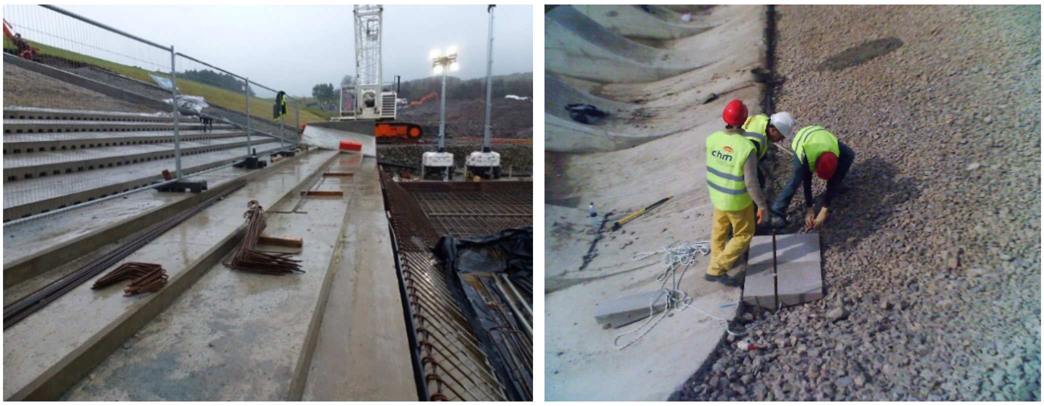

Fig. 44. Left: Support structure at the downstream end of the chute of the Odgen Dam (photo: P. Thurlwell. Pascoe, P&S Consulting Engineeers, Ltd.). Right: Placement of the first row of blocks supported on the flip bucket of the Barriga Dam

The quality and condition of the granular layer on which the blocks are installed along the dam slope are essential for ensuring system performance, since the majority of failures stem from settlement or erosion-related loss of material in these underlying layers. Additionally, the support layer must efficiently drain any seepage flows that occur during discharge, thereby preventing the development of uplift pressures that could compromise the stability of the WSBs. Consequently, blocks are typically placed over one or more layers of granular gravel with a precisely graded particle size, chosen to perform the following functions:

1. Create a uniform support surface for the blocks, enabling them to be placed in straight, properly leveled rows. Achieving this requires a moderate maximum gravel size to ensure consistent contact across the entire underside of each block. For instance, at Barriga Dam, a 20 cm thick gravel support layer was installed with an average particle size of 38 mm, limiting particles larger than 65 mm to 2% and particles smaller than 15 mm to 5%.
2. Ensure adequate drainage of the leakage flows seeping through the joints between blocks, considering the permeability of the underlying material. If the dam shoulder contains a high proportion of fine particles, the drained water will mainly flow within the drain, following a path roughly parallel to the dam slope. Conversely, if the dam material itself is permeable, it will facilitate drainage, and the thickness of the drainage layer may be less critical for flow evacuation. In both scenarios, it is vital to implement appropriate measures at the dam's toe to collect the drained water and direct it safely downstream.
3. Prevent the erosion and migration of material between underlying layers caused by water seeping through the joints between blocks. To achieve this, the lower layers must be auto

stable, and the filter compatibility condition between successive layers along the water flow direction must be satisfied.

For the preliminary sizing of the spillway discharge channel, CIRIA recommends a simplified formula (Eq. 19) to calculate the uniform flow depth (y) in the discharge channel, based on a modified Chezy formula and supported by the results of experimental testing:

$$y = K \cdot \left(\frac{q^2}{g \cdot s}\right)^{5/18} \cdot \Delta^{1/6} \qquad \text{Eq. 19}$$

Where: s is the slope of the discharge channel (as a dimensionless ratio); q is the unit flow in m^2/s; Δ, is the height of the step riser in meters; g is the acceleration due to gravity in m/s^2; and K is a coefficient with a value of 0.6 for channel height estimation and 0.46 for calculating face velocity, for example, when designing the energy dissipation device. The channel height can be determined by multiplying the computed flow depth by 1.5, ensuring sufficient freeboard. It is important to note that this calculation estimates the uniform flow depth, so flow depths near the crest will be higher than those calculated for the uniform regime.

Energy dissipation and protection of the dam toe

In practical applications constructed to date, a variety of solutions have been employed to dissipate energy at the dam's toe, including stilling basins, deflectors, and flip buckets (Fig. 45).

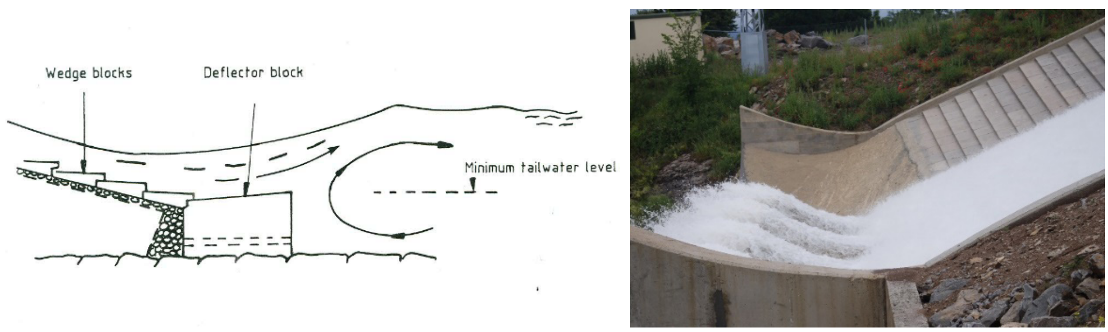

Fig. 45. Left: Cross-section of the deflector block proposed in the CIRIA manual (Baker, 1991).
Right: Flip bucket from the Barriga Dam in operation

The CIRIA manual provides sizing criteria for a deflector block at the dam toe. In all cases, the following essential conditions must be ensured at the base of the dam:

1. Prevent erosion at the dam toe that could compromise the safety of the dam structure.
2. Avoid high turbulence flows directly impacting the wedge-shaped blocks. Such flows may result from hydraulic jump formation on the blocks or direct jet impingement from downstream backwater. Laboratory tests have confirmed that these hydraulic conditions can destabilize the blocks. In cases where a hydraulic jump above the protection channel is anticipated, it may be necessary to increase block size in the affected area or replace the lower rows of the WSB lining with a rigid reinforced concrete structure (Fig. 44, left).
3. Ensure proper evacuation of drainage flows through the support layers of the chute, considering tailwater depths downstream of the dam for the design flow conditions.

Among the main advantages of using WSB protections is their cost-effectiveness compared to other hard protection methods such as mass concrete coatings or reinforced concrete slabs. Additionally, this system can adapt to the gradual settlement of the soil materials forming the dam shoulders. The ease of installation is another significant benefit, as once prepared, the blocks can be installed quickly and economically. Their erosion resistance is notable, as laboratory tests under normal operating conditions—with proper drainage of the support layer and flush flow over the WSBs—have not caused block instability.

However, one drawback is the system's sensitivity to the loss of individual blocks, since failure of a single block can compromise the entire assembly. This underscores the importance of maintenance and implementing preventive measures against sabotage, vandalism, or impact from floating debris. Moreover, the system is ineffective under hydraulic jump formation or direct jet impact on the blocks, so water flow over the WSBs must be limited to a skimming flow regime (or nappe flow with very low unit discharges). Tests to date have been limited to slopes of 2H:1V, so use on steeper slopes requires further experimental validation. Another consideration is the need to effectively transport and drain infiltration water through the drainage system.

Mass Concrete Linings

This chapter describes overtopping protection systems constructed using mass concrete coatings. Most of these projects involve roller-compacted concrete (RCC) placed on the downstream slope of the dam (Fig. 46).

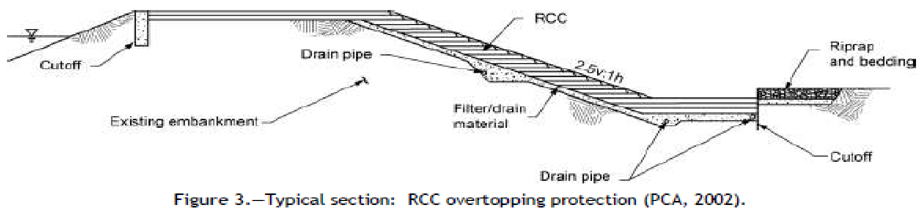

Fig. 46. Typical longitudinal cross section of a RCC protection with horizontal layers (FEMA, 2014)

For this reason, this section focuses exclusively on this type of concrete, although alternative installations could be considered if they prove economically competitive. The use of roller-compacted concrete (RCC) for dam erosion protection is a well-established technology, successfully implemented in numerous real-world cases since the 1980s. For example, in the United States alone, there are 130 documented dams in operation utilizing this protection system (Fig. 47). In several instances, these structures have withstood high unit overflow rates with positive results, prompting FEMA guidelines to recommend its use for flows up to 30 m^2/s on dams up to 60 meters in height. As a result, RCC protection is currently the most widespread and reliable protection technique. This technology has been applied both in the construction of spillways over the dam body and in direct overtopping protection. North American experience with the design and construction of this type of protection is summarized in the '*Design Manual for RCC Spillways and Overtopping Protection'* (Portland Cement Association, 2002).

Fig. 47. Examples of RCC overtopping protection in the USA. Left: Left Hand Valley Dam. Right: Leyden Dam

The construction process for the lining is similar to that used for building RCC dams. Typical binder contents average 200 kg/m^3, with maximum aggregate sizes around 40 mm. The greater thickness of this coating compared to other protection methods enables it to withstand the stresses induced by high unit discharges and impacts from large floating debris. In some cases, costs are reduced by omitting exterior surface formwork, resulting in a less aesthetic but still functional finish. Surface deterioration due to material fragility under freeze–thaw cycles has been reported where no protective treatments are applied to the external surface. From a construction standpoint, the horizontal joints between layers are treated similarly to those in RCC dams. However, a notable difference exists in the treatment of transverse joints: in these protections, such joints are typically omitted, allowing the concrete to crack freely.

From a design perspective, the discussion of this type of protection will be organized into its main components: the inlet structure, chute, and energy dissipator.

Inlet structure

The inlet structure can be constructed as a continuous RCC slab along the dam crest (Fig. 46 and Fig. 47, left), or within a recessed area designed to concentrate discharge onto a designated section of the downstream shoulder, where backrest protection is provided. This approach is the most straightforward in terms of cost and construction, but it results in a thick-walled embankment with a reduced weir coefficient. Alternatively, a wall with a hydrodynamic profile (Fig. 47, right, and Fig. 48, left) may be built as an extension of the upstream anchor trench, offering a greater weir coefficient than the continuous slab. Finally, the intake structure may be finished with a high-capacity Creager-type spillway, which entails higher construction costs due to additional concrete work and the need for curved formwork.

Fig. 48. Leyden Dam. Left: Inlet structure with spillway wall and stepped chute, seen from downstream. Right: Exit of one of the drainage conduits in the stepped slab of the chute.

The inlet structure is typically finished upstream with a cutoff wall that extends slightly deeper into the dam body, providing continuity to the dam's impermeable element.

Chute

The discharge channel is typically constructed as a stepped slab, with step heights equal to a multiple of the layer thickness used during construction—commonly 30 or 60 cm when 30 cm layers are placed. In some cases, compaction is performed perpendicular to the dam axis (parallel to the spillway axis), resulting in a flat slab parallel to the dam slope; however, this arrangement is less common, as it is more challenging to execute due to the steep slopes typical of dams. Flat slab designs are generally recommended only for dam protections with downstream slopes greater than 3H:1V and overflow heights less than 60 cm. While this configuration requires less material than horizontal casting, its unit cost is higher due to construction difficulties. It also has disadvantages, such as reduced energy dissipation and a lower resistance to the development of uplift pressures under the slab, as the coating's weight is less.

The most common option, however, is the stepped spillway (using horizontal concrete layers), which requires more material than the flat slab but is favored for easier construction and its greater energy dissipation due to the stepped profile. In stepped chutes, the concrete may be left exposed or formed. If formwork is used, it may be vertical or inclined (Fig. 48, left). Surface treatments can also be applied to improve durability and finish, which, however, further increases costs. In the United States, it is common practice not to form the steps to save on costs (Fig. 47, left), leaving the ends of the pours as unaccounted-for areas outside the effective protection thickness. Unformed or inclined-form steps dissipate less energy than vertically formed steps.

The choice of formwork approach influences the cost of the energy dissipation device—greater energy dissipation within the channel means a shorter stilling basin or plunge pool may be needed. In short, providing formed steps increases project costs and requires surface treatments on the concrete but also improves external finish, freeze-thaw and abrasion resistance, and overall energy dissipation.

The protection thickness in stepped chutes depends on the dam slope and drop height, but it is usually strongly influenced by compaction width, which is typically about 2.5 meters (measured horizontally) to allow adequate compaction with heavy machinery. Typical thickness ranges from 0.7 meters to 1 meter, with a recommended minimum of 0.6 meters.

Water within the stepped chute initially moves in a succession of falls ("nappe flow") between rows of steps. As discharge increases, flow transitions until it reaches a "skimming" regime (Fig. 49, left). The dominant hydraulic regime depends on the relative proportions of step height (h) to run (l) and critical flow depth to step height. As a rule of thumb, skimming flow generally occurs when the critical flow depth (d_c) exceeds 80% of the step height (h) (Fig. 49, right). Some studies show that maximum energy dissipation is achieved for step heights on the order of 30% of the critical flow depth (Ward, 2002).

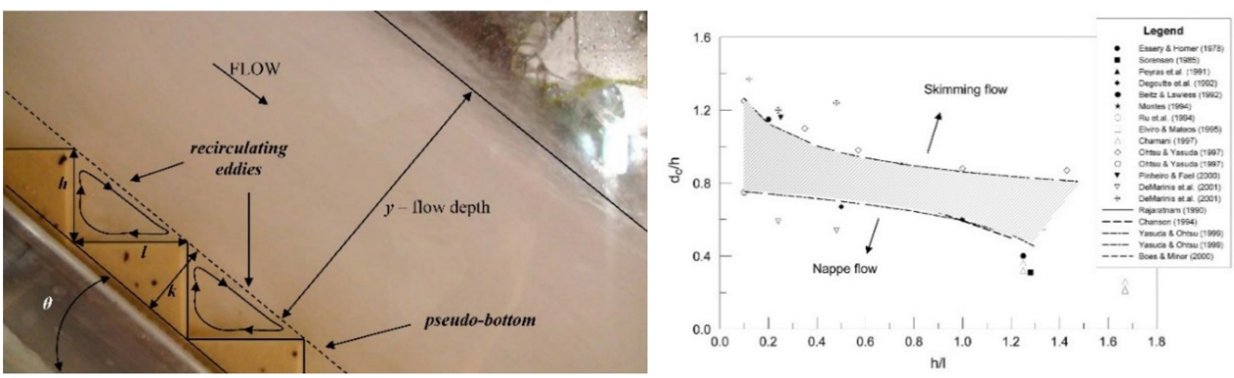

Fig. 49. Left: Skimming flow over a stepped spillway. Right: Chart for determining the type of water circulation in a stepped spillway (Frizell, Warren & Frizell, 2015).

The side walls of the chute can be constructed either from conventional reinforced concrete (Fig. 47, right) or with the same RCC used in the slab to protect the groins at the dam abutments (Fig. 47, left). For narrow spillway protections, the most economical option is to use conventional reinforced concrete walls, which can also be constructed afterward if needed. Estimating the required wall height involves hydraulic calculations that account for turbulence and aeration, and in the case of walls aligned parallel to the channel axis (Fig. 50, left), it can be simplified using methods proposed by Chanson or the Bureau of Reclamation (Chanson, 1995; Frizell, Warren & Frizell, 2015).

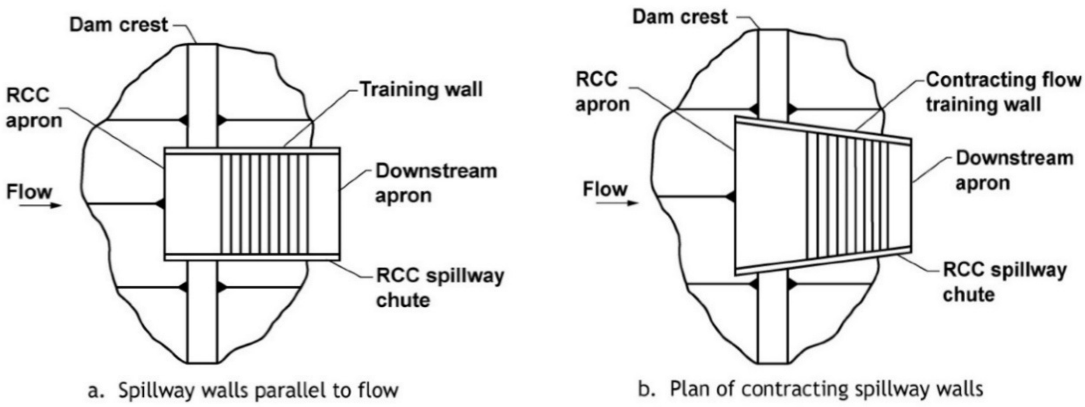

Fig. 50. Schematic plan of RCC protection. Left: Parallel side walls. Right: Converging sidewalls (FEMA 2014)

The Bureau of Reclamation proposes a calculation methodology for determining the height of side walls under uniform flow conditions, which requires estimating water depth while accounting for high levels of flow aeration. The average air concentration (C_m) in a section where uniform flow has been established can be estimated using Eq. 20.

$$\mathrm{C_m} = 0.75\cdot(\mathrm{sen}\theta)^{3/4} \qquad \text{Eq. 20}$$

Here, θ represents the angle between the plane of the pseudo-slab (the line connecting the protruding edges of the steps) and the horizontal, as illustrated as the *pseudo-bottom* in Fig. 49, left.

The modified Froude numbers (F_r and F_*) are also defined, adapted for the conditions of stepped spillways. These can be expressed in terms of either the thickness of the pseudo-slab (k, as shown in Fig. 49, left) or the step height (h, as shown in Fig. 49, left), according to Eq. 21 and Eq. 22.

$$\mathrm{F}_r = \frac{\mathrm{q_{cw}}}{\sqrt{g\cdot sen\theta\cdot k^3}} \qquad \text{Eq. 21}$$

$$\mathrm{F}_* = \frac{\mathrm{q_{cw}}}{\sqrt{g\cdot sen\theta\cdot h^3}} \qquad \text{Eq. 22}$$

Being q_{cw} unit flow under non-aerated conditions. The aerated uniform water depth (Y_{90u}) can be obtained from Eq. 23.

$$\frac{Y_{90u}}{h} = 0.5 \cdot F_*^{(0.1 \cdot \tan\theta + 0.5)} \qquad \text{Eq. 23}$$

The required height of the side walls (h_c) is estimated by applying a multiplier to the calculated flow depth as a safety margin against lateral spills. A factor of 1.2 is recommended for masonry works where spills do not cause hazardous erosion, and 1.5 should be used when channels are built on earthen dams, as is the case with protective structures. Therefore, for the case of embankment dam protection, the side wall height would be calculated following Eq. 24.

$$h_c = 1.5 \cdot Y_{90u} \qquad \text{Eq. 24}$$

The calculation of side wall height becomes more complex in cases where the channel converges laterally—that is, when the side walls are not parallel and the channel width narrows downstream (Fig. 50, right). Such configurations generate shock waves and pseudo-helical motion near the side walls, and accurately determining wall height often requires three-dimensional computational fluid dynamics modeling. However, specialized studies (Hunt, Sherry L., Kadavy, Abt, & Temple, 2008; Hunt, Sherry L., Temple, Abt, Kadavy, & Hanson, 2012; Hunt, Sherry Lynn, 2008) have investigated these convergent chutes for protecting embankment dams with RCC and can be consulted for design guidance.

As with other hard protection systems, drainage of the concrete channel is essential to prevent the buildup of interstitial pressures from leaking through the lining or from the dam shoulder itself. For RCC protections—especially those constructed with horizontal lifts—concrete thicknesses are typically substantial, and because of this mass, a much larger uplift pressure would be necessary to displace the lining. Nevertheless, where the downstream shoulder is of low permeability, drainage must be ensured by placing a gravel layer beneath the RCC (Fig. 46) and providing outlets at intervals along the channel, either by embedded pipes passing through the lining (Fig. 48, right), drainage holes, or side channels. The PCA manual specifically recommends installing vertical drains at 3-meter intervals with outlets located in the riser region to take advantage of suction generated by skimming flow over the steps. As with other cases, any added layers must be self-stable and meet filter criteria relative to the underlying material. In construction, the portion of the RCC in contact with the drainage layer cannot be compacted to the same standard as upper layers.

Sometimes a topsoil layer is applied to the RCC surface to improve its appearance and facilitate curing and frost protection; however, this cover is lost when spillway operation produces unit flows greater than what the vegetative layer can withstand.

Energy dissipation and protection of the dam toe

This aspect is crucial to the protection's effective operation because the dam toe perimeter involves a sudden change in flow direction that requires special measures to prevent damage to the dam body. Two main types of actions can be distinguished:

First is the protection of the dam toe at the abutments (Fig. 47, left), typically formed by an extension of the RCC itself. This structure absorbs the impact of incoming flows and redirects the water toward the central zone where the main energy dissipation device is located. Such protection is necessary only if the flow is not laterally confined by RCC walls that prevent water from impinging the abutments.

The second involves conventional energy dissipation at the dam toe in the riverbed area (Fig. 46). These are typically stilling basins formed by slabs composed of multiple horizontal RCC layers, with downstream concrete diaphragm walls. Stilling basins in stepped spillways tend to be significantly shorter than those in flat slab spillways; however, specific design criteria adapted for stepped spillways are lacking. The Bureau of Reclamation's Hydraulic Design Guide for Stepped Spillways (Frizell, Warren & Frizell, 2015) summarizes recent research on sizing Type III stilling basins, inlet conditions, and energy dissipation efficiency. Type III basins are usually recommended for small impoundments and dissipations with low Froude numbers, performing well even for Froude numbers above 4 (Frizell, K. W. & Svoboda, 2012; Frizell, K. W., Svoboda, & Matos, 2016). These studies conclude that hydraulic jumps stabilize much earlier in stepped spillway basins than in flat slab spillways and that chute blocks upstream of the basin are generally unnecessary in Type III stepped spillway basins for hydraulic jump stabilization.

The design procedure requires calculating the non-aerated flow depth at the pool inlet (D_1) to determine the Froude number (F_1) at the entrance, similarly to flat basin cases, using Eq. 25.

$$\mathrm{F_1} = \frac{\mathrm{V_1}}{\sqrt{\mathrm{g} \cdot \mathrm{D_1}}} \qquad \text{Eq. 25}$$

This step allows for determining the required downstream flow depth (D_2) using the classical conjugate depth equation (Eq. 26).

$$\frac{\mathrm{D_2}}{\mathrm{D_1}} = 0.5 \cdot \left(\sqrt{1 + 8 \cdot F_1^2} - 1\right) \qquad \text{Eq. 26}$$

In cases where uniform flow has been established in the channel at its entrance to the basin, once the values of (Y_{90u}) (Eq. 23) and air concentration (using Eq. 20) have been determined, the non-aerated flow depth at the inlet to the basin (D_1) —to be used in calculating the conjugate depth (D_2) and the velocity (V_1)—can be obtained using Eq. 27.

$$\mathrm{D_1} = (1 - \mathrm{C_m}) \cdot \mathrm{Y_{90u}} \qquad \text{Eq. 27}$$

From a construction standpoint, the Portland Cement Association manual provides detailed guidance for designing stilling basins as part of RCC protections, including a sample pre-dimensioning for a stepped spillway with a Type II configuration.

The primary advantage of this type of protection is its reliability, which has been demonstrated in numerous real-world cases where spill events have occurred, and the system has generally performed satisfactorily (Abdo & Adaska, 2007). For this reason, the FEMA guidelines permit the highest design unit flows for this protection method, allowing its use for dams up to 60 meters in height. It is a robust solution, resisting sabotage, vandalism, impacts from floating debris, and ensuring long-term durability. The substantial weight of the RCC lining also provides significant resistance to potential uplift pressures at the base, giving it an important advantage among hard protection options. Moreover, this solution is not restricted by patents or industrial property rights, making its application more flexible than others. In addition, there are well-established design manuals and guidelines with numerous tested examples and construction details from actual projects.

The main disadvantages are its high cost compared to alternative protections, although it may be more economical than expanding a conventional spillway to increase discharge capacity. Another consideration is that the unit cost of materials depends on the total quantity of concrete used, making the solution more attractive for large-scale projects and potentially unfeasible for smaller works. Some cases in the U.S. have highlighted issues with superficial deterioration due to low resistance to freeze-thaw cycles on the outer surface. However, this is often a result of economic decisions, as omitting surface treatments reduces overall costs without compromising the protection's effectiveness.

Reinforced Concrete Slabs

Protections using reinforced concrete slabs typically involve a continuous lining along the dam shoulder or its crest, serving as a discharge channel over the dam itself. The channel usually consists of a continuous flat slab and reinforced concrete vertical sidewalls. The hydraulic design and geometry closely resemble those of conventional external concrete spillways located on one or both abutments.

The primary difference from a conventional spillway is the slab's location relative to the dam. Here, the concrete slabs are supported directly on the embankment fill, which is far more deformable and susceptible to erosion than concrete. This construction places a rigid slab atop compacted fill, potentially subjecting it to deformations that may not accommodate. Furthermore, a lining failure in this configuration can immediately compromise the dam, leading to rapid downstream flooding. In contrast, failure of a conventional spillway lining doesn't always result in catastrophic dam failure and, if it does, the process is generally slower.

If the concrete slab protection acts as the main spillway, design precautions must be maximized to ensure reliability. If used exclusively for overtopping protection alongside a separate spillway, less stringent criteria may be applied, as overtopping events are infrequent—significantly improving dam safety compared to unprotected scenarios. The high cost of this technology has generally limited its use to spillways constructed over the dam body—either as service or emergency spillways—while its application solely for overtopping protection remains less common (Fig. 51).

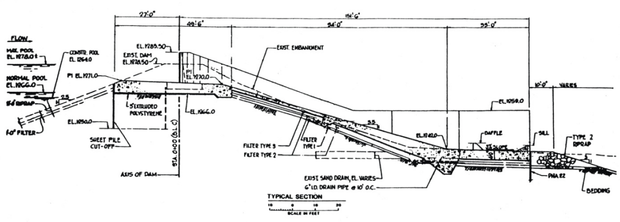

Fig. 51. Baldhill Dam (USA): View of the construction of the protection (in the background) (https://en.wikipedia.org/wiki/Baldhill_Dam).
Right: Cross section of the dam protection (T. Hepler)

There are several documented cases in Spain where spillways located in the dam body have utilized this technology, including the Molino de la Hoz and Llodio Dams (Fig. 52), both of which have demonstrated satisfactory performance over decades of operation (Alves & Morán, 2015). In the United States, additional examples include the Baldhill, Beaver Lake, Bingham Creek, Dry Creek, Green Canyon, Kinzua Upper Reservoir, Loud Thunder, and Silver Lake Flat Dams. South American references include the Guaremal and Regadera Dams, while the Ulley Dam serves as an example in the United Kingdom.

 

Fig. 52. Spillways with concrete slabs in the dam body. Left: Molino de la Hoz Dam. Right: Llodio Dam (Spain)

This technology has proven effective in real applications and has a well-documented history regarding design details and dimensioning criteria, offering a high level of safety against overtopping. As previously noted, the spillway's design closely resembles that of external spillways, with the primary differences stemming from geotechnical factors such as foundation and the interface between slabs and the dam's embankment. From a hydraulic perspective, the main issue affecting this type of protection is the relative displacement between adjoining slabs in the direction of water flow, particularly when a downstream slab is situated above the upstream slab ('vertical offset', as shown in Fig. 53).

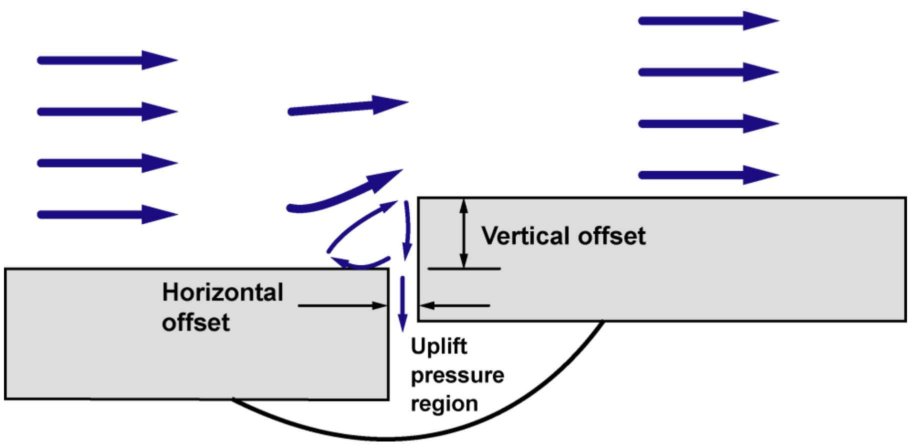

Fig. 53. Scheme of water flow pattern resulting from vertical offset between slabs (FEMA, 2014)

When this situation arises, pressures close to total (stagnation) pressure can develop at the step—this being the maximum average pressure that the circulating flow's available energy can produce. The underlying theory is straightforward and is easily derived by performing an energy balance along a streamline that closely follows the upper surface of the upstream slab (Eq. 28), from a section located upstream of the offset in Fig. 53 (section 1, not influenced by the offset) to a section at the vertical face of the offset (section 2).

$$z_1 + \frac{P_1}{\gamma} + \frac{V_1^2}{2g} = z_2 + \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + \Delta H \qquad \text{Eq. 28}$$

If the velocity at point 2 (V_2) is assumed to be zero—neglecting local effects such as eddies and flow trajectory changes—the analysis can be simplified for a horizontal slab (Eq. 29), disregarding pressure losses. (ΔH=0):

$$z_1 = z_2 \qquad \text{Eq. 29}$$

Eq. 28 is then transformed into Eq. 30:

$$\frac{P_2}{\gamma} \cong \frac{P_1}{\gamma} + \frac{V_1^2}{2g} \qquad \text{Eq. 30}$$

As observed at the step location, most of the fluid's kinetic energy is converted into pressure energy. Due to the highly turbulent regime, hydrodynamic pressures arise, resulting in pressure fluctuations along the interface between adjacent slabs. If the joint loses its seal, these fluctuations can be transmitted to the lower surface of the joint through hydraulic connectivity. The pressures generated under high flow velocities may become substantial, even with relatively minor vertical offsets between slabs. Studies by the Bureau of Reclamation (Frizell, W., 2007) measured uplift pressures and leakage unit discharges at slab joints for varying vertical ('vertical offset', Fig. 53) and horizontal ('horizontal offset', Fig. 53) displacements under different operating conditions—including both aerated and non-aerated seals (Fig. 54).

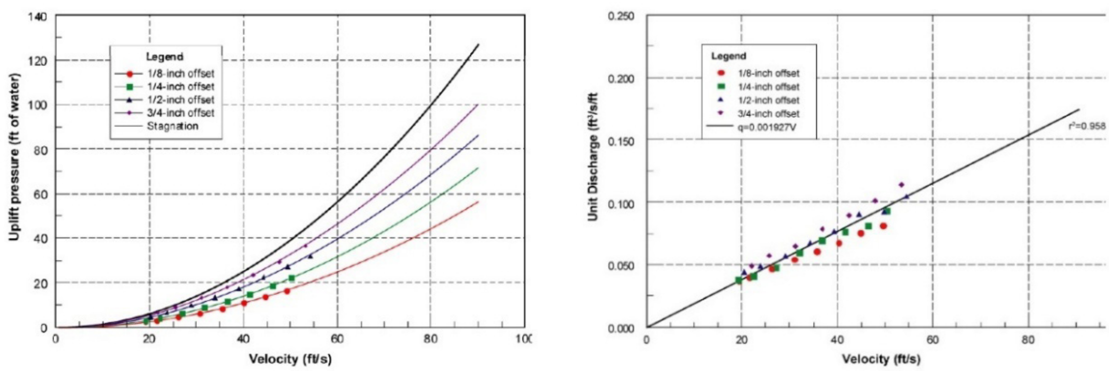

Fig. 54. Charts of uplift pressures in non-aerated conditions (left) and leakage unit discharges (right) in joints with horizontal opening of 3.2 mm with vertical displacements of 3.2; 6,4; 12.8 and 19 mm. (Frizell, W., 2007)

The uplift pressure results shown in Fig. 54 represent an envelope of the recorded pressures. Notably, the data indicate that uplift pressures and leakage flows increase as the horizontal joint openings decrease ('horizontal offset'), meaning that a smaller joint opening can be more hazardous than a larger one. Other effects may arise when such steps form between joints, including potential cavitation downstream. However, cavitation-related damage requires a certain incubation period, allowing time for repairs before severe deterioration occurs. Cavitation damage is estimated similarly to conventional spillways, based on cavitation index and expected hours of operation.

Additionally, as with other protective systems, there is a risk of failure resulting from erosion of the foundation material beneath the slabs. This may be caused by leakage flows through joints or from seepage within the dam body or its foundation.

Displacements of adjacent slabs along longitudinal joints, or at descending transverse joints (where the downstream slab lies lower than the upstream one), are less critical but should nevertheless be monitored to avoid loss of watertightness and erosion of the supporting material.

The consequences of relative displacement between slabs can be severe. In extreme instances, entire slabs may be displaced or torn away, as documented in the failures of the Big Sandy (see Fig. 55) and Hyrum Dam spillways. These failures occurred in conventional spillways that were physically separated from the dam embankments, thereby preventing catastrophic dam breach. By contrast, the failure of the Guapo Dam spillway in Venezuela directly affected the integrity of the dam itself and resulted in embankment rupture.

Good engineering practice for free-standing spillways mandates founding the reinforced concrete chute on competent rock or treated soil to minimize slab movements. In contrast, protective slabs on embankment dams rest on soil-like materials where movement is expected to be larger, necessitating special measures to prevent or mitigate damage that could compromise dam safety.

Recent research studies present experimental and theoretical analyses of uplift pressures, seepage, and hydraulic jacking at spillway chute slab joints, offering new practical equations and design guidelines to improve safety and assess failure mechanisms in dams subject to overtopping and high-velocity flow conditions (Wahl, Frizell, & Falvey, 2019; Wahl & Heiner, 2024a, 2024b, 2024c).

Fig. 55. Breaking of the slabs of the spillway channel of the Big Sandy Dam (Photo: William Fiedler)

Among the measures recommended by FEMA to prevent uplift pressures beneath slabs, in decreasing order of effectiveness, are:

- Seal the joints between slabs by installing water stop bands (WSB in Fig. 56) or employing alternative sealing methods.
- Provide transverse support beams beneath the joints to increase joint stiffness.
- Include through-reinforcement across joints to structurally connect adjacent slabs, with steel amounts in the slab reinforcement ranging from 0.5% to 0.7% of the slab section.
- Add anchor bars to cemented transverse rakes to limit potential relative movement between joints.

- Install drainage trenches beneath the joints, incorporating grooved conduits embedded in gravel and covered with filter fabrics to prevent erosion of the supporting material.

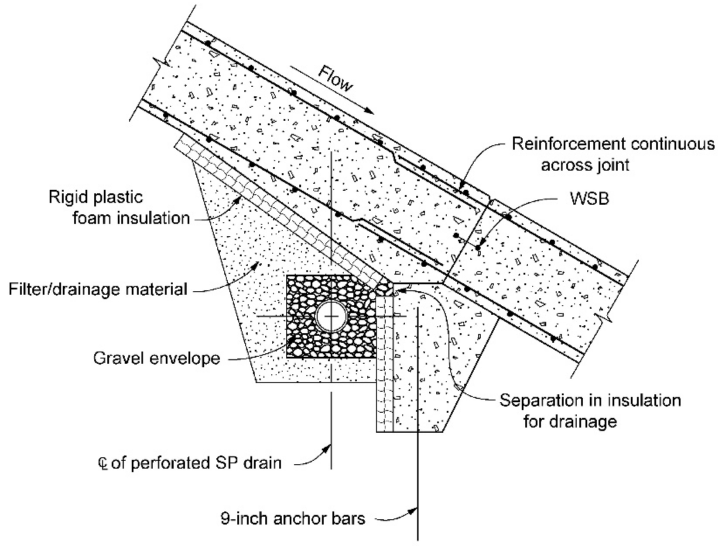

Fig. 56. Detail of joint between consecutive slabs proposed by the FEMA (William Fiedler)

The main advantages of this system are its high level of protection during normal operating conditions and its ability to withstand higher unit discharges and velocities compared to most alternative protection methods. Another benefit is its similarity to conventional spillway technology, which allows engineers to apply established design methodologies and guidelines validated through numerous real-world projects.

The primary disadvantage is the higher economic cost, along with the significant difference in stiffness between the foundation (the body of the embankment dam) and the reinforced concrete slabs. Poor adaptation to foundation settlements and differential movement between slabs are the most critical technical weaknesses of this system.

References

Abboud, B., & Coe, J. (2019). *Environmental and cost effectiveness of partially grouted riprap for scour countermeasure* (Publication No. FHWA-PA-2019-008-PIT WO 009). Pennsylvania Department of Transportation. https://rosap.ntl.bts.gov/view/dot/62351

Abdo, F. Y., & Adaska, W. S. (2007). Performance review of RCC spillways and overtopping protection. In *Proceedings of the 2008 Lawrence Kansas Dam Safety Conference*.

Abt, S. R., Thornton, C. I., Scholl, B. A., & Bender, T. R. (2013). Evaluation of overtopping riprap design relationships. *JAWRA Journal of the American Water Resources Association*, 49(4), 923–937. https://doi.org/10.1111/jawr.12061

Akter, A. (2016). A study on the hydraulic forces on geobag protected revetments. *Journal of Civil Engineering (IEB)*, 44(1), 53–63.

Alves, R. M., & Morán, R. (2015). Continuously-reinforced concrete slab spillways built over embankment dams in Spain: Molino de la Hoz and Llodio dams. In M. Á. Toledo, R. Morán, & E. Oñate (Eds.), *Dam protections against overtopping and accidental leakage* (pp. 161–174). CRC Press.

Alves, R. M., Toledo, M. Á., & Morán, R. (2016). Overflow for the complete failure of the downstream shell of a rockfill dam. [Conference paper].

AYRES Associates. (2001). Design manual for articulating concrete block systems. Harris County Flood Control District.

Baker, R. (1991). *Performance of wedge-shaped blocks in high velocity flow: Stage 2* (CIRIA Research Project 407). Construction Industry Research and Information Association.

Baker, R., & Gardiner, K. (1994). Construction and performance of a wedge block spillway at Brushes Clough Reservoir. In *Proceedings of the 8th Conference of the British Dam Society on Reservoir Safety and the Environment* (pp. 214–223).

Baker, R., & Gardiner, K. D. (1995). Building blocks. *International Water Power and Dam Construction,* 47(11), 2–5.

Bennabi, A., Herrier, G., & Lesueur, D. (2016). Lime treated soil erodibility investigated by EFA erosion testing. [Conference paper].

Berney, O., Charman, J., Kostov, L., Minetti, L., Stoutesdijk, J., & Tricoli, D. (2001). *Small dams and weirs in earth and gabion materials.* Maccaferri Gabions Ltd.

Bieberstein, J., Quieber, J., & Wörsching, H. (2004). *Open stone asphalt: A revetment for dams and embankments designed for overtopping.* Norwegian Public Roads Administration.

Blöschl, G., Hall, J., Viglione, A., Perdigão, R. A., Parajka, J., Merz, B., ... & Bilibashi, A. (2019). Changing climate both increases and decreases European river floods. *Nature*, 573(7772), 108–111. https://doi.org/10.1038/s41586-019-1495-6

Bunte, K., & Abt, S. R. (2001). Sampling surface and subsurface particle-size distributions in wadable gravel- and cobble-bed streams for analyses in sediment transport, hydraulics, and streambed monitoring (General Technical Report RMRS-GTR-74). U.S. Department of Agriculture, Forest Service, Rocky Mountain Research Station. https://doi.org/10.2737/RMRS-GTR-74

Caballero, F. J., Salazar, F., San Mauro, J., & Toledo, M. Á. (2015). Physical and numerical modeling for understanding the hydraulic behaviour of wedge-shaped-blocks spillways. In M. Á. Toledo, R. Morán, & E. Oñate (Eds.), *Dam protections against overtopping and accidental leakage* (pp. 193–205). CRC Press.

Caballero, F. J., Toledo, M. Á., Morán, R., & Peraita, J. (2023). Quasi-prototype size testing of wedge-shaped block for armoring embankment dams and levees. *Water*, 15(4), 662. https://doi.org/10.3390/w15040662

Caballero, F. J., Toledo, M. Á., Morán, R., & San Mauro, J. (2021). Hydrodynamic performance and design evolution of wedge-shaped blocks for dam protection against overtopping. *Water*, 13(12), 1665. https://doi.org/10.3390/w13121665

Chanson, H. (1995). *Hydraulic design of stepped cascades, channels, weirs and spillways.* Pergamon.

CNEGP. (2005). *Guía técnica de seguridad de presas nº 1: Seguridad de presas.* Colegio de Ingenieros de Caminos, Canales y Puertos / Comité Nacional Español de Grandes Presas.

Cooley, B. H., & Thornton, C. (2016). Using an innovative revetment for overtopping protection of levees and dams. In *Protections 2016: 2nd International Seminar on Dam Protection Against Overtopping.*

Du, Z., Zhou, J., Zhang, S., Zhong, Q., Li, H., Hu, Y., ... & Zhang, J. (2025). Experimental analysis on breaching mechanism of earth-rock dam induced by landslide generated waves. *Engineering Geology,* 346, 107913. https://doi.org/10.1016/j.enggeo.2024.107913

Elias, T., & Shirlal, K. G. (2022). Preliminary investigation on stability and hydraulic performance of geotextile sand container breakwaters filled with sand and cement. *Scientific Reports,* 12(1), 15244. https://doi.org/10.1038/s41598-022-19644-7

Elskens, F. (1995). Protecting overflow dikes for controlled flood areas in Belgium. In *PIANC Conference on Inland Waterways and Flood Control.* Brussels, Belgium.

Escarameia, M. (1998). *River and channel revetments: A design manual* (Report 20). Thomas Telford.

Federal Waterway Engineering and Research Institute (BAW). (1990). *Code of practice–Use of cement bonded and bituminous materials for grouting of armor stones on waterways.* BAW.

FEMA. (2014). *Technical manual: Overtopping protection for dams* (P-1015). U.S. Department of Homeland Security, Federal Emergency Management Agency.

Fiedler, W. R. (2016). Using risk to inform overtopping protection decisions. In *Protections 2016: 2nd International Seminar on Dam Protection Against Overtopping.*

Frizell, K. H., Mefford, B. W., Dodge, R. A., & Vermeyen, T. B. (1991). Embankment dams: Methods of protection during overtopping. *Hydro Review*, 10(2), 42–47.

Frizell, K. W., & Svoboda, C. (2012). *Performance of type III stilling basins–Stepped spillway studies* (HL-2012-02). U.S. Bureau of Reclamation.

Frizell, K. W., Svoboda, C., & Matos, J. (2016). Performance of type III stilling basins for stepped spillways. In *Protections 2016: 2nd International Seminar on Dam Protection Against Overtopping.*

Frizell, W. (2007). *Uplift and crack flow resulting from high velocity discharges over open offset joints–Laboratory studies* (Report DSO-07-07). U.S. Bureau of Reclamation.

Frizell, W., & Frizell, K. H. (2015). *Guidelines for hydraulic design of stepped spillways* (Report DSO-15-06). U.S. Bureau of Reclamation.

Hafilah Wan Ariffin, W. N., Sidek, L. M., Basri, H., Idros, N., Adrian, M. T., Abd Ghani, N. H., ... & Ahmed, A. N. (2025). Overtopping risk of high-hazard embankment dam under climate change condition. *PLoS ONE,* 20(2), e0311181. https://doi.org/10.1371/journal.pone.0311181

Hansen, K. D. (2002). *Erosion and abrasion resistance of soil-cement and roller-compacted concrete* (Research and Development Bulletin RD126). Portland Cement Association.

Haws, N. W., & Erickson, J. N. (2020). Revisiting riprap sizing relationships for overtopping flow conditions. *Journal of Hydraulic Engineering,* 146(11), 06020015. https://doi.org/10.1061/(ASCE)HY.1943-7900.0001822

Hepler, T., Fiedler, B., Vermeyen, T., Dewey, B., & Wahl, T. (2012). Overtopping protection for dams—A technical manual overview. In *Proceedings of the Dam Safety Conference.*

Herrier, G., Bonelli, S., Cornacchioli, F., Nerincx, N., Nicaise, S., Puiatti, D., ... & Tachker, P. (2018). Erosion resistant dikes thanks to soil treatment with lime. In *3rd International Conference on Protection Against Overtopping.*

Herrier, G., Berger, R., & Bonelli, S. (2012). The Friant-Kern Canal: A forgotten example of lime-treated structure in hydraulic conditions. In *6th International Conference on Scour and Erosion.*

Hewlett, H., Baker, R., May, R., & Pravdivets, Y. P. (1997). *Design of stepped-block spillways* (CIRIA Special Publication 142). Construction Industry Research and Information Association.

Hewlett, H., Boorman, L. A., & Bramley, L. A. (1987). *Design of reinforced grass waterways* (CIRIA Report 116). Construction Industry Research and Information Association.

Hiller, P. H. (2017). *Riprap design on the downstream slopes of rockfill dams* [Doctoral dissertation, University of Stellenbosch].

Hunt, S. L. (2008). *Design of converging stepped spillways* [Doctoral dissertation, Colorado State University].

Hunt, S. L., Kadavy, K. C., Abt, S. R., & Temple, D. M. (2008). Impact of converging chute walls for roller compacted concrete stepped spillways. *Journal of Hydraulic Engineering*, 134(7), 1000–1003. https://doi.org/10.1061/(ASCE)0733-9429(2008)134:7(1000)

Hunt, S. L., Temple, D. M., Abt, S. R., Kadavy, K. C., & Hanson, G. (2012). Converging stepped spillways: Simplified momentum analysis approach. *Journal of Hydraulic Engineering*, 138(9), 796–802. https://doi.org/10.1061/(ASCE)HY.1943-7900.0000584

ICOLD. (1993). *Bulletin 89: Reinforced rockfill and reinforced fill for dams–State of the art.* International Commission on Large Dams.

ICOLD. (1995). *Bulletin 99: Dam failures statistical analysis.* International Commission on Large Dams.

ICOLD. (2025). *Chengdu declaration on dams and reservoirs for energy transition and adaptation to climate change.* International Commission on Large Dams. https://www.icold-cigb.org/GB/icold/world_declarations.asp

Jandora, J., & Říha, J. (2008). *The failure of embankment dams due to overtopping.* Vutium.

Kadrmas, K. J., & Huzjak, R. J. (2016). Soil-cement for high-velocity spillway flow applications. In *Protections 2016: 2nd International Seminar on Dam Protection Against Overtopping.*

Khan, D., & Ahmad, Z. (2011). Stabilization of angular-shaped riprap under overtopping flows. *World Academy of Science, Engineering and Technology,* 59, 1153–1157.

Kiplesund, G. H., Ravindra, G. H., Rokstad, M. M., & Sigtryggsdóttir, F. G. (2021). Effects of toe configuration on throughflow properties of rockfill dams. *Journal of Applied Water Engineering and Research,* 9(4), 277–292. https://doi.org/10.1080/23249676.2021.1964388

Knauss, J. (1979). Computation of maximum discharge at overflow rockfill dams (A comparison of different model test results). In *Proceedings of ICOLD New Delhi Congress* (Q50, R9, pp. 143–160).

Kobayashi, R., Kobayashi, S., Takahashi, N., & Morii, T. (2024). Hydraulic and structural stability of overtopping protection of embankment dams using gabion mattresses. *GEOMATE Journal*, 26(117), 108–115. https://doi.org/10.21660/2024.117.4311

Koelewijn, A. R., Rikkert, S. J., Peeters, P., Depreiter, D., van Damme, M., & Zomer, W. (2022). Overflow tests on grass-covered embankments at the living lab Hedwige-Prosperpolder: An overview. *Water*, 14(18), 2859. https://doi.org/10.3390/w14182859

Lagasse, P. F., Clopper, P. E., & Arneson, L. A. (2008). Partially grouted riprap as a pier scour countermeasure. In *World Environmental and Water Resources Congress 2008: Ahupua'A* (pp. 1–10). https://doi.org/10.1061/40976(316)635

Larese, A., Rossi, R., Oñate, E., Toledo, M. Á., Morán, R., & Campos, H. (2015). Numerical and experimental study of overtopping and failure of rockfill dams. *International Journal of Geomechanics*, 15(4), 04014060. https://doi.org/10.1061/(ASCE)GM.1943-5622.0000345

Lee, D., Han, J., Kim, C., Ryu, J., Song, H., & Lee, Y. (2024). Experimental and seepage analysis of gabion retaining wall structure for preventing overtopping in reservoir dams. *Applied Sciences,* 14(10), 4041. https://doi.org/10.3390/app14104041

Lee, Y. H., Han, J. S., Yeom, T. M., Park, S. H., Song, S. W., Yun, B. R., ... & Lee, D. W. (2024). Development of overtopping protection system using gabion retaining wall in deteriorated agricultural reservoir embankment. *Korean Journal of Agricultural Science,* 51(4), 715–731. https://doi.org/10.7744/kjoas.510401

Lempérière, F., & Vigny, J. P. (2004). Safety evaluation and strengthening of existing dams in catchment areas under 500 km^2. In *International Seminar on Stability and Breaching of Embankment Dams*. Oslo, Norway.

Lempérière, F., Vigny, J. P., & Deroo, L. (2012). New methods and criteria for designing spillways could reduce risks and costs significantly. *Hydropower & Dams*, 19(3), 120–128.

Li, Y., Tian, C., Wen, L., Chen, A., Wang, L., Qiu, W., & Zhou, H. (2021). A study of the overtopping breach of a sand-gravel embankment dam using experimental models. *Engineering Failure Analysis*, 124, 105360. https://doi.org/10.1016/j.engfailanal.2021.105360

Lia, L., Vartdal, E. A., Skoglund, M., & Campos, H. E. (2013). Riprap protection of downstream slopes of rockfill dams—A measure to increase safety in an unpredictable future climate. [Conference paper].

Marr, J., Weaver, R. R., Mohseni, O., Taylor, C., Hilsendager, J., & Mielke, S. (2014). *Matrix (partially grouted) riprap lab flume study* (MnDOT Report 2014-35). Minnesota Department of Transportation.

Melville, B., Van Ballegooy, R., & Van Ballegooy, S. (2006). Flow-induced failure of cable-tied blocks. *Journal of Hydraulic Engineering,* 132(3), 324–327. https://doi.org/10.1061/(ASCE)0733-9429(2006)132:3(324)

Monteiro-Alves, R., Morán, R., Toledo, M. Á., Jimenez-Rodriguez, R., Picault, C., & Courivaud, J. (2025). Overflow-induced breaching in heterogeneous coarse-grained embankment dams and levees—A state of the art review. *Applied Sciences*, 15(16), 8808. https://doi.org/10.3390/app15168808

Morán, R. (2015). Review of embankment dam protections and a design methodology for downstream rockfill toes. In M. Á. Toledo, R. Morán, & E. Oñate (Eds.), *Dam protections against overtopping and accidental leakage* (pp. 25–40). CRC Press. https://doi.org/10.1201/b18831-5

Morán, R. (2016). Wedge-shaped blocks: A historical review. In *Protections 2016: 2nd International Seminar on Dam Protection Against Overtopping.*

Morán, R., Toledo, M. A., Sevilla, J. L., & García, J. (2008). Aliviadero con bloques en forma de cuña sobre la presa de Barriga (Burgos). In *VIII Jornadas Españolas de Presas.* Córdoba, Spain.

Morán, R., & Toledo, M. A. (2014). Design and construction of the Barriga Dam spillway through an improved wedge-shaped block technology. *Canadian Journal of Civil Engineering,* 41(10), 924–927. https://doi.org/10.1139/cjce-2014-0150

Morán, R., Toledo, M. Á., Larese, A., & Monteiro-Alves, R. (2019). A procedure to design toe protections for rockfill dams against extreme through-flows. *Engineering Structures*, 195, 400–412. https://doi.org/10.1016/j.engstruct.2019.06.002

Morán, R., Toledo, M. Á., Peraita, J., & Pellegrino, R. (2021). Energy dissipation in stilling basins with side jets from highly convergent chutes. *Water*, 13(10), 1343. https://doi.org/10.3390/w13101343

Moreno-Rodenas, A., Mantilla-Jones, J. D., & Valero, D. (2025). Age, climate and economic disparities drive the current state of global dam safety. *Nature Water*, 3, 63–74. https://doi.org/10.1038/s44221-024-00347-y

Mouy, V. (2013). Réhabilitation du barrage de Pannecière (Nièvre): Contraintes environnementales liées aux travaux. [Conference paper].

Mozer, A., Almström, B., Olsson, P. A., & Schüttrumpf, H. (2025). Estimating erosion resistance of vegetated sea dike embankments through in situ experiments. *Frontiers in Built Environment*, 11, 1528863. https://doi.org/10.3389/fbuil.2025.1528863

Muda, R. S., Kamal, N. S., Norkhairi, F. F., & Ismail, M. N. (2025). Development of ranking criteria to determine the emergency auxiliary spillway for dam overtopping failure. *International Journal of Integrated Engineering*, 17(3), 212–222. https://doi.org/10.30880/ijie.2025.17.03.017

NCMA. (2010). *Design manual for articulating concrete block (ACB) revetment systems*. National Concrete Masonry Association.

Nilsson, Å., & Rönnqvist, H. (2004). Measures to strengthening embankment dams in order to stop or control a possible through-flow process. In *International Seminar on Stability and Breaching of Embankment Dams*. Oslo, Norway.

Nombre, A. (2017). Cemented materials and improved soils: Opportunities for small dam technology. In *ICOLD-CSD Workshop at Annual Meeting.* Prague, Czech Republic.

NRCS. (2007). *Use of articulating concrete block revetment systems for stream restoration and stabilization projects* (Technical Supplement 14L). National Engineering Handbook, Natural Resources Conservation Service.

NVE. (2012). *Guidelines for embankment dams* (Guideline No. 4/2012). Norwegian Water Resources and Energy Directorate.

Paul Schweiger, P. E., & Darin Shaffer, P. E. (2013). The ABCs of ACBs–What have we learned, 12 years later armoring embankment dams, levees and spillways with ACBs. [Conference paper].

Peirson, W. L., Figlus, J., Pells, S. E., & Cox, R. J. (2008). Placed rock as protection against erosion by flow down steep slopes. *Journal of Hydraulic Engineering*, 134(9), 1370–1375. https://doi.org/10.1061/(ASCE)0733-9429(2008)134:9(1370)

Penman, A., Hinks, J., & Wilkes, D. (2024). Design and construction of an open stone asphalt spillway for Wychall Flood Storage Reservoir. In *Proceedings of the British Dam Society 2024 Conference*. British Dam Society.

Pether, R., Marsh, P., & Cartwright, P. (2009). An innovative new spillway for Bruton Flood Storage Reservoir. *Dams and Reservoirs*, 19(2), 67–72. https://doi.org/10.1680/dare.2009.19.2.67

Peyras, L., Royet, P., & Degoutte, G. (1991). Écoulement et dissipation sur les déversoirs en gradins de gabions. *La Houille Blanche*, 77(1), 37–47. https://doi.org/10.1051/lhb/1991003

Pheng, S., Hibi, K., Hori, T., & Kohgo, Y. (2019). Erosion resistance test of soil cement application for surface erosion protection. *Japanese Geotechnical Society Special Publication*, 7(2), 488–492. https://doi.org/10.3208/jgssp.v07.071

Portland Cement Association. (2002). *Design manual for RCC spillways and overtopping protection* (First ed.). Portland Cement Association.

Powledge, G. R., Ralston, D. C., Miller, P., Yung, H. C., Clopper, P. E., & Temple, D. M. (1989). Mechanics of overflow erosion on embankments. II: Hydraulic and design considerations. *Journal of Hydraulic Engineering*, 115(8), 1056–1075. https://doi.org/10.1061/(ASCE)0733-9429(1989)115:8(1056)

Pravdivets, Y. P., & Slissky, S. M. (1981). Passing floodwaters over embankment dams. *International Water Power and Dam Construction,* 33(7), 30–32.

Ravindra, G. H., Sigtryggsdottir, F. G., Asbølmo, M. F., & Lia, L. (2019). Toe support conditions for placed ripraps on rockfill dams—A field survey. VANN, 54(3), 185–199.

Ravindra, G. H. R. (2020). *Hydraulic and structural evaluation of rockfill dam behavior when exposed to throughflow and overtopping scenarios* [Doctoral dissertation, Norwegian University of Science and Technology].

Ravindra, G. H. R., Sigtryggsdottir, F. G., & Lia, L. (2018). Evaluation of design criteria for downstream riprap of rockfill dams. In *Twenty-Sixth International Congress on Large Dams* (Q101, pp. 1194–1209).

Ravindra, G. H., Sigtryggsdóttir, F. G., & Høydal, Ø. A. (2019). Non-linear flow through rockfill embankments. *Journal of Applied Water Engineering and Research,* 7(4), 247–262. https://doi.org/10.1080/23249676.2019.1640222

Real Decreto 264/2021, de 13 de abril, por el que se aprueba el Reglamento sobre Seguridad de Presas y sus Embalses. (2021). *Boletín Oficial del Estado.* https://www.boe.es/buscar/act.php?id=BOE-A-2021-5867

Relvas, A. J. (2008). *Descarregadores de cheias de blocos de betão prefabricados em forma de cunha sobre barragens de aterro* [Doctoral dissertation, Instituto Superior Técnico, Universidade Técnica de Lisboa].

Relvas, A. T., & Pinheiro, A. N. (2008). Inception point and air concentration in flows on stepped chutes lined with wedge-shaped concrete blocks. *Journal of Hydraulic Engineering*, 134(8), 1042–1051. https://doi.org/10.1061/(ASCE)0733-9429(2008)134:8(1042)

Schweiger, P., Shaffer, D., & Nadeau, J. (2016). ACB armoring potential failure modes at dam embankments and spillways. In *Protections 2016: 2nd International Seminar on Dam Protection Against Overtopping.*

Slovensky, G. G. (1993). *Near-prototype testing of wedge-block overtopping protection* [Master's thesis, Colorado State University].

Song, G., Zhao, T., Zhao, P., Fu, C., & Hu, X. (2024). Simulation of landslide dam failure due to overtopping considering wide-graded soil erosion. *Scientific Reports,* 14(1), 27272. https://doi.org/10.1038/s41598-024-78695-0

Soriano, E., Mediero, L., & Garijo, C. (2020). Quantification of expected changes in peak flow quantiles in climate change by combining continuous hydrological modelling with the modified curve number method. *Water Resources Management,* 34(14), 4381–4397. https://doi.org/10.1007/s11269-020-02670-w

Thompson, A., Oberhagemann, K., & She, Y. (2020). Geobag stability for riverbank erosion protection structures: Physical model study. *Geotextiles and Geomembranes,* 48(1), 110–119. https://doi.org/10.1016/j.geotexmem.2019.103497

Thompson, W. R., & Leyh, J. T. (2025). Enhancing embankment dam and levee resilience against overtopping erosion: The Muscatine Island Levee Project. [Conference paper].

Timblin, L. O. (1988). *Emergency spillways using geomembranes* (Report REC-ERC-88-3). U.S. Bureau of Reclamation.

Toledo, M. A. (1997). Embankment dams slip failure due to overtopping. In *Transactions of the International Congress on Large Dam*s (Vol. 4, pp. 317–330).

Toledo, M. Á., Morán, R., & Oñate, E. (Eds.). (2015). *Dam protections against overtopping and accidental leakage*. CRC Press. https://doi.org/10.1201/b18831

Toledo, M. Á., Campos, H., Lara, Á., & Cobo, R. (2015). Failure of the downstream shoulder of rockfill dams in overtopping or accidental leakage scenario. In M. Á. Toledo, R. Morán, & E. Oñate (Eds.), *Dam protections against overtopping and accidental leakage* (pp. 89–106). CRC Press.

van der Vegt, N. (2024). *Erosion of the landward revetment caused by wave overtopping: Literature report* [Master's thesis, Delft University of Technology].

Wahl, T. L., Frizell, K. W., & Falvey, H. T. (2019). Uplift pressures below spillway chute slabs at unvented open offset joints. *Journal of Hydraulic Engineering,* 145(11), 04019039. https://doi.org/10.1061/(ASCE)HY.1943-7900.0001646

Wahl, T. L., & Heiner, B. J. (2024a). Effects of joint and crack geometry on hydraulic jacking in lined and unlined spillways. J*ournal of Hydraulic Engineering,* 150(5), 04024030. https://doi.org/10.1061/JHEND8.HYENG-13972

Wahl, T. L., & Heiner, B. J. (2024b). Laboratory measurements of hydraulic jacking uplift pressure at offset joints and cracks. *Journal of Hydraulic Engineering,* 150(4), 04024016. https://doi.org/10.1061/JHEND8.HYENG-13885

Wahl, T. L., & Heiner, B. J. (2024c). Predicting uplift pressures and joint flows along a spillway chute. In *Proceedings of the 10th International Symposium on Hydraulic Structures (ISHS 2024)* (pp. 47–56). https://doi.org/10.26077/22fb-58e5

Wahl, T. L., & Mortensen, J. D. (2025). Friction factors for flow through cracks in concrete hydraulic structures. J*ournal of Hydraulic Engineering,* 151(1), 06024009. https://doi.org/10.1061/JHEND8.HYENG-14363

Ward, J. P. (2002). *Hydraulic design of stepped spillways* [Master's thesis, Colorado State University].

Watershed Geosynthetics. (2025). *Full-scale wave overtopping testing - Technical note: HydroTurf® performance testing at Colorado State University*. https://www.fergusonwaterworks.com/wp-content/uploads/2025/03/Full-Scale-Wave-Overtopping-Testing-Technical-Note.pdf

Yandem, B. (2015). *Grout design for partially grouted riprap* [Doctoral dissertation, Temple University].

About the authors

Rafael Morán

Dr. Rafael Morán holds a PhD from the Universidad Politécnica de Madrid and is an Associate Professor of Hydraulic Engineering at UPM's School of Civil Engineering. With nine years of experience in dam design and construction, his research since 2007 has focused on dam safety, including spillway hydraulics, overtopping failures of embankment dams, and protective technologies. He has participated in numerous national and industry projects. Dr. Morán is the co-director of the subcommittee on overtopping protections within the European Working Group on Overtopping Erosion of Dams (ICOLD) and contributes actively to SPANCOLD technical committees. His work has led to two industrial patents and strong international collaborations. He supervises young researchers and frequently contributes to conferences and technical journals in dam safety.

Miguel Ángel Toledo

Prof. Miguel Ángel Toledo is a Full Professor of Hydraulic Engineering at the Universidad Politécnica de Madrid, where he has carried out his teaching and research work since 1989, mainly focused on dam engineering. After an initial period of more than a decade working in consultancy, during which he combined professional and academic activity, since 2003 he has developed an intensive teaching career, delivering courses on dams and hydraulic infrastructures, alongside a strong research activity. He is the coordinator of the Dam Safety research group (SERPA), whose main lines of work are the study of embankment dam failure by overtopping and the means to protect them, and the analysis of dam safety through monitoring data using machine learning techniques.

Fco. Javier Caballero

PhD in Civil Engineering (2023), Civil Engineer (1999), Bachelor in Environmental Sciences (2002) and Master in Civil Engineering Systems (2018), all from Universidad Politécnica de Madrid (UPM). His career combines professional engineering practice and applied research, focused on dam engineering and fluvial hydraulics. He is currently Adjunct Professor of Hydraulic Engineering and Dams at UPM and Technical Director at the technology startup ACIS2in. Since 2013, he has been a member of the SERPA Research Group, contributing to research on dam protection against overtopping and unconventional spillways. His PhD received the 2023 "Carlos González Cruz" Award for best thesis in hydraulic engineering and dams. He has participated in numerous national research projects, holds two industrial patents, and actively contributes to SPANCOLD committees and international dam safety conferences.

Overtopping Protection of Embankment Dams

Technical Monograph

This book provides a comprehensive and up-to-date overview of protection technologies for embankment dams facing overtopping risks. As climate extremes increase, safeguarding hydraulic infrastructures has never been more critical. Readers will explore proven traditional methods alongside innovative solutions—wedge-shaped blocks, reinforced geotextiles, and emerging technologies—backed by real case studies and technical analysis. Designed for hydraulic engineers and technical professionals, it combines scientific foundations with practical tools for designing, assessing, and maintaining protective systems. A must-read for those committed to the safety and sustainability of dams, this book bridges cutting-edge research with hands-on experience to address one of the sector's greatest challenges.

ISBN 979-138798527-1
9 791387 985271
editorialaulamagna.com
Mc Graw Hill | AULAMAGNA PROYECTO CLAVE