TOP LAYER STABILITY OF OVERFLOW ROCKFILL DAMS

stabilité de la carapace de barrages en enrochements submergés

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ABSTRACT

An analysis of the most important experimental data available from literature and research carried out at the Delft Hydraulics Laboratory has been performed in order to present a systematic comprehensive picture of the phenomenon and to establish clearly defined design criteria.

Design criteria involving maximum acceptable discharges and upstream waterlevels have been derived for various typical flow conditions, as a function of the hydraulic and structural parameters.

Résumé

Une analyse des données expérimentales les plus importantes disponibles dans la littérature spécialisée et issues de propres recherches du Laboratoire d'Hydraulique de Delft a été effectuée afin de présenter une vue d'ensemble systématique du phénomène et pour définir clairement les critères de calcul.

Des critères de calcul impliquant des débits et des niveaux d'eau amont maxima ont été dérivés en fonction des paramètres hydrauliques et structurels pour une variété de conditions d'écoulement.

1. Introduction

Rockfill dams are used in many ways in the field of coastal and river engineering. A few examples are closure works in rivers and estuaries, sills as a foundation for various structures, spillways in large and small dams, various river training works, etc.

A general design procedure is to minimize the stone size of the top layer, as this may result in appreciable financial savings especially in those countries where rock is expensive, such as The Netherlands. Savings may also be achieved on the handling of stones as, in most cases, the winning and handling of large stones is much more expensive than for smaller stones.

The minimum acceptable stone size in the top layer of a rockfill dam will be determined by the flow conditions expected, and therefore knowledge of the relationship between these two parameters is necessary for the preparation of an optimal design.

Many results of investigations in small and large scale models have been reported during the past two decades. Most publications however concern specific flow and design conditions and a systematic comprehensive picture of the phenomenon was not available.

In view of the large amount of data available concerning the stability of the top layer of overflow rockfill dams, the Delft Hydraulics Laboratory decided to prepare a synthesis of these data in order to provide a comprehensive picture and to deduce general design criteria. For this purpose, about 350 tests, reported in the literature and the results of its own research, were analysed.
The findings of this synthesis form the subject of the present report.

2. Theoretical aspects

2.1 Definitions

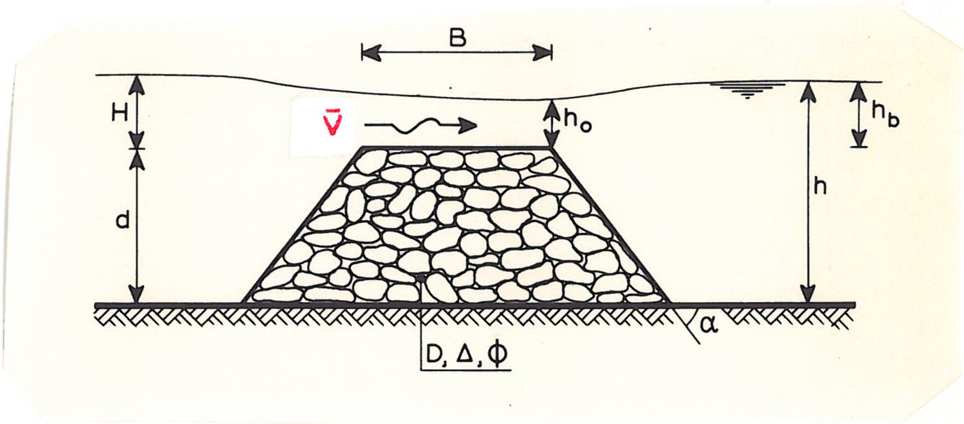


fig 1. Definitions

d	= dam height	(m)
B	= crest width	(m)
H	= upstream waterdepth related to the dam crest	(m)
h	= downstream waterdepth	(m)
h_b	= downstream waterdepth related to the dam crest	(m)
h_o	= waterdepth at the downstream crest	(m)
$\overline{V}$	= flow velocity at the downstream crest	(m/s)
α	= downstream slope of the dam	(rad)
D	= characteristic stone diameter	(m)
Δ	= relative stone density under water	(-)
ϕ	= angle of repose of the stones under water	(rad)

For reasons of simplicity the following definition of the characteristic stone diameter has been used in this study:

$$D = \left(\frac{M_{50}}{\rho_s}\right)^{1/3} \qquad (1)$$

where:

M_{50} = stone mass which is exceeded by 50% (by mass) of the stones (median stone mass). (kg)

ρ_s = stone density (kg/m^3)

Hence the stones are considered as cubes with the same mass and density.

2.2 Initiation of motion

The following relationship may be deduced from Shields' diagram for stones and gravel with $D > 10^{-3}$ m:

$$V_{*cr} = \sqrt{\Psi_{cr}} \sqrt{\Delta g D} \qquad (2)$$

in which:

V_{*cr} = critical shear stress velocity for initiation of motion (m/s)

Ψ_{cr} = Shields' parameter (-)

The value of Shield's parameter depends on the accepted damage at "initiation of motion". A commonly used value is 0.03 to 0.04.

The determination of initiation of motion is however quite subjective as it is observed visually. In the literature a differentiation is usually made between the "threshold" discharge and the "collapse" or "failure" discharge, the former referring to initiation of motion, the latter to continuous transport of stones leading to failure of the top layer. In present report the term "critical" discharge refers to the "threshold" discharge. A more detailed description of the observation of initiation of motion is given by Linford & Saunders [3].

The mean critical flow velocity can be deduced from:

$$\overline{V}_{cr} = \frac{C}{\sqrt{g}} V_{*cr} \qquad (3)$$

where:

C = Chézy roughness coefficient $(m^{\frac{1}{2}}/s)$

The roughness coefficient is given by the White-Colebrook relationship:

$$C = 18 \log \frac{12h'}{k + \delta/4} \quad (4)$$

where:

k	=	equivalent bed roughness	(m)
δ	=	thickness of the viscous sublayer	(m)
h'	=	theoretical waterdepth	

When dealing with coarse material the thickness of the viscous sublayer is negligible with respect to the bed roughness k.
In the case of uniform coarse bed material, the bed roughness can be given as k = 2D.
The theoretical level of the bottom is usally assumed to be somewhat below the top of the stones of the top layer to account for the flow within the top layer. Hence the theoretical waterdepth equals:

$$h' = h_o + a\ D \quad (5)$$

where a is a coefficient generally smaller then unity.
The critical unit discharge can be expressed as:

$$q_{cr} = \overline{V}_{cr}\ (h_o + b\ D) \quad (6)$$

where b is a coefficient similar to the above-mentioned coefficient a.
The following equation is obtained after substitution of equations (2), (3), (4) and (5) in equation (6):

$$\frac{q_{cr}}{D \sqrt{\Delta g D}} = 5.75 \sqrt{\Psi_{cr}}\ (\frac{h_o}{D} + b) \log 6\ (\frac{h_o}{D} + a) \quad (7)$$

It appears in many cases that h_o, on the crest of the dam, is not known precisely, and that more exact information is available about the downstream waterdepth h_b, related to the dam crest. The following approximation was made in this study:

$$h_o \approx h_b \tag{8}$$

This equality will obviously diverge as h_o reduces to small values, however it appeared during the study, that equation (8) can be substituted in equation (7) in a large variety of cases, yielding:

$$\frac{q_{cr}}{D\sqrt{\Delta g D}} = 5.75\sqrt{\Psi_{cr}}\left(\frac{h_b}{D} + b\right) \log 6 \left(\frac{h_b}{D} + a\right) \tag{9}$$

Equation (9) was derived for the stability of stones on the crest of the dam. It may however be modified to describe the stone stability on the downstream slope in the case of a very small downstream waterdepth ($h \ll d$) with overflow over a dam with an impervious core. In this case the flow is running down the slope and according to the observations of various investigators, damage can occur in the area where the equilibrium depth h_e (uniform flow) is reached.

Hence the waterdepth h_b in equation (9) may be replaced by the equilibrium depth h_e down the slope which can be deduced from the wellknown equation used in river engineering for uniform flow:

$$h_e = \left[\frac{q_{cr}^2 \ \text{cotg}\ \alpha}{\{18 \log 6 (\frac{he}{D} + a)\}^2}\right]^{1/3} \tag{10}$$

The calculation of q_{cr} from equations (9) and (10) involves a somewhat complicated double iterative process, but is feasible.

A second modification which has to be introduced when dealing with the stability of the top layer on the downstream slope, is the influence of the slope on the stone stability. The reduction of q_{cr} may be found using the wellknown formula:

$$\sqrt{\frac{\sin(\phi - \alpha)}{\sin \phi}} \tag{11}$$

where:

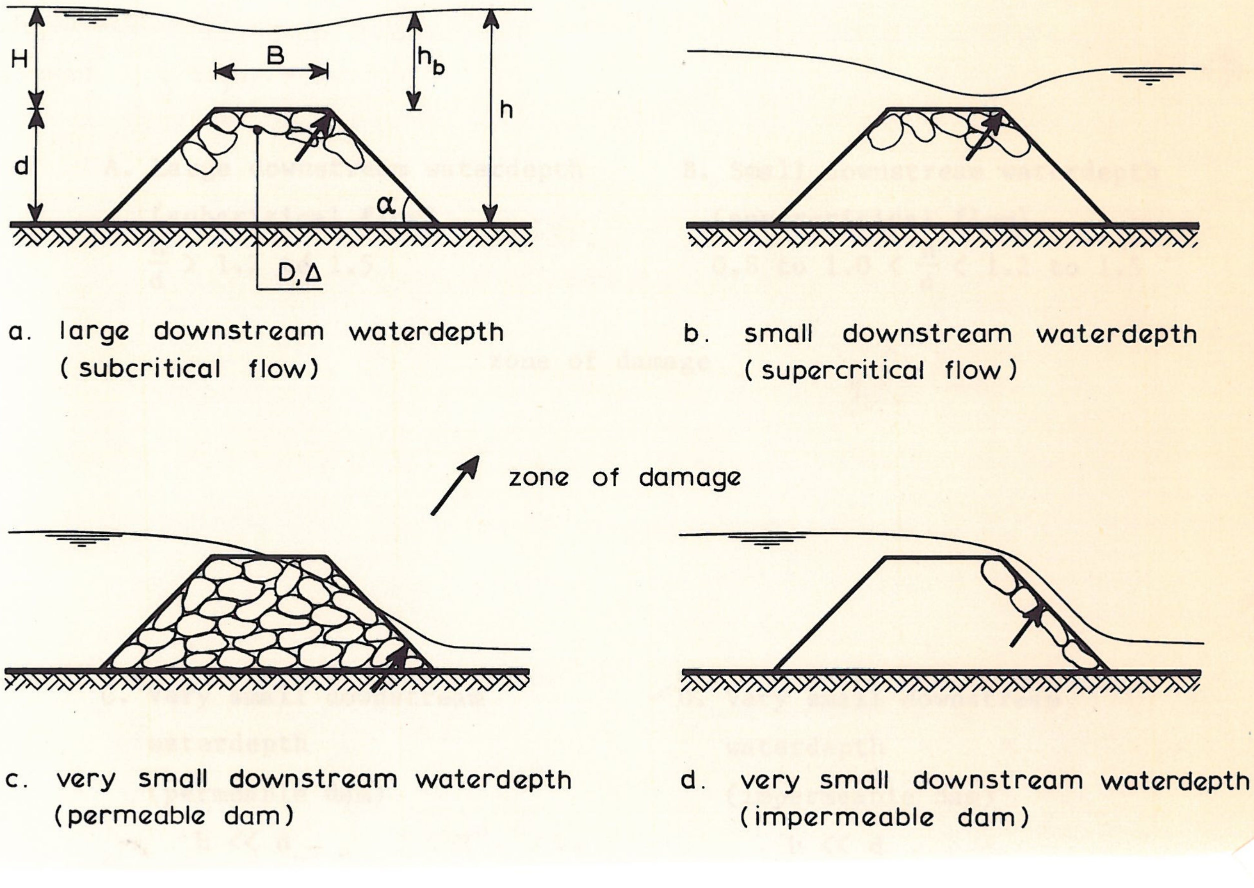
ϕ	= angle of repose of the stones	(rd)
α	= downstream slope	(rd)

It should be stressed that this procedure is only valid when the downstream waterlevel is very low, yielding a length down the slope sufficient for an equilibrium depth to be reached.

3. Critical discharges

3.1 Flow conditions

The following flow conditions may be distinguished:



figuur 2. Flow conditions

An initial distinction is to be made between cases A and B ($h > 0.8$ to $1.0\ d$) and cases C and D ($h \ll d$). The latter cases can be subdivided according to the permeability of the core of the dam.

A further distinction was made by Knauss[2] between gentle and steep slopes in cases C and D on the basis of air entrainment on steep slopes ($\cot g\ \alpha < 5$).

3.2 <u>Downstream waterlevel above the dam crest</u> (cases A and B)

Cases A and B in figure 2 show damage at the downstream crest and in the area immediately upstream of it.

The critical discharge parameter appears to be described fairly well by equation (9) in both cases.

An optimisation of the coefficients of equation (9) has been carried out, leading to a = 0 and b = 1, hence:

$$\frac{q_{cr}}{D\sqrt{\Delta g D}} = 5.75\sqrt{\Psi_{cr}}\left(\frac{h_b}{D} + 1\right)\log 6\,\frac{h_b}{D} \qquad (12)$$

with $\Psi_{cr} = 0.04$

Figure 3 below shows the above equation with the available experimental data.

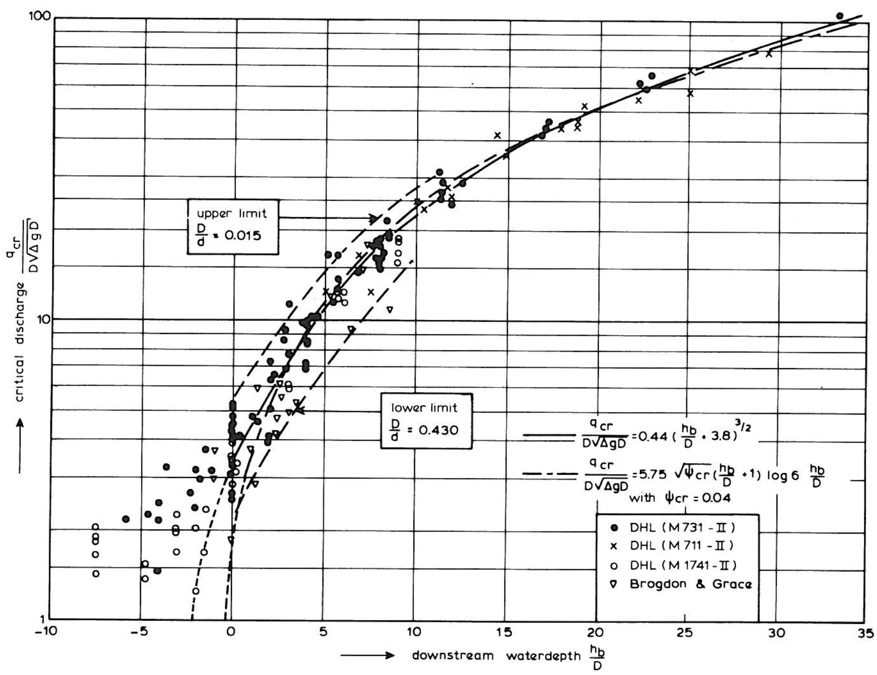


Figure 3. Critical discharge parameter with downstream waterlevel above dam crest.

It can be observed that equation (12) does not fit the experimental data very well for $3 < h_b / D < 0$.
Therefore another function was looked for and appeared to be even more simple, ie:

$$\frac{q_{cr}}{D\sqrt{\Delta g D}} = 0.44\left(\frac{h_b}{D} + 3.8\right)^{3/2} \tag{13}$$

This purely empirical equation fits the experimental data very nicely with a relative standard deviation of 17% for $h_b/D \geq 0$.

3.3 Very small downstream waterdepth (cases C and D)

Cases C and D showed damage on the downstream slope where the equilibrium depth is reached and the critical discharge parameter may be found from equations (9), (10),and (11) in which the coefficients $a = 0$ and $b = 1$ are to be substituted ($\Psi_{cr} = 0.04$ and $\phi = 40°$).
The result is shown in figure 4 together with the experimental data.

According to Knauss [2] a higher critical discharge is found with steep slopes (cotg $\alpha < 5$) due to air entrainment. His formula may be written as follows (dumped stones):

$$\frac{q_{cr}}{D\sqrt{\Delta g D}} = 2.5 - 3.2 \sin \alpha \tag{14}$$

For gentle slopes (cotg $\alpha > 5$), the results of both Linford & Saunders [3] and Lysne & Tvinnereim [4] may be described by the equation:

$$\frac{q_{cr}}{D\sqrt{\Delta g D}} = 0.2\,(\text{cotg}\ \alpha)^{7/6} \tag{15}$$

Equations (14) and (15) are given in figure 4 and show a fair agreement with the experimental data.
It should be noted finally that a comparison of dams with a permeable and an impermeable core made by Lindford & Saunders, shows a negligible influence of the dam permeability as far as the critical discharge parameter is concerned.

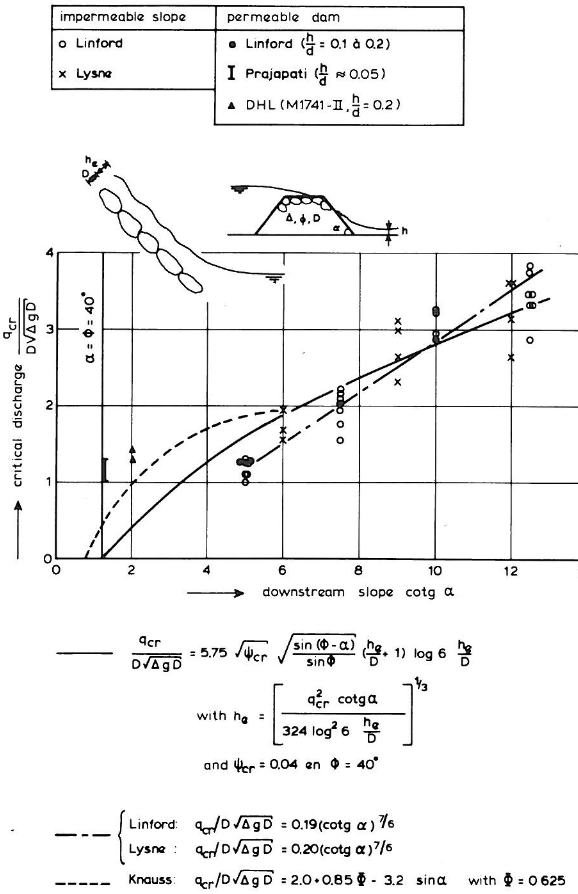


Figure 4. Critical discharge parameter with a very small downstream waterdepth.

It may be concluded from this and the previous section that a single theoretical approach enables a satisfactory description of the top layer stability of overflow rockfill dams to be made both for situations with high and very low downstream waterlevel. It appears however, that equations (13), (14) and (15), derived from different approaches, fit the experimental data in a better way for each specific situation.

3.4 Intermediate downstream waterdepth

Experimental data is unfortunately very scarce for this situation ($0.1\ d < h < 0.8\ d$). It may be expected however that both parameters involved in both previous sections, namely: the downstream waterdepth

(relevant for high waterlevels) and the downstream slope (relevant for very low waterlevels) will also have an influence in this case. This relationship has indeed been pointed out by Prajapati [5].
The following equation might be used as a provisional guideline:

$$\frac{q_{cr}}{D\sqrt{\Delta g D}} = 0.6\ (\text{cotg}\ \alpha)^{7/6}\ (\frac{h}{D})^{1/3} \qquad (16)$$

The relationship with h/D is confirmed by the experimental data (figure 5), but variations of Δ and α are too small to give a confirmation of equation (16) as a whole. This equation may however be used for the time being, since the relationship with cotg α has been confirmed in the previous section.

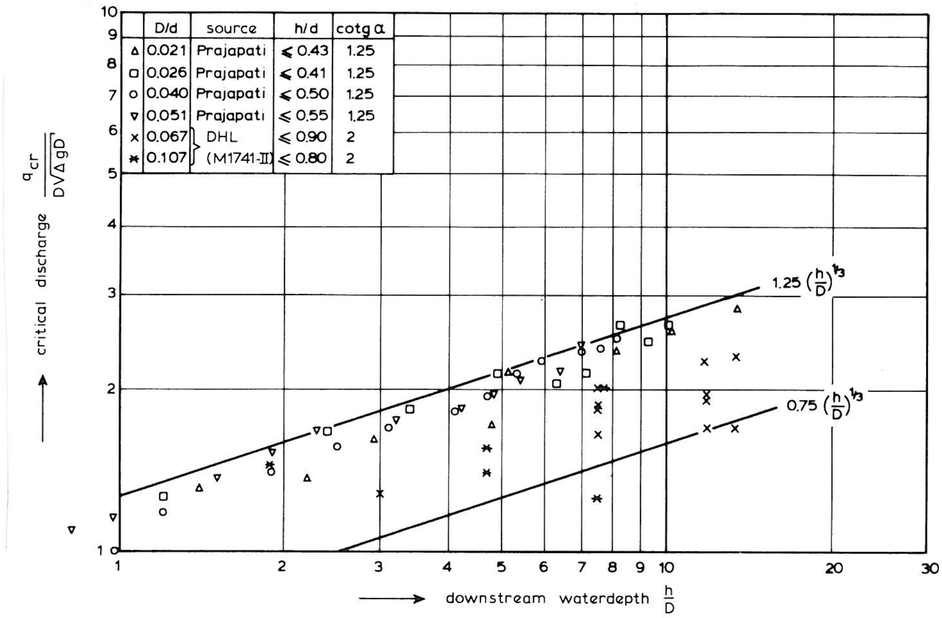


Figure 5. Critical discharge parameter with intermediate downstream waterdepth.

4. Critical upstream waterlevels

During the design of rockfill dams it is convenient to use the upstream waterlevel as a criterion instead of the discharge, since the former is generally better known in the design phase.
Hence a translation of the above-mentioned relationships from discharges into waterlevels is required.
The critical upstream waterdepth can be given by the parameter $H_{cr}/\Delta D$ and may be deduced from the critical discharge relationships using the wellknown discharge formulae for flow on spillways.

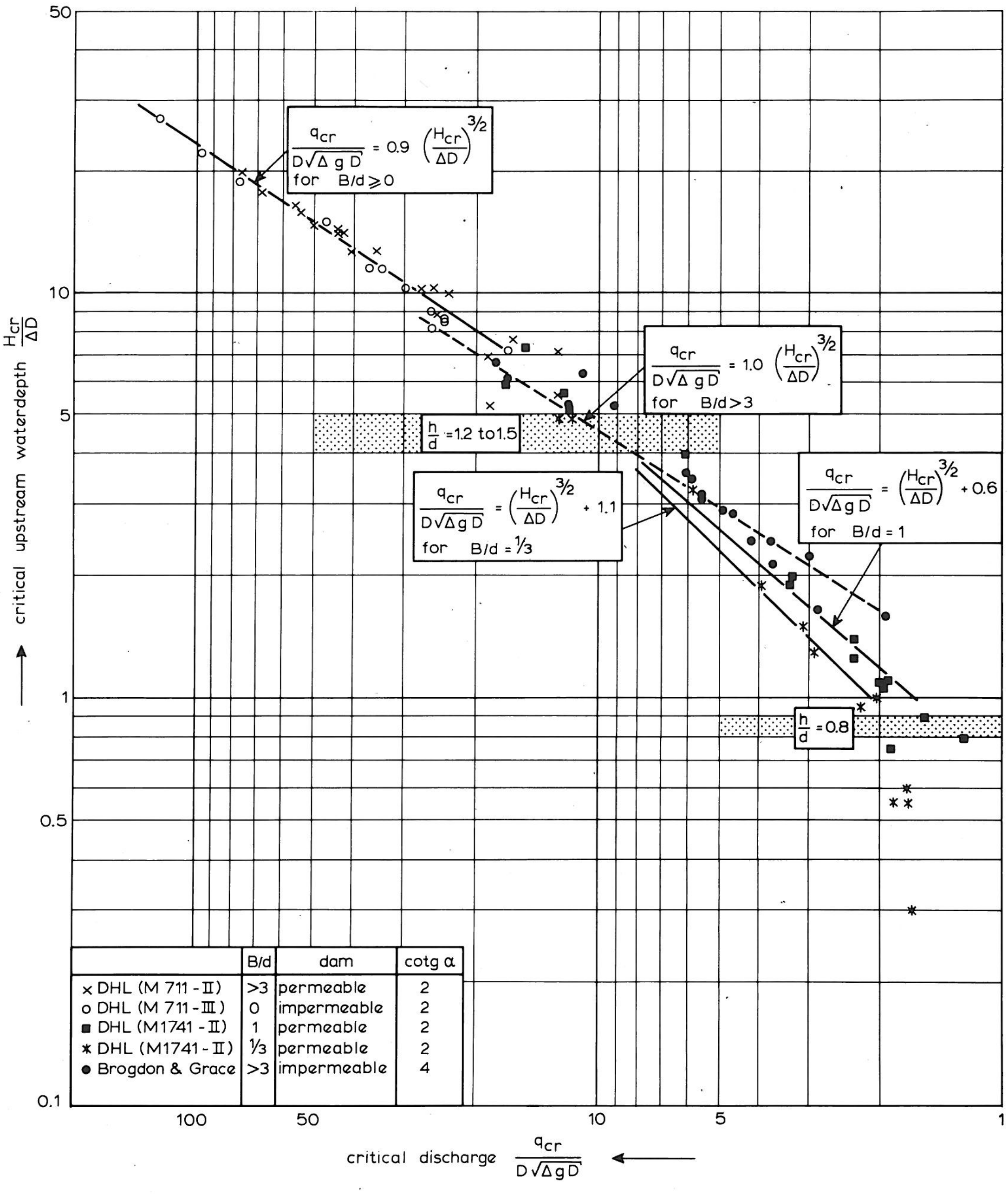


Figure 6. Critical waterlevel parameter as a function of critical discharge parameter.

The flow through the dam has an increasing influence when the downstream waterdepth is reduced. The permeability of the dam must therefore be introduced as an additional parameter. It appeared during the study that the relative width (B/d) can give a fair approximation of this influence of the permeability of the dam when the dam is made of uniform material (figure 6).

A better approximation might be found by introducing the permeability of the material into the equations as well, but this seems of secondary importance for the time being.

The translation of discharges into waterlevels for the case of a downstream waterlevel below the crest level of the dam ($h < 0.8$ to 1.0 d), that is the case of flow occuring completely through the (permeable) dam, is not yet clear.
Theoretical considerations indicate a relationship between the discharge and the square root of the head loss over the dam. This is confirmed by experimental data only when the head loss is smaller than 0.3 d. When the head loss is larger, the experimental data of Prajapati [5] seem to lead to a much higher power of the head loss (2 to 3) which, as yet, cannot be explained.

5. Other aspects

5.1 Gradation of material

According to various investigations, the influence of the material gradation on the critical discharge is small. When graded material is used, the characteristic stone diameter (equation 1) should be computed with M_{60} to M_{65} according to Brogdon & Grace [1].

5.2 Stone shape

The equations mentioned above are valid for crushed stone. According to [3] and [4] a reduction of 20 to 35% should be applied to the critical discharges when rounded materials are concerned.
This quite important influence should also be kept in mind in scale models.

5.3 Stone placing

The equations mentioned above are valid for random-dumped stones. The influence of hand placing of stones is quite important, according to [3]. If the stones are placed on edge, a substantial increase of the critical discharge may be achieved (up to 100%), but if the stones are laid with their flat face parallel to the slope a reduction up to 40% is found.

It is interesting to mention the results of Smith [7] concerning hand placed wedge-shaped prefabricated concrete blocks (figure 7). The measured critical discharge appeared to be 10 times higher than for dumped stones with the same mass. This is explained by the flow curvature over the top of the blocks which causes a downward pressure and also by a low pressure pocket immediately downstream of the step which is transmitted to the underlayer of the blocks via vertical channels.

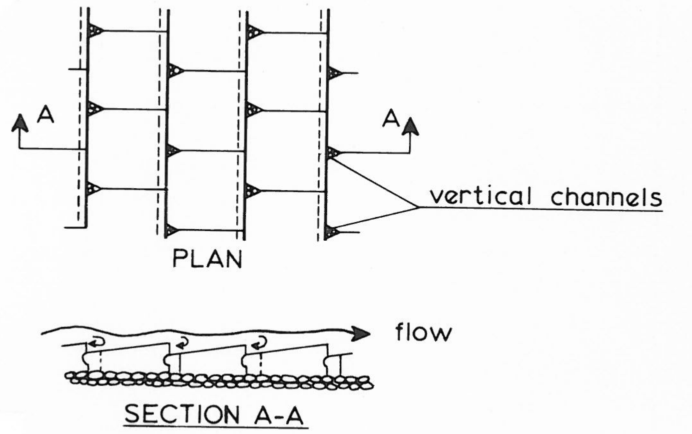


figure 7. Wedge-shaped blocks.

5.4 Layer thickness

Smith [7] reports an influence of the top layer thickness when "failure" discharges are concerned. The layer thickness can be seen as a storage of spare stones used for minor rearrangements of stones down the slope when the critical discharge is exceeded. According to the test results it can be concluded that the failure discharge can be 100% larger when the layer tickness is increased from one to three stone diameters. This positive influence levels off when the layer thickness exceeds 3 stone diameters.

Although the layer thickness is irrelevant for the critical (threshold) discharges, a thickness of 3 stone diameters may give some safety margin for the failure of the top layer.

5.5 Large stones

Most investigations have been limited to relatively small stones (D/d <1/3), which holds therefore also for the equations mentioned above.
The influence of large stones may be estimated by considering the extreme case D/d= 1. This is the case of a dam consisting of only one row of large stones. It can be seen in the literature that the critical velocity of a single stone on the bottom is about half the critical velocity of a stone of the same size located in a layer.
This means that the critical discharges should be reduced when dealing with large stones (D/d > 1/3).
The very scarce experimental data on this matter seem to suggest that an increasing reduction of the critical discharge parameter, from 0% to 50%, must be introduced when the parameter D/d is increased from 1/3 to 1/2 respectively.

5.6 Concrete cubes

It appears from the available experimental data that the critical discharge parameter of dumped prefabricated concrete cubes is only 2/3 of that of crushed stones with the same mass. This might be due to the smoother surface of the blocks.
It must be noted furthermore, that the influence on the critical discharge of large stone diameters mentioned in the previous section, is valid for concrete cubes as well. As a matter of fact the conclusions drawn in the previous section are based mainly on experimental data concerning concrete cubes.

5.7 Waves

The influence of waves propagating from the upstream side in the direction of the flow should be introduced by means of a correction to the critical upstream waterlevel (H_{cr}) rather than to the critical discharge. It can be deduced from the limited amount of data available concerning situations with a combination of waves and overflow, that the following equation holds for irregular waves:

$$\overline{H} = H_{cr} - (1/4 \text{ to } 1/2)\ H_s \tag{17}$$

in which:

H_{cr}	= critical upstream waterdepth related to the dam crest and valid for a situations with steady flow only	(m)
$\overline{H}$	= time averaged upstream waterdepth related to the dam crest	(m)
H_s	= significant wave height at the upstream side of the dam	(m)

The equation above has to be used with care for the time being, as it is based on investigations in which the parameters h/d and D/d were hardly varied.

The situation with waves only has of course been the subject of many investigations, but is beyond the scope of this contribution.

6. Conclusion

The study carried out by the Delft Hydraulics Laboratory has shown that a systematic comprehensive picture of a number of relevant aspects concerning the stability of overflow rockfill dams can be drawn based on a quite simple theoretical basis. The main difficulty being the distinction between really important aspects influencing the stability and the secondary aspects. Once this distinction is made, a classification of flow conditions can be made leading to a systematic approach of the design criteria.
Although this contribution may be of some help for the design of simple rockfill dams, a lot of systematic research still has to be performed in order to solve special problems. For the time being model investigations will be required in many cases to support practical design aspects.

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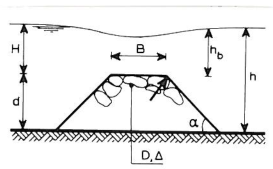
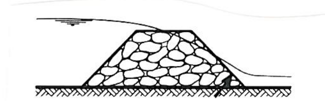
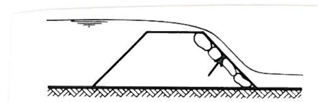
downstream waterlevel	flow			source	crit. disch. $q_{cr}/D\sqrt{\Delta g D}$	damage
HIGH $(h > 0.8\ d)$	subcritical $(h > 1.2$ to $1.5\ d)$			DHL Brogdon	$0.44[\frac{h_b}{D} - 3.8]^{3/2}$	
	supercritical $(0.8\ d < h < 1.2$ to $1.5\ d)$			DHL Brogdon	$0.015 < D/d < 0.350$	
LOW $(h < 0.8\ d)$	$h/d \approx 0$	$\cot g\alpha \geq 5$	impermeable	Lysne Linford	$0,2\ (\cot g\ \alpha)^{7/6}$	permeable 
			permeable	Linford		
		$\cot g\alpha < 5$	impermeable	Knauss	$2.5 - 3.2 \sin \alpha$	
			permeable	(DHL) Prajapati		
	$0 < h < 0.8d$	$\cot g\alpha \geq 5$	impermeable	(Lysne) (Linford)	$0.6\ (\cot g\ \alpha)^{7/6}\ (\frac{h}{D})^{1/3}$	impermeable 
			permeable	(Linford)		
		$\cot g\alpha < 5$	impermeable	-		
			permeable	DHL Prajapati		↗ location of damage
downstream waterlevel		downstr. slope	dam	source	crit. disch. $q_{cr}/D\sqrt{\Delta g D}$	damage