DELFT HYDRAULICS LABORATORY

AND

RIJKSWATERSTAAT

PUBLICATION No. 73

SOME ASPECTS OF FLOW THROUGH AND UNDER HYDRAULIC STRUCTURES USED FOR CLOSING ESTUARIES

BY

J.J. VINJÉ and F. SPAARGAREN

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SERIES 3 : MODEL STUDIES OF HYDRAULIC WORKS

Group 36 : Experiments on hydraulic machines and closed conduits
Section 36.64: Spillways and penstocks combined in one structure

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SOME ASPECTS OF FLOW THROUGH AND UNDER HYDRAULIC STRUCTURES USED FOR CLOSING OF ESTUARIES

by

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Summary

For closing of estuaries in the Netherlands both sluice-caissons and closure-dams have to be based on a sandy bottom.
For stability reasons it is necessary to construct a bottomprotection consisting of fascine-mattresses or composing of different layers of rubble (filter-bed).
Some considerations are given of the flow through and under the structures mentioned above.

Sommaire

Pour fermer les estuaires dans les Pays-Bas on se sert de deux méthodes, c'est à dire les clôtures avec des caissons à pertuis et les clôtures avec exhaussement en partant du fond (clôture graduelle).
Les caissons et les digues de clôture doivent être fondés sur un fond sableux.
Pour des raisons de stabilité il est nécessaire de construire une protection de fond composée de fascinages ou des couches différentes de moellons. Dans cette communication quelques considérations sont données par rapport à l'écoulement à travers et au dessous des constructions mentionées.

1. Introduction

The design of an adequate bottomprotection is always an essential condition for the succes of a closing-operation of tidal channels in estuaries where the bed consists of fine sand.
Both for closures by means of caissons and by means of gradual heightening bottomprotections are necessary in order to avoid inadmissible erosions which would result in loss of stability of the structure [1] , [2] .
Bottomprotections should preferably be strong, flexible and sandtight.
The main types of bottomprotections are classical fascine-mattresses, graded protective filter-structures and asphalt-mats.

Fascine-mattresses are applied already during centuries. These structures were based on experience. The scientific research started only during the last decade which resulted in numerous improvements both for the structure (sandtightness) and way of execution.

Graded filter structures are applied for caisson-closure (for the construction of the sill), for gradual closing-methods as foundation for porous dams and as a bottomprotection downstream of discharge-sluices. This type of bottomprotection consists of a number of layers each of which composed of materials with different size. Grading, size of the material and thickness of the layer are criteria for a sandtight bottomprotection, permeable to water. The boundary layers of the different materials have to be stable for the headloss over the structure, so the finer material should not be washed away. Considerations and results of modeltests to determine design-criteria for these structures are given in chapter 3.

The asphalt-mat - a warm mixture of sand and bitumen - is a closed bottomprotection. This means that pressure due to groundwater against the bottom of the structure,have to be taken into account.
Result of research and some considerations for determination of the thickness of the asphalt-mat as a function of a number of variables are dealt with in chapter 4.

A special problem concerning flow under hydraulic structures arises for caissons placed on a rubble bed. To guarantee the stability of the caissons it is necessary to construct rubble dams on the sill against the walls of the caissons. The stability of these structures as a result of flow underneath the caissons is considered in chapter 5.

2. Flow-conditions and scale-laws

The relation between the discharge velocity (U) and the total head loss (ΔH) for flow through porous media can be represented by

$$\frac{Q}{A} = U = k.\left(\frac{\Delta H}{L}\right)^{\alpha} \qquad (1)$$

From theoretical considerations on groundwater-flow it proves that for small values of

$Re = \frac{Ud}{\nu}$ (< 10) the flow is laminar and $\alpha = 1$. (Darcy's law).

For higher values of Re α decreases and for fully turbulent flow $\alpha = 0.5$. According to [3] it is possible to characterize the permanent flow through porous media by means of

$$S = \frac{\Delta H}{L} = \frac{C}{md}\,\frac{U^2}{2g}\,\frac{1}{\varepsilon 5} \qquad (2)$$

where m = shape-factor of rubble
C = coefficient of resistance
ε = porosity
d = diameter of rubble
L = seepage length

Generally C = f(Re) but for turbulent flow through porous media C is constant.

The flow through the rubble dams against the walls of the caissons (see chapter 5 and figure 5 and 6) will be turbulent in prototype due to the relatively high total head over the structure and the size of the material. For a correct reproduction it will be clear that the flow in a model should be turbulent too, so $(Re)_{model} > 10^3$.
For this case the following scale-rules can be derived.

$$n_S = \frac{n_C}{n_m n_d} \cdot \frac{n_U{}^2}{n_{\varepsilon}5} \qquad (3)$$

$$n_C = \frac{C\ prot.}{C\ model} = \frac{0,5}{0,5} = 1 \qquad (4)$$

$$n_m = 1;\ n_\varepsilon = 1 \text{ (scale-rule for inertia)} \tag{5}$$

$$n_d = n_U^{\ 2} = n_l \quad \text{(Froude scale)} \tag{6}$$

The length-scale of the model depends on the total head-loss to be expected and the diameter of the rubble in prototype and can be determined with the above mentioned scale-rules.

In a similar way scale-rules can be derived if the flow is laminar or in the transition zone.

Equation (2) is also valid for the turbulent permanent flow in graded protective filters.

To determine the pressures along the upperside of an asphalt-mat by means of a hydraulic model (see chapter 4) it is necessary to reproduce the variations of the waterlevel in a correct way, which means (6) applies. Pressures along the lower side of the asphalt revetment due to groundwater flow are derived from a electric analogy. This approach is based on the analogy between the laws of Darcy and Ohm formulating respectively the flow through porous media and electric flow through conductors.

3. Graded protective filter-structures

The protective value of a layer of loose material depends a.o. on the following factors: material of the layer (grain-diameter, size-distribution, porosity), thickness of the layer, characteristics of the material to be protected, the flow velocity over the structure and groundwater flow in vertical direction through the filter-structure.

In case of a filter-structure consisting of a system of different layers material with an increasing diameter has to be chosen in the successive layer e.g. for a sill in a closure-gap. The sand-tightness of this structure is decisive for the quality, which means that the lower layer of the bed should be sand-tight. The criterion at the boundary of two layers is that this boundary can resist the pressure gradients in horizontal and vertical sense, so it is not allowed that the finer material will be washed away.

According to research in the U.S.A. concerning protective filters and the stability for flow perpendicular to the layers of the structure [4] it is required that the material of a layer should be more permeable than that of the underlying layer to prevent superpressures of the water. The thickness of the layer should be sufficient to guarantee a good distribution of the different sizes in the layers. Moreover the voids of the material should be sufficient small in order to prevent penetration of the base material. This implies a certain relation between the diameters of the material at both sides of the boundary. These criteria were derived for small sized materials ($<3''$). In hydraulic engineering however the pressure gradients are generally both in a horizontal and vertical way. The criteria for the relation of the diameters at both sides of the boundary according to [4] cannot be used without comment. For this reason it will be advisable to carry out modeltests.

For flow in porous media parallel to the layers of the filters many modeltests for various materials have been carried out in a flume (3×3 m^2) on scale 1 : 1 to determine the critical values of the pressure gradient. This was achieved by increasing the head-loss over the structure. The set-up for these modeltests is shown on figure 1. The critical values for the pressure gradients are plotted in figure 2 as a function of the relation between the diameter of the filter and the base material. Dependent on this relation the critical gradient varies from 0.06 to 0.30 for the given materials.
Clear distinction should be made between high Reynolds numbers (turbulent flow) valid for gravel under heavy rubble (discharge velocities 0.10 to 0.25 m/sec) and low Reynolds numbers (laminar flow) as occurs for sand under gravel with discharge velocities in the gravel varying from 0.01 to 0.06 m/sec (see fig. 2).

Besides the flow through the filter-structure as described above also flow over a filter - so included turbulence giving pressure variation in the filter - has to be considered.
Various flume-tests with a two-dimensional flow, using sand (d_{50} = 140 μ), covered respectively with gravel with a diameter d_{50} varying from 0.015 m to 0.085 m have shown that a good bottomprotection can be obtained with a layer of loose material. The thickness must be at least three till four times the average grainsize. In this case the stability of the gravel endangered by flow will be less than that of the sand.
Generally one protective layer however will be insufficient due to the stability of the layer under the influence of flow.

For this reason the upper layer of a filter-structure often composes of heavy rubble. The thickness of this layer is for practical reasons not more than a few times the mean diameter. The risk will then be present that the material of the underlying layer will be washed away before the stability of the upper layer is reached, especially when vortex-streets are present (three dimensional flow).
In table 1 two types of filter-structures have been summarized which have been investigated in modeltests 1 : 10. Flow velocities exceeded 5 m/sec and strong vortex-streets occurred.

TYPE I		TYPE II	
	thickness of layer		thickness of layer
rubble 10/300 kg	0.80 m	rubble 300/1000 kg	1.20 m
gravel 0.03-0.08 m	0.35 m	rubble 10 - 40 kg	0.50 m
gravel 0.003-0.008 m	0.20 m	gravel 0.03-0.08 m	0.35 m
sand - gravel	0.15 m	gravel 0.003-0.008 m	0.20 m
sand (d = 150 μ)		sand gravel	0.15 m
		sand (d = 150 μ)	

For both types it proved that a thickness of the upper layer of 1.5 times the mean diameter would be sufficient to avoid instability at the boundary of the underlying layer. In that case the stability is determined by the resistance of the rubble against flow over the structure.

4. Asphalt bottomprotections

The difference from a hydraulic point of view between asphalt bottomprotections and graded filter-structures and fascine mattresses is the impermeability of the asphalt structures.
If a closure-dike will be constructed on an asphalt-mat (gradual closure) or a caisson will be placed on an asphalt bottomprotection accelerations of the flow occur whilst downstream inertia forces are present, resulting in water-level variations and change of pressure along the bottom at the upperside of the asphalt.

The groundwaterflow and consequently the pressure under the closed asphalt-mat are determined by the boundary conditions of the bottomprotection.
The structure will be lifted from the bottom if the pressure under the revetment are higher than those above the protection increased by the weight of the asphalt-mat. In that case the stability will be insufficient especially if the structure is situated on a slope.
To obtain insight in the value of the pressure modeltests have been carried out in a hydraulic model to determine the change of pressure along the upperside of the asphalt-mat. In an electric analogy the pressure under the mat have been measured using the data obtained from the hydraulic model. Waterdepth, geometry and permeability of the dam, head-loss over the dam, the length of the asphalt-mat at both sides of the dam and the thickness of the permeable sand layer under the structure have been varied during a systematic research.
Some results finally giving the thickness of the asphalt revetment are given in figures 3 and 4.
Test-results revealed that the magnitude of the superpressures is mainly determined by the head-loss, the length of the mat and in small extent to the waterdepth and the geometry of the dam.

5. Stability of structures on a sill to avoid instability of placed caissons

Caissons used for closures of tidal channels in estuaries are generally placed on a sill, constructed as a graded filter-structure. The upper layer of the sill generally consists of rubble in connection with the stability of the sill for the flow over it. The sill is not always completely level due to the sizes of the stones. If caissons are placed on the sill gaps under the caisson will be present with a height in the order of magnitude of the diameter of the rubble.
High velocities can occur in the gaps as a result of the head-loss over the caissons during the closing operations. This can endanger the stability of the rubble on top of the sill and thus the structure as a whole. Therefore it is necessary to dump stones against the walls of the caissons. If these structures consist of rubble dams the stability of this dam - especially the downstream one - must be assured. For this situation see figure 5.

Distinction should be made between the stability of the rubble dam as a whole and the stability of the individual stones. Considering the equilibrium of an individual stone on a slope α the critical force F exerted by the filter flow perpendicular to the slope is determined by

$$F = W \cos \alpha \left(1 - \frac{\tan \alpha}{\tan \varphi}\right) \tag{7}$$

where φ = angle of repose

W = weight of the individual stone

The hydrodynamic force F can also be given by

$$F = \rho_w \, C . \frac{u^2}{2g} d^2 \tag{8}$$

Further $u = \frac{U}{\varepsilon}$ (9)

Combining (2), (7), (8) and (9) and for m = 1 the critical pressure gradient is

$$S_{cr} = \Delta \varepsilon^{-3} \cos \alpha \left(1 - \frac{\tan \alpha}{\tan \varphi}\right) \tag{10}$$

For the stability of the rubble dam as a whole, considering a unit volume, the slope of the hydraulic grade-line, S_{cr}, can in a similar way be derived.

$$S_{cr} = (1 - \varepsilon) \, \Delta \cos \alpha \left(1 - \frac{\tan \alpha}{\tan \varphi}\right) \tag{11}$$

Comparing (10) and (11) it can be stated that the stability of the rubble dam as a whole is significant, because for every value of ε: $1 - \varepsilon < \varepsilon^{-3}$.

For the determination of the volume of the rubble dam which must be dumped against the walls of the caissons - at one side or at both sides of the caissons - it is necessary to know the distribution of the total head-loss over the dam(s) and the gap under the caisson.
Model tests have been carried out by the Delft Hydraulics Laboratory in order to obtain insight in this matter. Variation of gap-height and number and shape of the rubble dam have been considered. The characteristics of the material for the dams were also varied like diameter, density and grading of the stones. During the tests the course of the water pressure along the surface of the sill has been measured, sothat the distribution of the head-loss could be determined (see fig. 6). Moreover the discharge was measured as a function of the head-loss, increasing the latter till the critical value at which the dam collapsed.

In this way the criterion for stability mentioned above could be verified which proved to be correct.
For turbulent flow the relation between head-loss and discharge is given by

$$\Delta H = c\ q^2 \tag{12}$$

in which c = constant (dependent on geometry and material).
Equation (12) is valid as long as the rubble dam was intact (see fig. 6).
The moment of collapse, preceded by deformation of the rubble dam can be determined by means of the relation between the discharge and the head-loss because a sudden increase in discharge is found at this very moment (see fig. 6).

It can be concluded that the stability of the rubble dam depends on the porosity, the density of the material and the head-loss. The part of the total head-loss over the downstream rubble-dam is strongly influenced by the presence of the upstream dam and the gap-height.
Influence of grading of the material is manifested in the porosity. No significant influence of diameter of the rubble could be determined.

6. References

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4 KARPOFF, K.F. - The use of laboratory tests to develop design criteria for protective filters. Proc. American Society for Testing Materials. Vol. 55, 1955.

7. Symbols

Q = discharge

A = cross sectional area of flow

U = discharge velocity = seepage velocity

u = flow velocity

k = factor of permeability

ΔH = total head-loss

L = seepage length

S = pressure gradient

C = coefficient of resistance

m = shape factor of rubble

ϵ = porosity

g = acceleration of gravity

d = diameter of material

Re = Reynolds number

ν = viscosity

W = weight of stone

φ = angle of repose

α = angle of inclination

$\Delta = \frac{\rho_m - \rho_w}{\rho}$ = relative density

ρ_m = density of material

ρ_w = density of water.

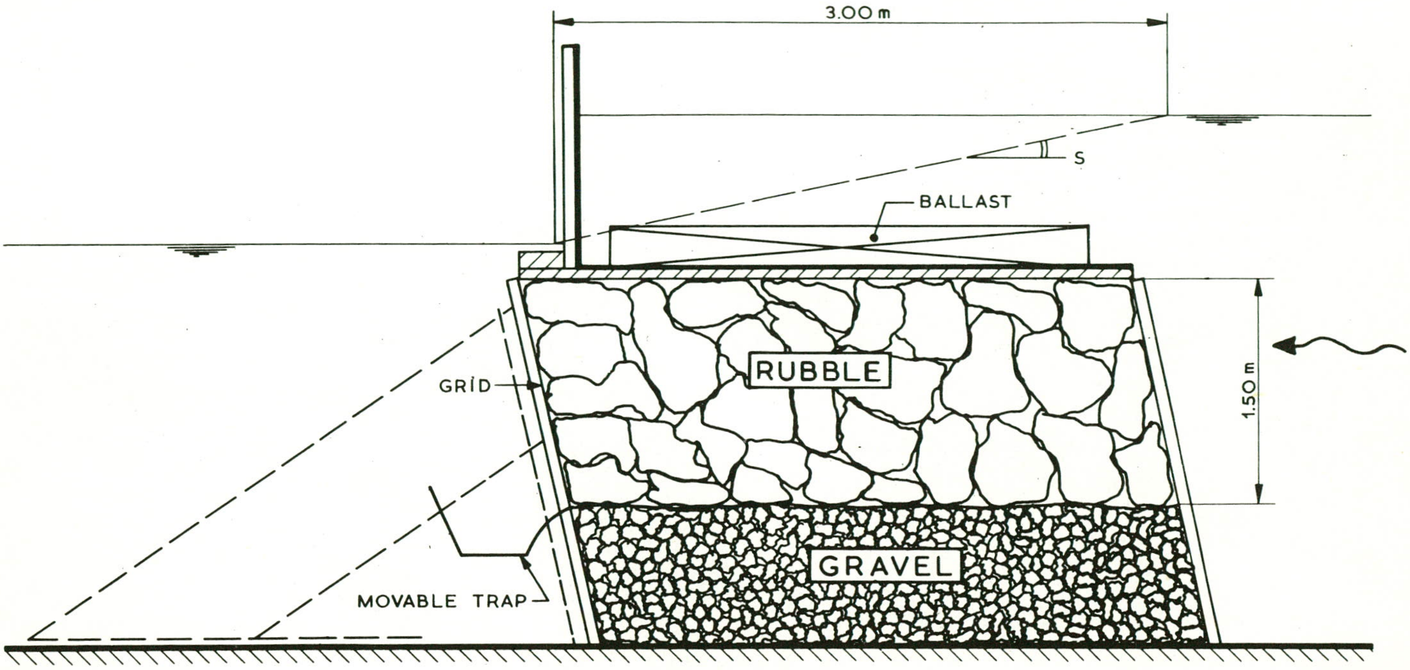


FIGURE 1

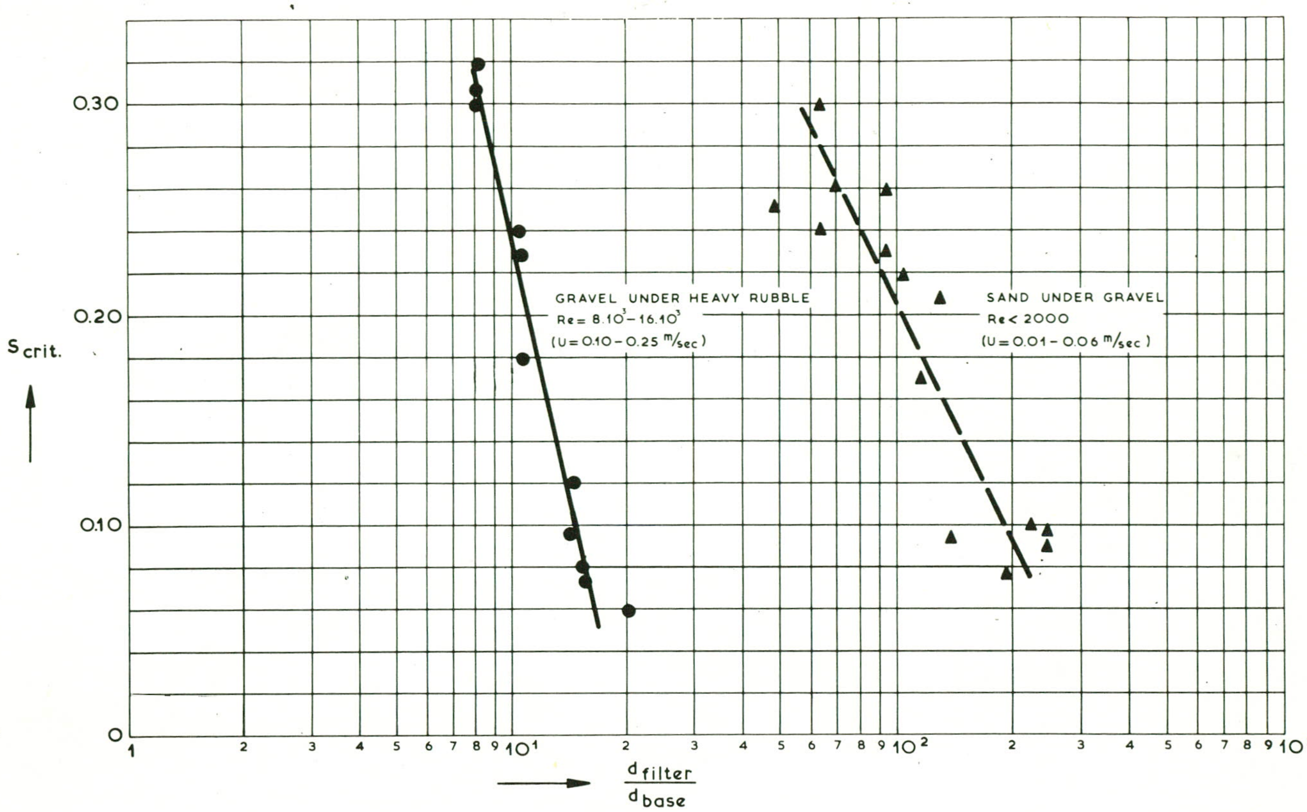

GRAVEL UNDER HEAVY RUBBLE
Re= 8.10³–16.10³
(U=0.10–0.25 m/sec)
SAND UNDER GRAVEL
Re< 2000
(U=0.01–0.06 m/sec)
S crit.
0.30
0.20
0.10
0
1
2
3
4
5
6
7
8
9
10¹
10²
10³
d filter
d base

FIGURE 2

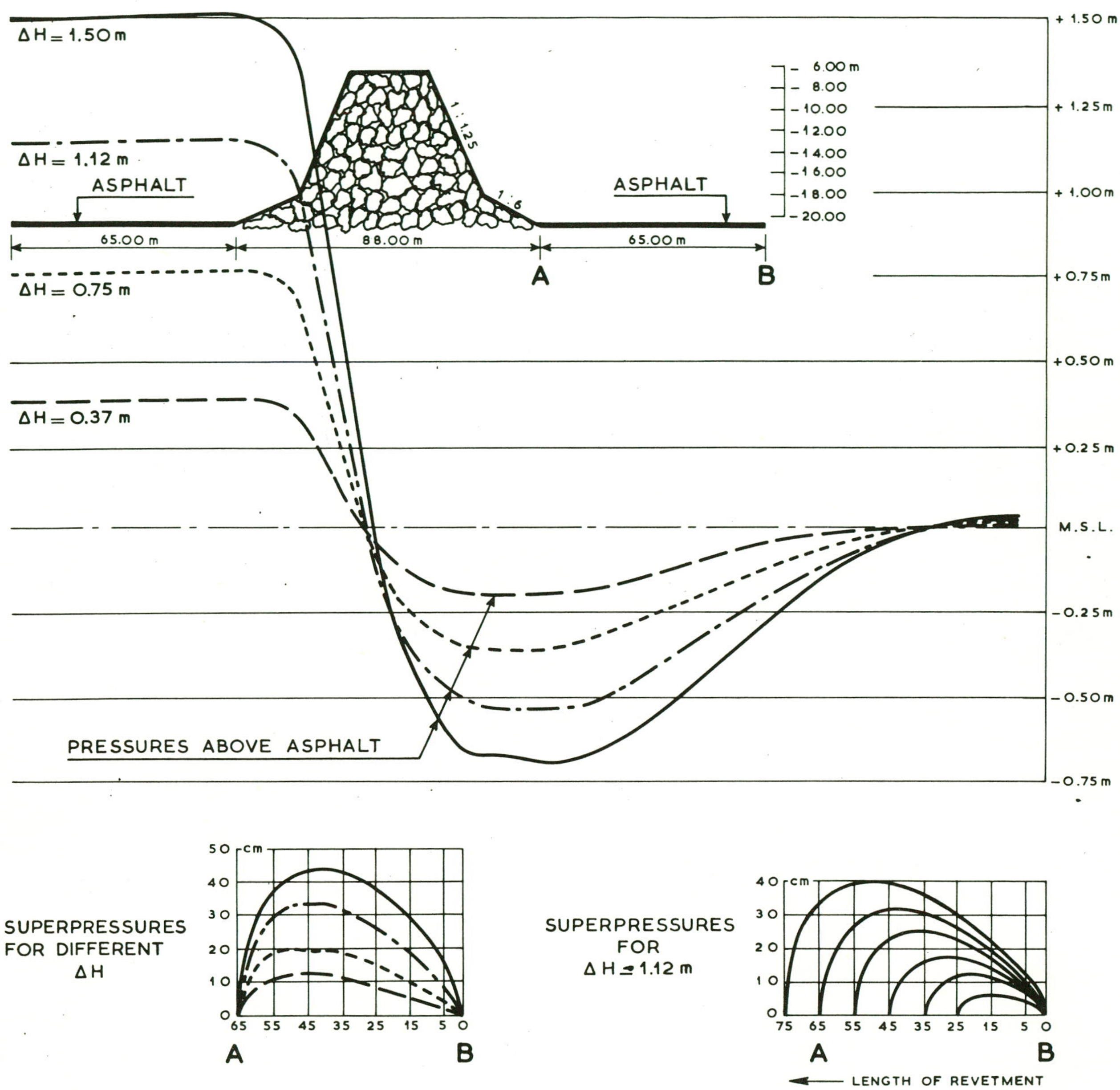
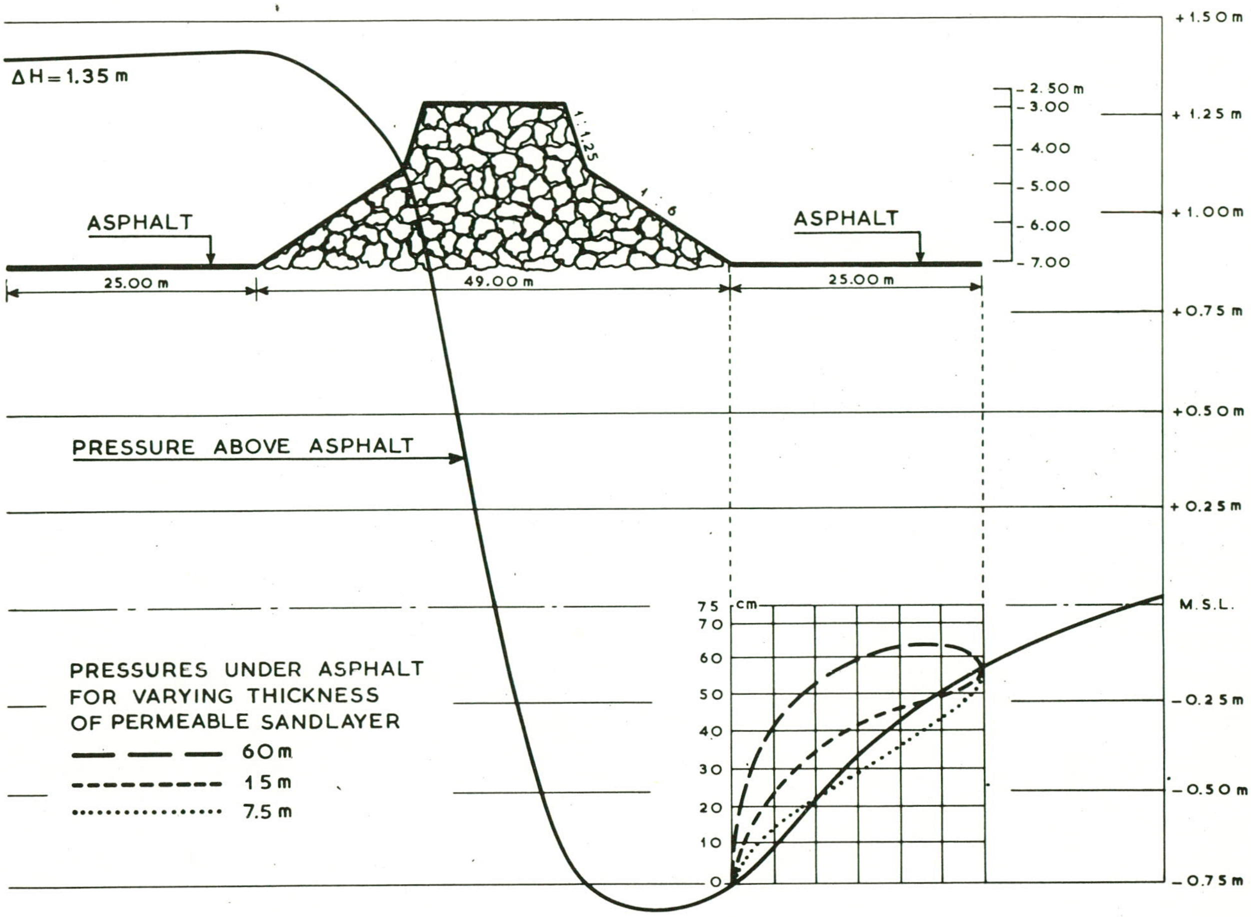

ΔH = 1.50 m
ΔH = 1.12 m
ΔH = 0.75 m
ΔH = 0.37 m
ASPHALT
ASPHALT
1 : 1.25
1 : 6
65.00 m
88.00 m
65.00 m
A
B
- 6.00 m
- 8.00
- 10.00
- 12.00
- 14.00
- 16.00
- 18.00
- 20.00
+ 1.50 m
+ 1.25 m
+ 1.00 m
+ 0.75 m
+ 0.50 m
+ 0.25 m
M.S.L.
- 0.25 m
- 0.50 m
- 0.75 m
PRESSURES ABOVE ASPHALT
SUPERPRESSURES FOR DIFFERENT ΔH
50 cm
40
30
20
10
0
65 55 45 35 25 15 5 0
A
B
SUPERPRESSURES FOR ΔH = 1.12 m
40 cm
30
20
10
0
75 65 55 45 35 25 15 5 0
A
B
LENGTH OF REVETMENT

FIGURE 3



ΔH = 1.35 m
ASPHALT
ASPHALT
1 : 1.25
1 : 6
25.00 m
49.00 m
25.00 m
- 2.50 m
- 3.00
- 4.00
- 5.00
- 6.00
- 7.00
+ 1.50 m
+ 1.25 m
+ 1.00 m
+ 0.75 m
+ 0.50 m
+ 0.25 m
M.S.L.
- 0.25 m
- 0.50 m
- 0.75 m
PRESSURE ABOVE ASPHALT
PRESSURES UNDER ASPHALT
FOR VARYING THICKNESS
OF PERMEABLE SANDLAYER
60 m
15 m
7.5 m
75 cm
70
60
50
40
30
20
10
0

FIGURE 4

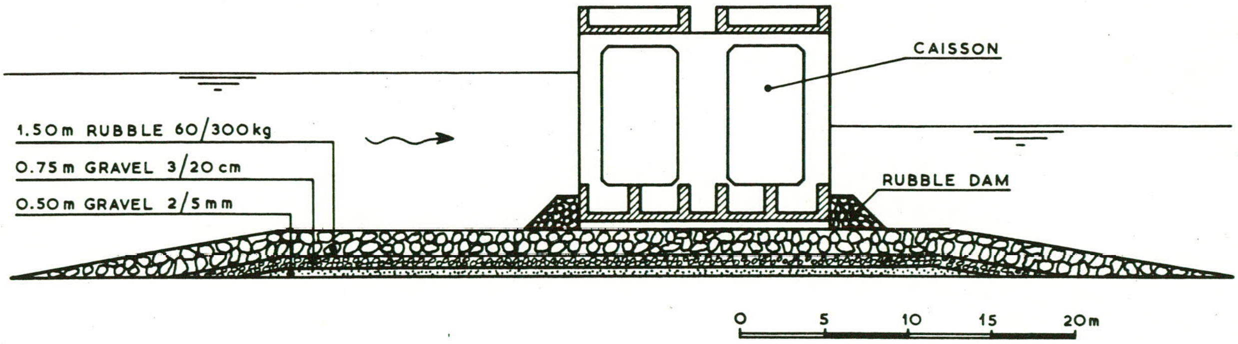
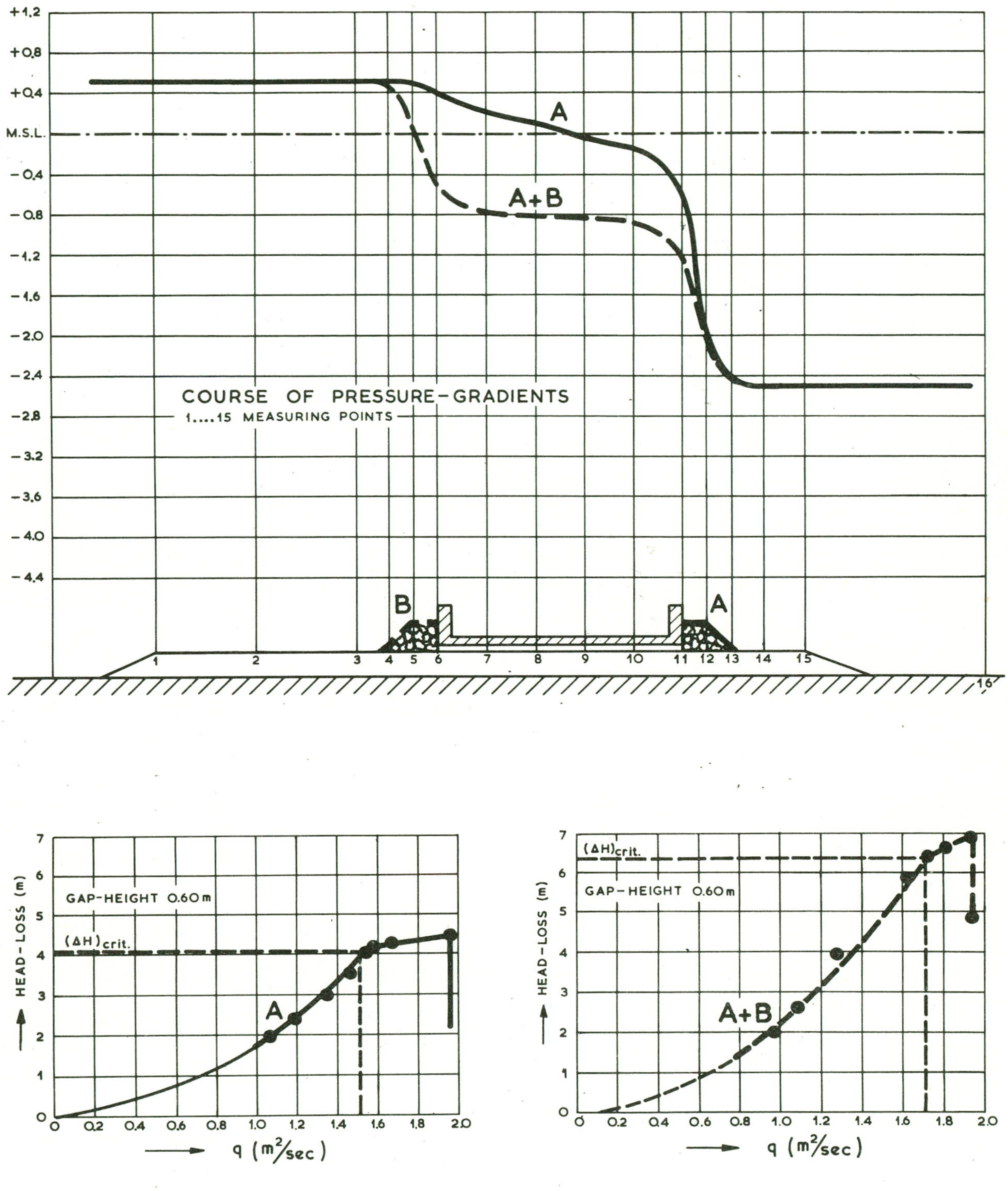

CAISSON
1.50m RUBBLE 60/300kg
0.75m GRAVEL 3/20cm
0.50m GRAVEL 2/5mm
RUBBLE DAM
0 5 10 15 20m

FIGURE 5



+1.2
+0.8
+0.4
M.S.L.
-0.4
-0.8
-1.2
-1.6
-2.0
-2.4
-2.8
-3.2
-3.6
-4.0
-4.4
A
A+B
COURSE OF PRESSURE-GRADIENTS
1....15 MEASURING POINTS
B
A
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
16
HEAD-LOSS (m)
GAP-HEIGHT 0.60 m
(ΔH)crit.
A
q (m²/sec)
0 0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 1.8 2.0
0 1 2 3 4 5 6 7
(ΔH)crit.
GAP-HEIGHT 0.60 m
HEAD-LOSS (m)
A+B
q (m²/sec)

FIGURE 6