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Riprap Protection Exposed to Overtopping Phenomena: A Review of Laboratory Experimental Models

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Abstract: There are increasing demands from dam safety regulations and guidelines to upgrade the rockfill dams, especially in Norway where over 180 large rockfill dams are present. To protect the hydraulic structure against overtopping events or leakages, it is important to use defence mechanisms such as a protective layer of riprap on the downstream slope. In this article, we display 9 experimental setups of riprap, conducted at the hydraulic laboratory of NTNU (Trondheim) and subjected to overtopping phenomena with increasing water discharge, until the complete failure of the model. These tests were performed on models with dumped and placed riprap, with or without toe support, with or without the downstream rockfill shoulder, and finally on models with a full dam profile. The models with downstream rockfill shoulder as well as with full dam profiles allowed for throughflow. The model behaviour during these experimental tests is described and discussed, according to their respective critical discharge values and associated failure mechanisms. Limitations are also discussed. The results bring to light the benefit of placed riprap compared to dumped riprap structures. As the results show a placed riprap can withstand a significantly higher overtopping discharge than a dumped riprap. Also, the use of toe support enables a significant increase of resistance against overtopping of placed riprap structures. However, toe supports have not proven any significant improvement in stability for dumped riprap structures. This research also puts forward that dumped riprap undergoes a surface erosion process with smaller slides. Placed riprap undergoes a sliding failure mechanism when unsupported at the toe, and a buckling deformation when supported.

Keywords: rockfill dams; dam safety; riprap; physical modelling; overtopping; protections

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1. Introduction

According to the International Commission on Large Dams (ICOLD) [1], embankment dams, constructed with locally excavated rock-fill or earth-fill materials consist of 78% of the total existing dams worldwide. The dams made for over 50% from coarse-grained material are described as rockfill dams and represent 13% of the entire world's dam population. Specifically, in Norway, more than 360 large dams (over 15 m high) are present and over half of them are rockfill dams (Figure 1).

Because of the possible disastrous consequences of a dam break, dam safety is crucial. ICOLD [1] states the overtopping phenomenon as the principal cause of embankment dam breakage (as the primary factor for 31% of the total number of ruptures, and 18% as a secondary factor). Thus, having rockfill dams equipped with defence mechanisms to protect the dam structure against unexpected overtopping or leakage events is essential from a dam safety aspect.

During the overtopping of an embankment dam, the downstream slope of the dam is subjected to highly destabilizing dynamic forces. These are produced by turbulent overflow (overtopping of the crest) and throughflow mechanisms. Here, throughflow entails flowing through the supporting fill of the dam due to overtopping of the impervious core (Figure 2a), with the top of the core at a lower level than the dam crest. Under throughflow conditions, the high-velocity turbulent flow within the dam can induce internal erosion processes, particularly if the internal stability criteria are not fulfilled with well-graded material. Furthermore, throughflow can induce external erosion in the exit zone and destabilization of the downstream embankment slope because of the pore pressure increase. If the crest is overtopped (Figure 2b,c), the downstream slope is then subjected to high-velocity, turbulent surface flow, provoking a progressive external erosion process that can lead to a dam breach.

Figure 1. Placed riprap constructed on the downstream slope of Oddatjørn dam, a 142 m high rockfill dam built in Suldal, Norway [2].

Ripraps are one of the most employed defence mechanisms for several in-stream hydraulic structures such as embankment dams, spillways, streambeds, riverbanks, bridge piers, and abutments [3–7]. For rockfill dam engineering, ripraps are encountered both on the upstream embankment and on the downstream slope. This enables protection against erosion from wave impacts and ice-induced forces on the upstream slope and protection against external erosion from accidental leakage or overtopping events on the downstream slope.

Two types of riprap structures can be distinguished on rockfill dams: placed riprap and dumped riprap. While dumped riprap consists of randomly placed stones, placed riprap corresponds to an interlocking arrangement of stones on the dam shoulder. Thanks to this specific building pattern, placed riprap has displayed greater resistance against overtopping phenomena [8,9]. Nonetheless, placed riprap construction remains more expensive than dumped riprap from an economic perspective. Also, these protective structures can be combined with the presence of toe support. A better understanding of the riprap structure and resistance against overtopping would help improve the reinforcement and building techniques.

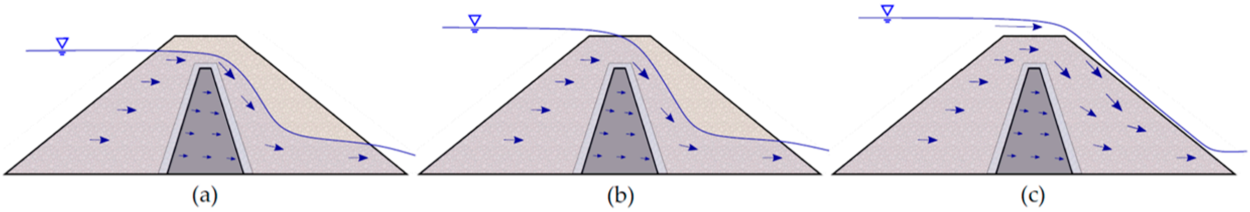

Figure 2. (**a**) Overtopping of the core with throughflow. (**b**) Partial overtopping of the crest with throughflow. (**c**) Complete overtopping of the crest with throughflow and overflow of rockfill dams. Modified from [10].

In Abt and Thornton [11], the progress in research on riprap design against overtopping is detailed, and important authors and works [6,7,12–14] are mentioned, proposing riprap design relationships for overtopping flow conditions for multiple stone sizes. Monteiro-Alves et al. [15] study the failure of the downstream shoulder submitted to overflow from a large number of experimental models with modification of specific parameters (dimensions, material size. Also, it is important to reference scientific publications on placed riprap designs that include toe support [16–20]. They demonstrate that the transport of individual stones in placed riprap does not automatically induce the failure of the whole structure and that placed ripraps display greater stability than dumped ones.

For some years, in the hydraulic laboratory of the Norwegian University of Science and Technology (NTNU), Trondheim, several experimental models have been set up. The main objective of those experiments was to investigate the failure mechanisms of riprap on a steep downstream slope exposed to overtopping events as well as to compare the resistance of different designs according to the overtopping discharge level. In this research article, a review of nine different designs is introduced and their different associated failure mechanisms as well as resistance against overtopping are detailed and discussed. The aim is to bring forth the impact of each of the following characteristics on the dam stability: presence or absence of toe support and throughflow as well as dumped riprap versus placed riprap structure.

2. Experimental Setup and Testing Program

2.1. Flume Dimension in the Hydraulic Laboratory

All nine setups of experimental models introduced and discussed in this work were built in a flume 25 m long, 2 m high, and 1 m wide at the hydraulic laboratory at NTNU. A representation of that flume from the side and top perspective is displayed in Figure 3. All the experimental models are 1:10 scale models of real rockfill dams and were all designed assuming Froude similarity. The setups imply a material section of 1 m width, built on a base platform 0.35 m high, the length of the platform was 3 m, subsequently extended to 5 m for the later full dam profile tests. The slopes of the model dams are 1:1.5 (S = 0.67). The protection materials as well as the dimensions were chosen in agreement with the Norwegian guidelines for the construction of embankment dams (Norwegian Water Resources and Energy Directorate [21]). The gradation of the supporting fill material was scaled down from a database of gradation curves from large-scale rockfill dam construction with a scaling ratio of 1:10, however, adopting the coarser range of the database due to restrictions on the use of very fine particles (<0.5 mm) in the laboratory. Kiplesund et al. [22] provide the grain size distribution of the shell material. For the construction of the dumped riprap layer, the riprap stones were placed randomly with arbitrary orientation and without any interlocking pattern. However, the placed riprap stones were placed by hand on the slope in an interlocking pattern, from downstream to upstream position. The stones were placed with an angle of β ≈ 60° between the chute-bottom and the stones' longest axis and with an angle of β ≈ 90° on the horizontal crest of the model. These values are characteristic of existing riprap structures on Norwegian dams [23].

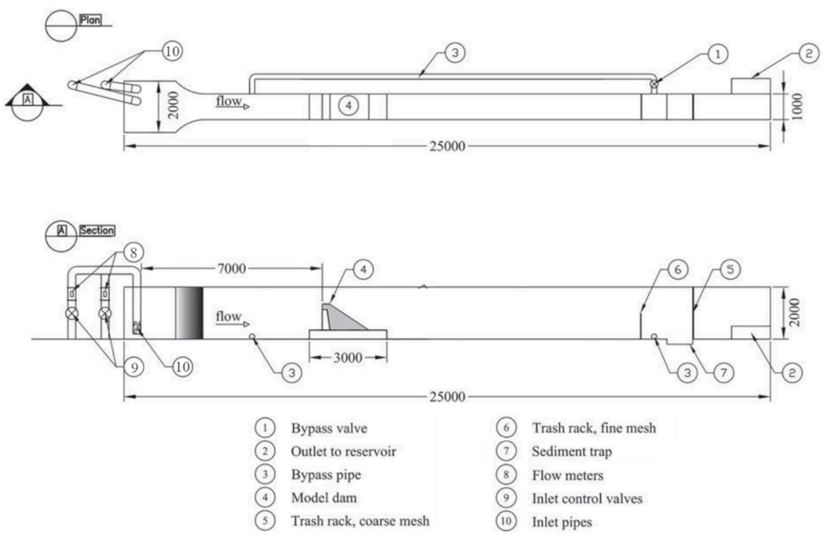

Figure 3. Representation of the flume present at the hydraulic laboratory of NTNU (Trondheim) used for the 9 setups introduced in this work. Modified from Kiplesund et al. [22].

2.2. *Different Setups*

2.2.1. Riprap Model Supported at the Toe

The first historical experimental model was displayed by Hiller et al. [8]. It consists of a scale model of a single riprap layer made from rhyolite stones on the downstream section of a dam. The median riprap stone diameter is d_{50} = 0.057 m, this is based on requirements in the NVE guidelines for embankment dams [21], with $d_{50} = (abc)^{1/3}$ where a, b and c stand for the longest, intermediate, and shortest axis of the stone, respectively a = 0.091 m, b = 0.053 m, c = 0.038 m. The density of the riprap stones being ρ_{Riprap} = 2710 kg·m^{-3}.

The riprap layer was built on a filter layer 0.1 m thick placed on an inclined ramp, with d_{50} = 0.025 m and ρ_{Filter} = 3050 kg·m^{-3}. The materials are distributed across the whole width of the flume, with a chute length of L_s = 1.8 m (Figure 4) and with a horizontal crest length of 0.55 m. Both dumped and placed riprap layers were set up and a metallic sheet perpendicular to the chute was fixed on the ramp, at the toe section, to support the riprap (Figure 4c). The riprap toe is supported in the slope direction (x-axis) but is free to move in the yz-plane.

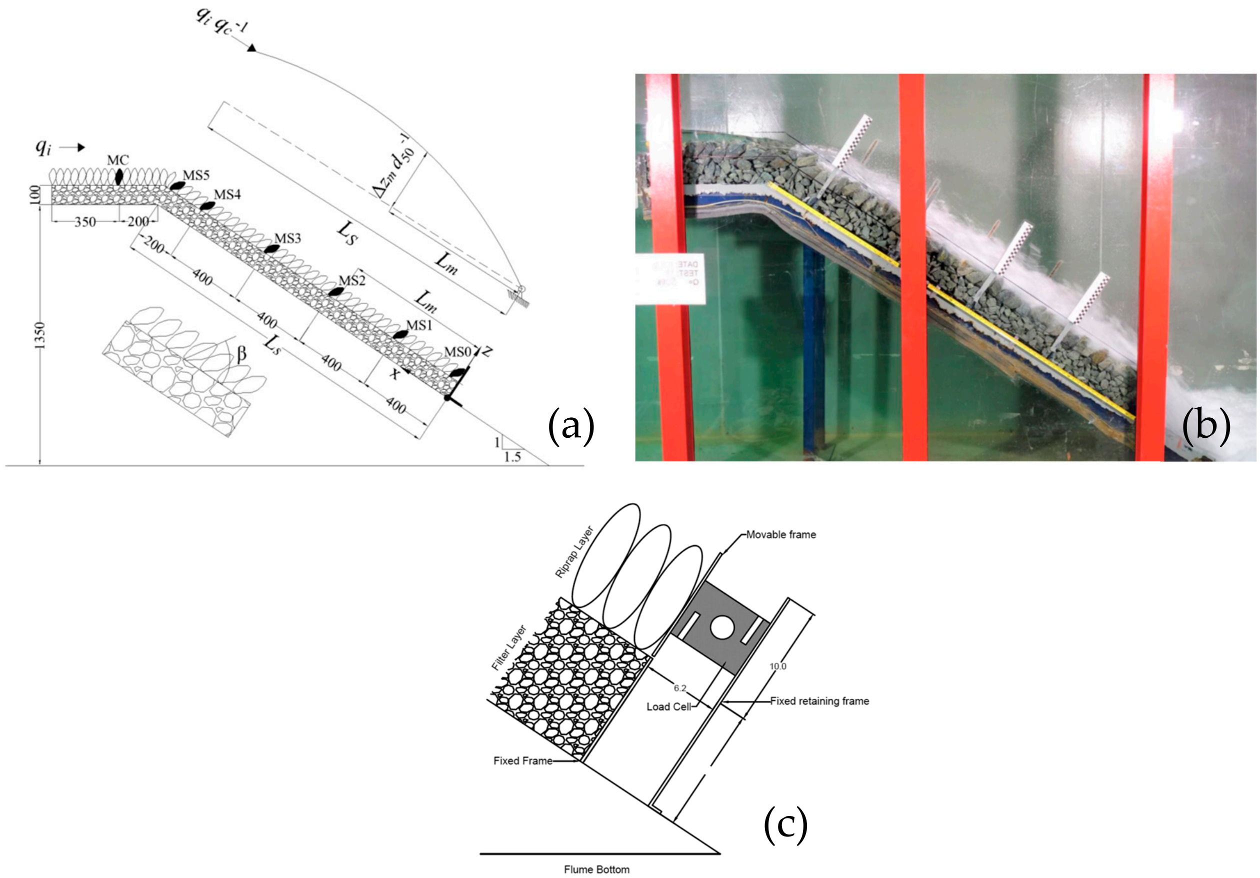

Figure 4. From Ravindra et al. [24], (**a**) depiction of experimental set-up of riprap model supported at the toe, (**b**) image of placed riprap model with such setup and (**c**) toe support positioning, from [25].

From such an experimental model, Hiller et al. [8] compare the resistance of placed riprap versus dumped riprap and studied the 1D displacement of individually placed riprap stones with laser measurements. Going further with the same model setup, Ravindra et al. [24] then demonstrate a buckling analogy for 2D deformation of placed riprap layer supported at the toe when exposed to overtopping. Then, on the same model with placed riprap, 6 load cells were added within the toe support to measure the load evolution during the experiment (Figure 4c). Thanks to these data, Dezert et al. [25] relate the 2D displacement of individual riprap stones with load values and discharge levels.

2.2.2. Riprap Model Unsupported at the Toe

The existing state of toe support conditions for placed ripraps constructed on several Norwegian rockfill dams was conducted by Ravindra et al. [2] and the study findings demonstrated that none of the surveyed ripraps were currently provided with well-defined toe support measures. Thus, additional experimental models with dumped and placed riprap were set up with a similar design as the one described in the previous section but without any toe support (Figure 5).

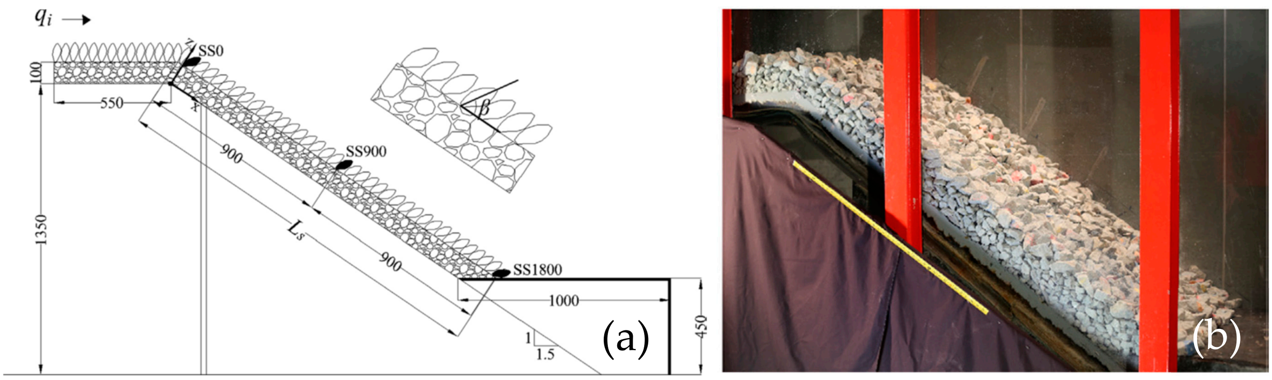

Figure 5. From Ravindra et al. [9], (**a**) depiction of experimental set-up of riprap model unsupported at the toe and (**b**) image of dumped riprap model with such setup.

In these models, the fixed toe support structure was replaced with a horizontal platform at the downstream extremity of the riprap chute to ease the construction of riprap models (Figure 5a). As the unsupported riprap model detailed previously, the chute length is equal to 1.8 m and the crest length is equal to 0.55 m. The same riprap stones and filter material were used. From these experimental tests, Ravindra et al. [8] use smartstone probes and Particle Image Velocimetry techniques to understand and discuss the failure mechanisms.

2.2.3. Half Dam Model with Unsupported Riprap

The principal objective of this research project is to have a global evaluation of rockfill dam stability during overtopping events. Hence, the next step ahead was the inclusion of ripraps with the parent dam structure, which consists in adding a rockfill shoulder. Such an addition could help in getting details regarding the interactions between the different dam components and in identifying the critical component and location for the initiation of dam failure.

Thus, tests on a model comprising the downstream half of a rockfill dam were conducted. First on half dams without riprap protection but with different toe configurations as presented by Kiplesund et al. [22]. Further tests, reported in [26], incorporated protection on the downstream shoulder with unsupported placed and dumped riprap and those are presented herein. Instrumentation is the same as detailed in [22]. The riprap layer was built on a 0.1 m thick filter layer with uniformly graded coarse rockfill of density ρ_{Filter} = 2900 $kg{\cdot}m^{-3}$, with median stone size d_{50} = 0.022 m. The riprap stones used were the same as the ones used in the previously described models. The dam shoulder is comprised of well-graded rockfill material of density $\rho_{Shoulder}$ = 2720 $kg{\cdot}m^{-3}$, median particle size d_{50} = 0.0065 m. The material was scaled down from a database of gradation curves from large-scale rockfill dam constructions with a scaling ratio of 1:10 [22] as previously mentioned. The chute length is still equal to 1.8 m with a crest length of 0.3 m. An impervious aluminum element representing the central core and filter zones was incorporated to simplify the model design. The thickness of the supporting fill at the dam crest (the vertical distance between the top of the core element and the top of the supporting fill) was set as 0.2 m (Figure 6), this is based on requirements in the Norwegian regulations for moraine core dams. Above the supporting fill comes the 0.1 m thick filter layer and the approximately 0.1 m thick riprap layer.

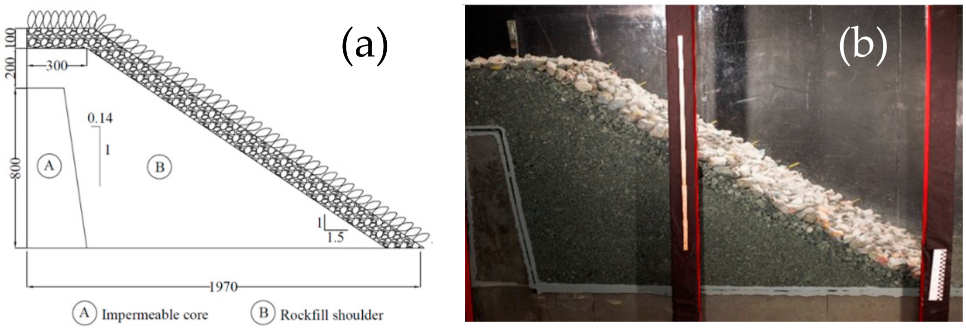

Figure 6. (**a**) Depiction of experimental setup of half dam model with core and (**b**) image of dumped riprap model with such setup from [26].

2.2.4. Half Dam Model with Riprap Supported at the Toe

The dimensioning of these additional experimental models is similar to the ones described in the previous section. They consist of a rockfill dam shoulder with a filter layer and placed ripraps on the downstream slope (Figure 7). However, the ripraps were provided with fixed toe supports and the experimental models were set up with load cells at the toe support. Such a setup would help in obtaining more details concerning interactions between the different dam components exposed to overtopping and understanding the impact of such an event on load measured at the toe. Only tests with placed riprap layers were carried out and no dumped riprap model was built.

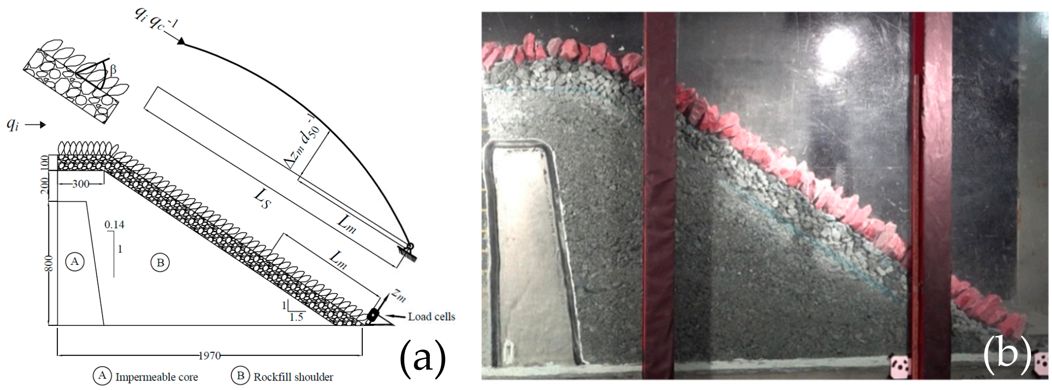

Figure 7. (**a**) Depiction of experimental setup of half dam model with core and placed riprap supported at the toe and (**b**) image of the model built in the flume.

The riprap stone density was equal to ρ_{Riprap} = 2600 kg·m^{-3} with d_{50} = 0.057 m, and the filter material density was equal to ρ_{Filter} = 3050 kg·m^{-3} with d_{50} = 0.022 m. The same shoulder material as the one used previously was used for these tests, comprising a well-graded rockfill material of density $\rho_{Shoulder}$ = 2720 kg·m^{-3}, median particle size d_{50} = 0.0065 m.

2.2.5. Full Dam Model with Unsupported Riprap

Continuing the tests on the breaching of riprap-protected dams, a series of tests were conducted on full dam profile models. For these tests also the upstream supporting fill was included in the model and the metal core element was replaced with an impermeable rubber membrane. The primary purpose of these tests was to investigate the breach development beyond what could be derived from the half dam tests due to the influence of the metal core element and the missing upstream half of the crest and supporting fill. Data on breach initiation was however also recorded for these tests as for the previous ones and thus add to the overall body of data available. A full description of the setup and instrumentation of the models can be found in [27].

Tests on a full dam profile model with unsupported placed and dumped riprap were conducted (Figure 8). The riprap stone density was equal to ρ_{Riprap} = 2600 kg·m^{-3} with d_{50} = 0.057 m, and the filter material density was equal to ρ_{Filter} = 3050 kg·m^{-3} with d_{50} = 0.022 m. The dam shoulder is comprised of well-graded rockfill material of density $\rho_{Shoulder}$ = 2720 kg·m^{-3}, median particle size d_{50} = 0.0065 m. Two of four placed riprap tests included a 0.1 m deep pilot channel along the glass wall, the purpose of this was to investigate lateral breach development which will not be discussed in this paper.

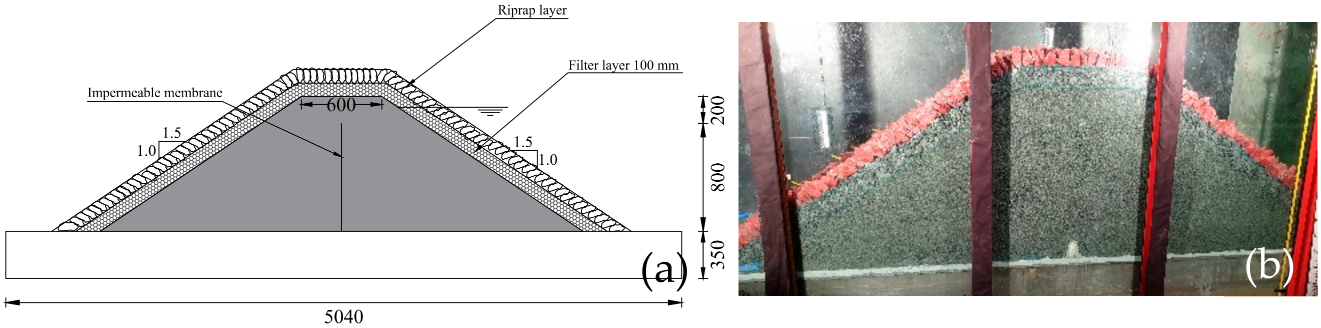

Figure 8. (**a**) Depiction of experimental setup of full dam model with flexible core, and (**b**) image of placed riprap model with such a setup.

2.3. Overtopping Procedure

The experimental models were exposed to successive overtopping events with increasing water discharge levels supplied by pumps (Figure 3) for a combined maximum delivery capacity of q = 400 L·s^{-1}, corresponding to approximately 12.6·10^3 m^3·s^{-1} considering the stone-related Froude number and a 1:10 scale model. All the models were located far enough from the inflow section to ensure calm flow conditions at the upstream part.

After the initial load period, the discharge level q_i was successively increased by fixed discharge increment (Δq) and maintained constant for a fixed time interval (Δt = 1800–3600 s) until complete failure of the model was achieved for a critical discharge value, q_c. For some of the experiments, the water flow was stopped between each discharge increment to measure the positions of the marked riprap stones with the 3D-traverse laser. Studies with the full dam models did not require such laser measurements and thus did not have the water flow stopped between discharge increases. Δq, q_i, and quantity of tests for each setup are displayed in Table 1.

Table 1. Initial discharge (q_i), discharge increment (Δq), and quantity of test for each experimental setup.

Model	q_i (10^{-3} m^2·s^{-1})	Δq (10^{-3} m^2·s^{-1})	Quantity of Test
Dumped riprap model supported	6	15–20	1
Placed riprap model supported	50–200	20–50	10
Dumped riprap model unsupported	20	20	3
Placed riprap model unsupported	20	20	6
Half dam with dumped riprap unsupported	5	5	2
Half dam with placed riprap unsupported	5	5	2
Half dam with placed riprap supported	50	25	2
Full dam with dumped riprap unsupported	10	5	1
Full dam with placed riprap unsupported	10	5	3

3. Data Analysis

Before analysing the data for each setup, a sum-up of the critical discharge values (q_c), their standard deviation (σ_{q_c}), the quantity of executed tests, and the failure mechanisms associated with each setup is displayed in Table 2. The 3 failure mechanisms are displayed in Figure 9. The standard deviations are computed according to the number of tests, critical discharge value for each test, and average critical discharge. It must be highlighted that the standard deviation values must be considered with care as the sample sizes are quite small for statistical analysis. For unsupported riprap models, the standard deviation computation cannot apply since only one model from these setups was exposed to increasing discharge values, the other ones were directly exposed to the critical discharge value obtained from the pilot test (respectively 40 and 60 $L \cdot s^{-1}$ for dumped and placed riprap model). More information on that specific point can be found in [9].

Table 2. Average and standard deviations of critical discharge, quantity of test, and failure mechanism associated with each experimental setup.

Model	Average q_c (10^{-3} $m^2 \cdot s^{-1}$)	σ_{q_c} (10^{-3} $m^2 \cdot s^{-1}$)	Quantity of Test	Failure Mechanism
Dumped riprap model supported	40	Do not apply	1	Surface erosion
Placed riprap model supported	275	72	10	2D buckling
Dumped riprap model unsupported	40	Do not apply	3	Surface erosion
Placed riprap model unsupported	60	Do not apply	6	Sliding
Half dam with dumped riprap unsupported	17.5	2.5	2	Surface erosion
Half dam with placed riprap unsupported	30	0	2	Sliding
Half dam with placed riprap supported	100	25	2	2D buckling
Full dam with dumped riprap unsupported	20	Do not apply	1	Surface erosion
Full dam with placed riprap unsupported	48	7.6	3	Sliding

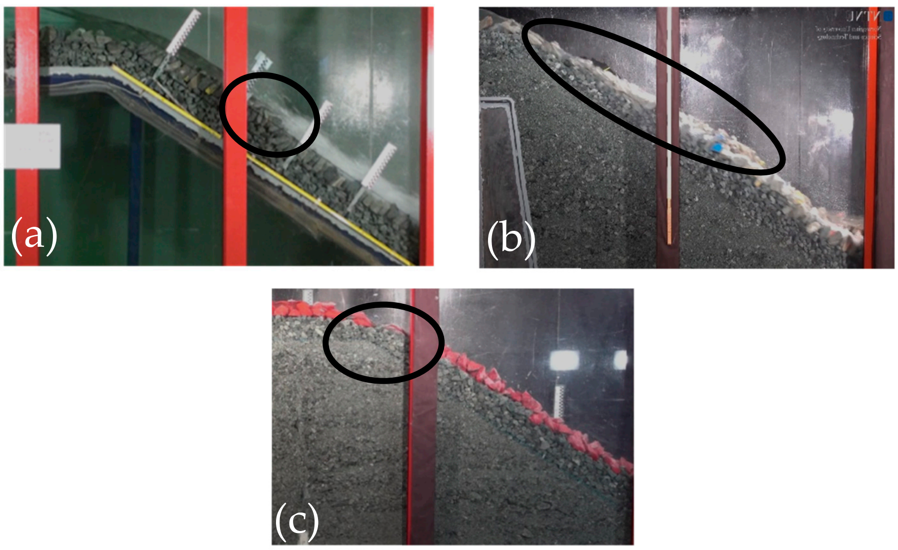

Figure 9. Image of the possible failure mechanisms from all tests with (**a**) buckling (**b**) sliding and (**c**) surface erosion mechanisms.

3.1. Riprap Model Supported at the Toe

For riprap models supported at the toe, the comparison of critical discharges for dumped and placed riprap (Table 2) demonstrates that the placed riprap (q_c = 245 L·s^{-1}) has almost seven times higher stability than the dumped one (q_c = 40 L·s^{-1}), even though this value must be considered carefully since only one test was carried out with dumped riprap model.

For the placed riprap model, Hiller et al. [8] observe that the removal of a single riprap stone does not necessarily affect the structural integrity of the whole riprap layer. The loose stones were easily identified because of their trembling motion during the overflow, and those rocks could more easily be removed by the flow. Nevertheless, it must be highlighted that some stones stabilized after some time, thanks to the compaction of the riprap, and did not get dragged downstream. Considering 1D displacement in the x-direction, the failure of the model occurred at the transition between the crest and the downstream slope, where a gap was forming. This gap formation is the direct consequence of the compaction of the riprap layer on the chute in the flow direction, increasing along with the discharge values. The displacements of the riprap stones were provoked by flow-induced vibrations. The deformation of the riprap layer could be observed in both x and z directions (Figure 9a) as explained in [24,25]. It is then interesting to point out that the mechanism of progressive 2D deformation of placed riprap stones supported at the toe can be compared to the mechanism of buckling observed in a slender-long column, pinned at one end and free at the other [24]. The explanation lies in the interlocking forces generated between the riprap stones that create a bearing structure able to resist certain levels of deformations.

For the dumped riprap model, what can be described as a surface erosion process was observed (Figure 9c). As for the placed riprap model, some individual stones were eroded by the action of destabilizing turbulent flow forces. However, what was described as a bearing structure for placed riprap model does not exist in such a setup. Thus, dumped riprap failure occurs much sooner, being the result of progressive unravelling external erosion.

3.2. Riprap Model Unsupported at the Toe

For riprap models unsupported at the toe, the comparison of critical discharges for dumped and placed riprap (Table 2) demonstrates that the placed riprap (q_c = 60 L·s^{-1}) has 1.5 times higher stability than the dumped one (q_c = 40 L·s^{-1}). However, as explained in [9], only one experimental test was carried out for placed and for dumped models with increasing discharges (q_i = 20 L·s^{-1} and Δq = 20 L·s^{-1}) so no standard deviations could be estimated. The other experimental models were directly exposed to the critical discharge value since the smartstone batteries' life used in these experiments was limited [9].

For the placed riprap model, minor displacements along the x and z axes were recorded prior to failure initiation. After the first overtopping event, the hydraulic drag and lift forces rearrange the individual stones, leading to a compaction of the riprap layer. This more compacted protection layer gained stability and formed a unified structure. So as the discharge increases, the destabilizing forces increase and some of these are partially transferred to the filter layer below as frictional forces and towards the unsupported riprap toe lying on the geotextile membrane. The sliding of the whole riprap layer occurs when the hydrodynamic forces exceed the limiting values of the static frictional forces between the toe stones and the horizontal platform [9].

For dumped riprap model, the failure mechanism is comparable to the one for the supported dumped riprap. The individual riprap stones undergo progressive erosion by the flow forces. They first resist through the self-weight of the stones and from the frictional forces between individual rocks and at the riprap-filter interface. Failure initiates when the hydrodynamic forces exceed the resultant of these self-weight and frictional forces.

3.3. Half Dam Model with Unsupported Riprap

For the half dam model with riprap unsupported at the toe, the placed riprap (q_c = 17.5 L·s^{-1}) setup has been demonstrated to be almost two times as resistant as the dumped riprap setup (q_c = 30 L·s^{-1}).

Concerning the failure mechanisms, the observations that can be made for these tests are comparable to the descriptions of the ones for the riprap model unsupported at the toe. The placed riprap setup undergoes a sliding failure with initial compaction of the riprap layer during the first overtopping stage. It can be pointed out that no important movements of the riprap stones nor of the toe stones were observed because of the throughflow. Also, the sliding mechanism started at the toe section and the rockfill shoulder of the model was stable, suggesting that the critical component is the riprap protection layer for failure initiation. The failure of dumped riprap model, as for the previous setups, could be attributed to the progressive erosion of individual riprap stones.

3.4. Half Dam Model with Riprap Supported at the Toe

The half dam models with placed riprap supported at the toe were found to fail at an average discharge value of q_c = 100 L·s^{-1}. At first sight, the mechanism is comparable to the one described for the riprap model supported at the toe, even though the failure occurs sooner. Indeed, a gap was forming on the top of the downstream slope, at the transition with the crest and this is where the initiation of the failure could be observed. The displacement of the riprap layer in both x and z directions also suggests a buckling deformation similar to that observed on the supported toe riprap model, this will still have to be confirmed in a future research article.

3.5. Full Dam Model with Unsupported Riprap

The full dam models with placed riprap unsupported at the toe were found to fail at an average discharge value of q_c = 48 L·s^{-1}, one additional test was performed without geotextile at the base, resulting in a failure discharge of 25 l·s^{-1} (not included in Tables 1 and 2). The full dam model with dumped riprap unsupported at the toe was found to fail at an average discharge value of q_c = 20 L·s^{-1}. The failure mechanism observed is quite similar to the half dam tests. There are some deviations in failure discharge both for the dumped and placed riprap models, these deviations are most likely primarily attributable to differences in the construction of the riprap layer.

4. Discussion

4.1. Role of the Toe Support

Looking at the failure mechanisms, the toe support has no impact on the surface erosion mechanism of individual stones for dumped riprap models. However, for the placed riprap models, the sliding phenomenon described for unsupported riprap is accompanied by a 2D buckling process for the supported ones. This was observed for both models built on a ramp and on a half dam. This buckling process was first described by Ravindra et al. [24].

This process is not observed within unsupported models. In such cases, some hydraulic forces are directed towards the riprap toe. Then, the static frictional forces between the toe stones and the geotextile (on the horizontal platform, Figure 4) increase to counter the increasing hydrodynamic forces transferred towards the toe. When the hydrodynamic forces exceed these static frictional forces, a displacement of the toe stones occurs and the complete riprap layer undergoes a progressive slide on the underlying filter layer. The importance of toe stone friction is also demonstrated well by the single test performed without the geotextile in place, resulting in a much lower failure discharge due to the lower sliding friction between rocks and the smooth metal platform.

It is interesting to point out that the experimental models equipped with supported placed riprap are much more resistant than the unsupported placed riprap ones (Table 2). Indeed, the average critical discharge value for supported models on a ramp (q_c = 275 L·s^{-1}) are almost 5 times more resistant than the unsupported models on a ramp (q_c =

60 L·s^{-1}). Also, the half dam models with supported placed riprap are 3 to 4 times more resistant (q_c = 100 L·s^{-1}) than the half dam models with unsupported riprap layer (q_c = 30 L·s^{-1}). It is noteworthy that the gain in resistance is huge with the simple addition of support at the toe of the structure. Nonetheless, toe support brings no benefits to dumped riprap models where the protective layer does not act as a bearing structure. The supported and unsupported dumped riprap models both failed at q_c = 40 L·s^{-1}.

4.2. Difference between Placed and Dumped Riprap for Dam Stability

The way of building a riprap layer has a significant impact on the whole structure's resistance. Both the failure mechanisms and the critical discharge when the failure occurs are quite different according to the type of riprap in place. In fact, when the protective layer consists of interlocking placed riprap stones, the observed associated failure mechanism is always a sliding process (associated with a buckling deformation for the supported riprap layer). On the other hand, dumped riprap models were always associated with surface erosion processes of individual stones and smaller slides of multiple stones that lead to failure.

Previous studies on placed riprap had already concluded that the dislodgement of individual riprap stones does not necessarily imply the failure of the whole layer [17–19]. This compilation of experimental test results also moves in that direction, highlighting that the placed riprap layer acts as a unified structure thanks to the interlocking forces and the remaining stones still offer an important resistance against turbulent flow forces.

However, these structural differences not only have an impact on the failure mechanism but also on the resistance to overtopping discharges. While the supported placed riprap models (q_c = 275 L·s^{-1}) are almost 7 times more resistant than the supported dumped riprap ones (q_c = 40 L·s^{-1}), in particular, because of the buckling process described earlier, all the other unsupported placed riprap models (built on a ramp, on the half dam, and on the full dam) demonstrated a resistance 1.5 to almost 2.5 times superior to dumped riprap ones (Table 2).

4.3. Impact of Throughflow

Even though the failure mechanisms for all models appear to initiate with surface erosion (Figure 9c) or sliding processes (Figure 9b) of the protective riprap layer, the throughflow within the dam shoulder has an important impact on the structures' stability, as previously demonstrated by [28]. When comparing the average critical discharges for models without throughflow (built on a ramp) and models with throughflow (built on half dam shoulder), structures with throughflow are 2 to 2.75 times less resistant than the ones with only overflow (Table 2). Full dam models also demonstrated a smaller resistance compared to riprap models built on a ramp, confirming the role of throughflow. The flow inside the shoulder increases the pore pressures, cumulating with the drag and lift forces from the overflow, and the destabilization of the riprap layers is enhanced.

Dumped riprap models showed a non-significant difference between half and full dam models. However, from our experiments, full dam models with unsupported placed riprap (q_c = 48 L·s^{-1}) demonstrated a resistance 1.5 times greater than what could be observed from half dam models (q_c = 30 L·s^{-1}). Such a difference could lie in a difference in the construction process from the builders.

4.4. Recommendations and Limitations

First, the repeatability of the experimental results and critical discharge values can be discussed. From Table 2, it can be observed that some results could not be associated with standard deviations on critical discharge values, either because of a lack of tests or modification of the overtopping procedure. Even the standard deviations that could be computed must be considered carefully since the number of tests remains quite limited for statistical analysis. The variability of q_c for similar models shows that even though the building procedure is the same on paper, the perfect repeatability of each test cannot be granted. According to the experience of the builder, the exact arrangement of the individual stones cannot be repeated the same way, either for placed or for dumped riprap. It is more likely that if more stones are loose in a model, they can collapse at lower discharge levels and leave part of the structure unprotected. Also, some of the full dam profile tests were done with a pilot channel in place to help provoke a breach along one wall.

The reader should also be aware that the validity of these results is confined to models with these specific material physical parameters, dimensions, and great slope value (S = 0.67) and that different outcomes could be obtained from different materials, different riprap stone shapes, and milder slopes. Especially, results that one would obtain with rounded shape stones would certainly be quite different since the interlocking pattern would not be effective with such shapes. Note also that these tests were all carried out under conditions of a fixed foundation, a dumped riprap placed on soil rather than bedrock would still need a toe structure to prevent undercutting of the toe stones, either by way of a buried toe or a horizontally extended toe structure as is common in river ripraps and rock weirs. The reader eager to learn more could also be interested in reading the recent research article [15], presenting the results from an important quantity of failure tests due to overtopping or throughflow. In further studies, it would be pertinent to focus on some specific scale effects such as the viscous scale effect, friction scale effect, and aeration scale effect that were not taken into consideration in these research works.

Globally, even if placed riprap suggests a better resistance against overtopping events, the sliding failure mechanism described seems to occur much more abruptly than the more progressive erosion surface observed with dumped riprap models. This specific point should be considered when coming to real dams and to the concerned issues located downstream of the hydraulic work.

Finally, it is worth noting that many scientific research articles were issued from all these experimental tests and that more are expected to come, considering the use of pore pressure and load data as well as structure from motion and particle image velocimetry techniques. Especially, the buckling process in the half dam supported placed riprap model will have to be studied carefully and compared to the one described in the supported placed riprap model without throughflow.

5. Conclusions

This article displayed nine different experimental models carried out within the last few years at the Department of Civil and Environmental Engineering at NTNU. These nine models implied variation of specific characteristics: the presence of dumped or placed riprap, presence or absence of toe support, and presence or absence of downstream and upstream rockfill shoulder. These models were all submitted to overtopping events with increasing water discharges until the complete failure of the structure.

Even though overtopping should always be avoided at all costs, this research has described the respective resistances against overtopping of each model as well as their associated failure mechanisms. The results of the tests highlight the importance of placed riprap protective layer in the dam resistance against overtopping processes as well as the use of toe support for placed riprap models. Also, the results underline the importance of studying the riprap resistance when built above a dam shoulder, to take into consideration the throughflow mechanisms that induce an increase in the pore pressure and destabilizing flow forces. In fact, structures with throughflow were 2 to 2.75 times less resistant than the ones with only overflow (without rockfill shoulder). This research also shows that placed riprap undergoes an abrupt sliding failure mechanism, with a buckling phenomenon when supported at the toe, while dumped riprap goes through a process of smaller slides and surface erosion.

The results from each test as well as the associated scientific discussion, when corroborated by additional data to come on complementary tests or from the bibliography, could be precious to enhance the comprehension of riprap stability on rockfill dams but also to provide recommendations for dam design and reinforcement methods for existing dams.

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