CHAPTER 98

USE OF ASPHALT IN BREAKWATER CONSTRUCTION

by

K d'Angremond, Delft Hydraulics Laboratory
H J Th Span, Royal Road Building Cy
J van der Weide, Delft Hydraulics Laboratory
A J Waestenenk, Bitumarin

1 SUMMARY

Among the many types of breakwater constructions the so-called "rubble-mound" type is widely used For the construction of exposed rubble-mound breakwaters relatively large units are necessary to create a stable structure In many places in the world rock of the required size is not available at reasonable cost, which gave rise to the development of a great variety of armour units

Lately also the use of asphalt in breakwater construction has proved feasible
The experience gained during the construction of several projects in the Netherlands resulted in a special method of the use of stone-asphalt in breakwater construction Several cross-sections based on this concept were subjected to model tests to compare their behaviour under wave-attack with that of conventional cross-sections It appeared that the increase in stability can be expressed in terms of an "upgrading factor" Attention was also paid to wave run-up
Finally, examples of other applications will be presented which incorporate both practical experience and basic research

2 INTRODUCTION

The use of asphalt in road-building is well known, but in the past few decades the hydraulic uses of asphalt have also become common knowledge to civil engineers all over the world Among the many publications in this field, van Asbeck's encyclopaedic work (Reference 1) should be mentioned

An important contribution to the development of hydraulic applications of asphalt has been made by the Netherlands as a result of the interest shown in the subject by Rijkswaterstaat (State Traffic and Waterways Department) supported by investigations set up at the Delft Hydraulics Laboratory

In 1960 two major Dutch road-building firms joined their efforts in the field by establishing BITUMARIN, an affiliate company specializing in the development and use of bitumen in hydraulic engineering Close cooperation was established with the asphalt laboratory of the Royal Dutch/Shell Group Kerkhoven, one of their leading engineers, reported on the joint achievements reached together to the American Association of Asphalt Paving Technologists during its 1965 meeting at Philadelphia (2)

Now that the development of the various asphalt uses in hydraulic engineering has expanded it seems useful to outline specific developments like the use of asphalt in breakwater construction, which is the subject of this Paper

In chapter 3 a short historical review of asphalt techniques in breakwater construction is given, leading to a discussion of the pattern-grouting technique, which is believed to be most promising for the further development of asphalt uses in breakwater construction As the average hydraulic engineer will not be familiar with the latest developments of asphalt technology, this Chapter ends with a summary of recently developed theories pertaining to the grouting of stones with asphalt mixes

Chapter 4 is devoted to model investigations on the hydraulic properties of the constructions described in Chapter 3, introducing an "upgrading factor" for pattern-grouted slopes

In Chapter 5 the recent construction of the Separating Jetty at the Hoek of Holland is discussed, illustrating the various techniques mentioned in this Paper

3 DEVELOPMENT OF ASPHALT TECHNIQUES IN BREAKWATER CONSTRUCTION IN THE NETHERLANDS

3 1 Early marine uses

In the Netherlands the use of asphalt in sea defence works in the tidal zone started immediately after World War II Examples are the grouting with mastic-asphalt of groynes at the North Sea Coast between the Hook of Holland and The Hague, and of the breakwaters at the Hook of Holland The purpose of these repair works was to stabilize mounds and layers of discrete stones against heavy wave-attack by pouring hot mastic asphalt between the stones, thus keeping the stones in a fixed position

Asphaltic grouting proved to be very effective for two reasons

(i) after having cooled down to ambient temperatures mastic-asphalt behaves like a solid mass with high elasticity modulus under short loading times such as wave-attack, and
(ii) as a plastic material of very high viscosity under prolonged loading times, thus being able to follow subsoil settlements

In due time it was recognized by the Authorities that the asphaltic grouting technique was suitable to replace the traditional pitching of stones, and thus, when the Delta Plan came into execution, the asphaltic grouting technique was adopted as a standard method of protecting the slopes of the dikes in the tidal zone Examples of this use can be found in the cross-sections of the Veersegat Dam, the Grevelingen Dam, the Haringvliet Dam and the Brouwershavense Gat Dam (figure 1)

The first applications were "in the dry", even though in a tidal area Before long, however, methods were developed for use under water As a result of this development, the asphalt-ship "Jan Heijmans" was built, able to apply mastic-asphalt for grouting underwater sills or plainly for sea-bed protection in coastal inlets

3 2 IJmuiden breakwaters

The first important use of asphalt in "full size" breakwater construction can be found in the IJmuiden breakwaters (1963 - 1967) The old southern and northern breakwater had to be extended into deeper water by 2,100 and 1,200 m respectively, as a result of which the southern breakwater would project 3 km (above 2 miles) into the open sea

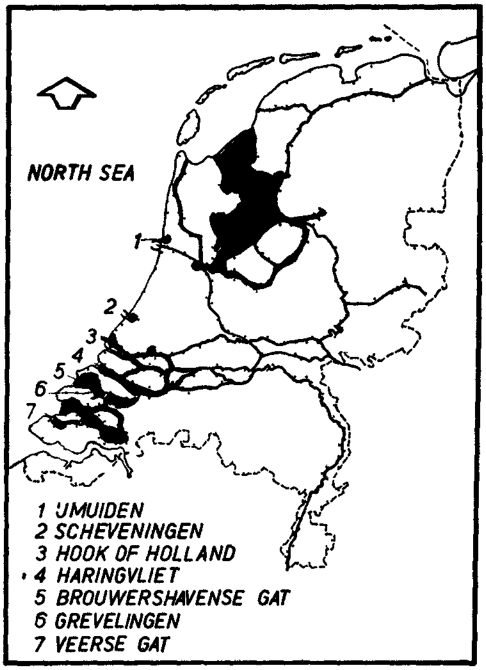

NORTH SEA
1
2
3
4
5
6
7
1 IJMUIDEN
2 SCHEVENINGEN
3 HOOK OF HOLLAND
4 HARINGVLIET
5 BROUWERSHAVENSE GAT
6 GREVELINGEN
7 VEERSE GAT

FIG 1

In principle the design of these breakwaters is of the "rubble-mound" type crowned with a prefab concrete crest-element The core is made of 300 - 1,000 kg stone to obtain a reasonable degree of core-stability during construction Nevertheless, frequent re-handling of the stone still appeared to be necessary because of the continuous bad weather conditions in the North Sea In fact, the 300 - 1,000 kg stones showed considerable lack of stability during construction phases above a level of M S L - 4 m

A substantial increase in core-stability was achieved by grouting the discrete stones with light stone-asphalt before receiving their final armour The grouting material was designed in such a way that only the upper two layers of the core were penetrated, and that no further "cold" flow into the core took place With mastic-asphalt, as was hitherto in use, such a controlled flow would have been impossible, but by the use in the mixture of stones up to 10 kg a kind of "clogging" effect was introduced, enabling control of flow of the grouting material

Instead of the conventional armour layer of discrete though more or less interlocking elements, an impervious monolithic layer of stone-asphalt was adopted The thickness of the layer was dictated by the internal water pressures caused by wave-action in the open rock-core against the impervious armour The thickness of the armour was chosen to be 2 25 m (Figure 3)

The construction of asphaltic layers of such a thickness at the steepest slope possible constituted a problem in itself By the time the breakwaters were designed the grouting techniques had not developed to the extent that controlled grouting of stone-layers of several thicknesses at water depths of 5 to 10 metres could be considered feasible

Therefore a premixed product had to be used Conventional asphaltic concrete with its aggregate-size limited to 6 - 8 centimeters lacked too much stability in the hot phase to enable the construction of thick layers as steep slopes The solution to this problem was the development of a new mixture with aggregate-sizes up to 60 kg, called stone-asphalt, which has already been mentioned in this Paper With this material slopes of 1 in 1 75 under water and 1 in 2 above water were found to be feasible

The experience gained with the IJmuiden breakwaters after three years' service is satisfactory in general, but nevertheless continuous creep of the stone-asphalt layer is causing cracks, especially in summertime (temperatures of both water and air) However, the damage is decreasing every year as a result of the formation of an internal skeleton in the stone-asphalt aggregate

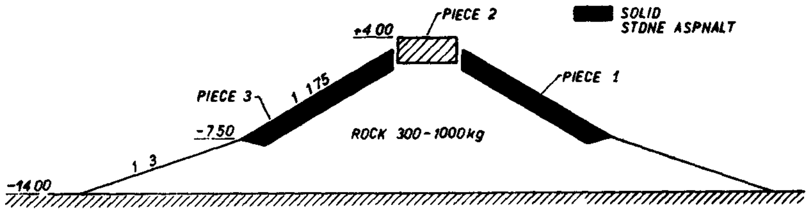
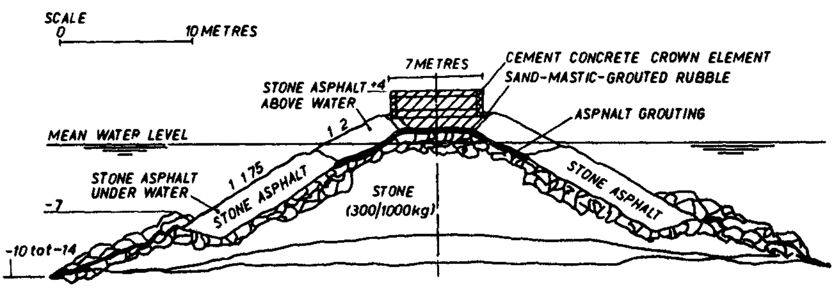


FIG 2



CROSS SECTION OF NEW BREAKWATERS UNDER CONSTRUCTION AT JMUIDEN
(FROM DATA SUPPLIED BY RIJKSWATERSTAAT)

FIG 3

As hos just been stoted, the IJmuiden works gove new impulses to the development of grouting techniques The grouting of the core-stone 300 - 1000 kg with light stone-ospholt hos olreody been mentioned os stondord procedure, but moreover o test-section wos corried out succesfully by grouting rock of 300 - 1000 kg with horizontol loyers of 1 5 to 2 m thickness constructing o monolithic ond stoble cop of heovy stone grouted mossively with light stone-ospholt

In considering the first importont use of ospholt in breokwoter construction embodied by the IJmuiden breokwoters, it con be stoted thot its performonce is sotisfoctory in generol, but thot for future works the following two drowbocks will hove to be deolt with

a The concept of on impervious loyer covering o highly permeoble rock-core should be obondoned to ovoid the effect of internol woter-pressures due to wove-oction
b Creep ond extended setting-time of thick ospholtic loyers should be ovoided by replocing the use of premixed loyers by the grouting of loyers of discrete stones, which hove olreody developed o skeleton of their own ond ore therefore no longer susceptible to setting

The experience goined in the IJmuiden works mode the solutions to these problems possible, os will be seen in the next porogroph

3 3 Pottern grouting

To ovoid the problem of internol woter pressures originoting from the impermeobility of the stone-ospholt ormour loyer o new concept wos introduced by the ideo of increosing the stobility of on olreody foirly stoble rock slope by locol grouting with stone-ospholt in o regulor pottern, thus mointoining the permeoble chorocter of the slope By using o grouting method the problem of creep ond extended setting-time would olso be coped with This system of "internol ormour" wos mode technicolly possible by the development of controlled grouting techniques during the IJmuiden works

In working out the ideo of pottern-grouting it wos reolized thot in filling up more thon obout 70 % the interstices between the rocks no guorontee could be given for the overoll permeobility of the construction Preliminory model tests executed in the Delft Hydroulics Loborotory showed on increose in stobility os o function of on increosing degree of filling up the interstices, with o relotively slow increose beyond 50 % So the filling up to 50 to 70 % of the interstices seemed to be optimol

For the colculotion of the required weights of ormour-elements, loyer thickness, etc reference is mode to Shore Protection, Plonning ond Design of the U S Corps of Engineers (Reference 7)

As to the size and spacing of the plots the following can be said An individual plot will penetrate to a depth of 2 d, d being one layer-thickness The shape depends on the local conditions (shape and direction of interstices between the stones) but it can be idealized to the slope of a cube with contents 8 d^3, of which approximately 60 % is solid rock and 40 % grouting material So the contents of one plot is 0,4 8d^3 = 3 2 d^3 and its weight

$$P = 3\ 2 \quad \gamma_g \quad d^3$$

The rock weight $W = \gamma_s \quad d^3$

Thus

$$P = 3\ 2 \quad \gamma_g/\gamma_s \quad W$$

in which P = weight of plot in tons
γ_g = spec weight of grout in tons/m^3
γ_s = spec weight of rock in tons/m^3
W = weight of rock in tons

The spacing of the plots should be such that 50 to 70 % of the surface is covered

If placed in such a pattern the plots (each fixating 5 to 10 stones) will touch each other at the edges This leaves stones uncovered at some places, which, however, are "keyed" between the others For reasons of safety it is recommended to use three layers instead of two, only grouting the top two layers A few loose stones will probably be washed away by the waves, which is not dangerous at all, but even if a whole plot were washed away for one reason or another, a third layer would still provide protection to the core, because it would be "keyed" to the surrounding plots

As to the stability under wave-attack, a so-called "factor of upgrading" F Could be attributed to the pattern-grouted system This means that pattern-grouted rubble in the X-ton-class has the same stability as discrete rubble in the F X-ton class

From preliminary model tests a value of F = 5 seemed a conservative estimate, which has been confirmed by the more elaborate tests discussed in the next Chapter This reduction of the required maximum stone-size has a favourable effect on the area of the cross-section of the breakwater because of the reduction in layer-thickness and the absence of secondary layers in most cases

Ease of construction is obtained by the reduction of both crane-reach and crane-load, or by the possibility of working on more gentle slopes

It seems that the time-proven asphalt grouting-technique has grown into a real competitor in rubble-mound breakwater design With increasing demands on stability under wave-attack, quarries are proving steadily unable to produce heavy armour stone, and this has caused the development of a series of artificial armour-blocks All these blocks have in common that increased stability can only be obtained by increased weight, which is necessarily accompanied by increased surface for wave-attack

As distinct from these external armours, the internal armour presented in this paragraph has the advantage of diminishing the wave-attack on the discrete armour elements by partly filling up the interstices between them, while on the other hand their stability is increased as a result of the "keying" effect of the grouting material

3 4 Asphalt mixtures for patch-grouting

The general principles for design and properties of asphaltic mixtures for hydraulic application, as developed in the Netherlands, are described by Kerkhoven (Reference 2)

For pattern grouting of the armour layer of large sized stones, some additional principles are necessary

The mix-design of the patches must be related to the large size of the stones and the shape and weight of the patches In this connection it is important whether the grouting is executed under water or not, as in the first case the hot stage of the mixture is rather short and, consequently, also the time for settlement

Experiments have shown that for grouting with patches of limited size a relation exists as a blocking criterium between the small-sized stones in the armour layer, defined as d_{15} 1), and the large-sized stones in the asphalt mixture, defined as d_{85} 1) For underwater grouting the relation d_{15}/d_{85} = ca 10 was found and for above-water-grouting d_{15}/d_{85} = ca 5

The six design for small-sized stones, and sand inside the grouting material, depends on the working circumstances during execution and on the place of use in the total breakwater construction It is common that for patch-grouting under water level, an asphaltic mixture in grop-grading is adopted and for patch grouting above water level an asphaltic mixture in concrete-grading

The flow in the hot stage and the viscous creep in the cold stage, in relation to the size and slope of the armour large-sized stone layer, depend on the percentage and type of filler and bitumen in the grouting mixture

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1) d_{15} c q d_{85} are the equivalent diameters d of stone size-distribution, passing in percentage of weight for 15 c q 85 %

4 MODEL INVESTIGATIONS

4 1 Introduction

Since 1964 breakwaters with the use of asphalt have been the regular subject of model investigations in the Delft Hydraulics Laboratory In the beginning the knowledge of the material was insufficient to reproduce the properties of the material on model scale Therefore the "asphalt-design" for the IJmuiden breakwaters was originally not tested in a model When, already during the execution of the works, again discussion arose on the required thickness of the stone-asphalt, it was decided to start simplified tests In these tests the flexible structure was schematized applying rigid concrete mortar in the model, instead of stone-asphalt, which means that investigations into the mechanic and elastic behaviour of stone-asphalt were prevented
The cap of concrete and stone-asphalt was reproduced as 3 independant rigid and relatively strong pieces of concrete (compare Figures 2 and 3)

It was shown visually in these tests that the overall stability of the slope cap was insufficient due to water pressures under the cover layer To solve this problem the toe of the slope was loaded with rubble and concrete blocks to a level of - 4 m for the exposed sections

When, thereafter, Bitumarin proposed an application of stone-asphalt to prevent the uplift pressures by keeping the outer layer permeable, it was decided to carry out further model tests, by replacing asphalt by concrete grouting

From a comparison between cross-sections with and without patch-grouting it appeared that the stability number k_D increased considerably It must be noted, however, that the schematization of the tests was such that the elasticity and the strength of the patches were not to scale

4 2 Reproduction of asphalt

Since pattern grouting proved to be a feasible method of stabilizing rubble-mound structures, a number of additional tests have been carried out to study the behaviour of grouted structures in more detail

When pattern-grouting is used, the principle of the rubble-mound structure is maintained but the stability is increased considerably Due to the effect of the grout, the surface of the structure is smoothed, resulting in a larger amount of uprush and overtopping

Attention was paid, therefore, to the wave uprush and the stobility of the armour layer, under various conditions using different cross-sections

Due regard was paid to the proper reproduction of asphalt to obtain both geometrical and dynamic similarity between model and prototype

Geometrical similarity was obtained by using the appropriate grading of the armour stone and a low viscosity of the grout to arrive at a depth and width of penetrotion comparable to those found on the site in question

Dynamic similarity wos obtained by composing the grout in the model in such a woy that the density of the mix, os well os its stiffness ond strength, wos reproduced correctly

Asphaltic bitumen is characterized in terms of the penetration and the ring and ball softening point (see Reference 1) Starting from these dota, the stiffness modulus of the material, defined as the ratio between stress ond stroin

$$(S = \frac{\sigma}{\varepsilon}),$$

can be determined for a given temperoture and a given frequency of loading

When oggregotes are added, the dynamic properties of the mix chonge, the stiffness of the mix being o function of the stiffness modulus of the asphalt and the concentration by volume (C_V) of the mineral aggregate

In the octuol project o grouting mixture will be used which consists of (by weight) 94 % stone, sand and filler
6 % bitumen 80/100 pen

So the volume concentration of minerals in the mixture C_V = 0 80 to 0 82 and the density = 2300 kg/m^3 Two mixtures were tried out to give both the required density and stiffness

The compositions of the mixes used in the model were

Mixture No 1
(by weight) 40 % dune sond
40 % barium sulphate filler
20 % bitumen 80/100 pen

The volume concentration of minerals in the mix C_V = 0 55, the density = 2,300 kg/m^3

Mixture No 2
(by weight) 60 % barium sulphate filler
40 % bitumen 280/320 pen

The volume concentration of minerals in the mix is C_V = 0 25 and the density = 1,850 kg/m^3

The stiffness of the mixtures wos calculated with the help of the monographs mentioned in References 3 ond 4 The ombient temperature in nature is $T = 5 - 15^{\circ}$ C, in the model it was 2 5° C The results are plotted in Figure 4

It is seen thot mixture No I, which is on scole as the density is concerned, is too stiff, while mixture No II is too light ond also on the supple side os the stiffness-modulus is concerned The strength of the material both in noture ond in model is in the order of 1 to 5 10^6 N/m^2 (Reference 4), meaning thot the grout in the model is too strong

From the above it follows that the density ond the stiffness con be brought to scole (though not in one mix), while the tensile strength is alwoys too high in the model This means that in the event that the collapse of the ormour loyer would be caused by the cracking of the patches, the model would give too fovourable results

Therefore also some tests hove been run in which the patches were already artificially broken beforehond, in order to eliminote ony fovourable effect of tensile strength of the grouting mixture, thus exoggerating the effect in the opposite direction

For this purpose the pattern-grouted armour was frozen ond then deliberotely "demolished" by hitting with o bar, in such a way that the potches broke down internolly, only leaving a three-dimensional "chinese puzzle" In the latter tests a situation is represented in which only the "keying" effect of the grout con be called upon, while the patches themselves have lost all internol bond

To eliminote the odhesion between stone and aspholt, the stone in the model wos covered with lime before grouting This was done to represent the situation in nature, where due to the wet environment little or no odhesion of the grout to the stones can be expected

4 3 Stability of grouted slopes

The results of stobility tests for rubble-mound breakwoters are generolly expressed in terms of the dimensionless stability number

$$K_D = \frac{H^3}{W \Delta^3 \operatorname{cotg} a}$$

Vorious outhors hove determined volues for K_D in order to obtain a stoble structure

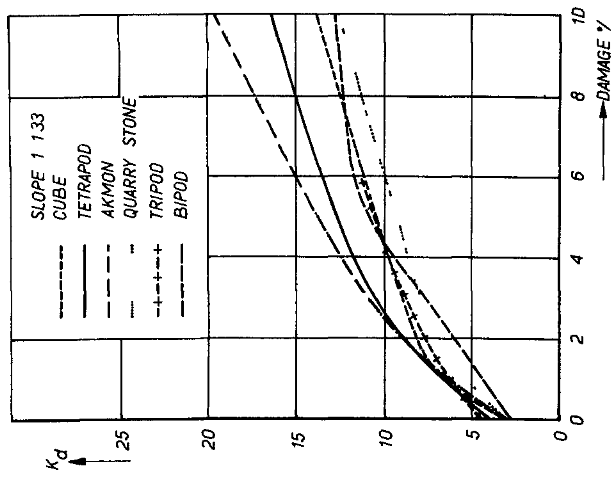
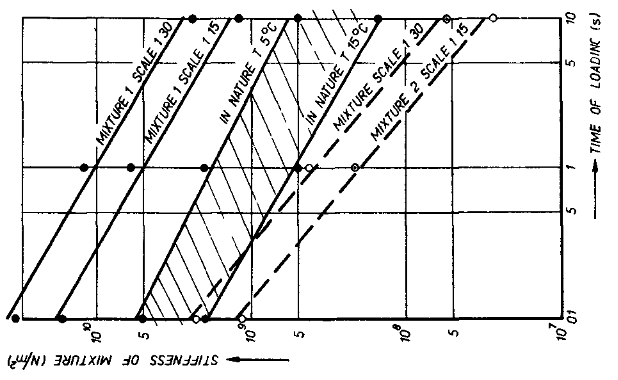


FIG 5



RELATION BETWEEN S_{mix} AND DURATION OF LOADING

FIG 4

A relation between the K_D value and the percentage of damage is given in Figure 5 (Reference 5)
Numerous references are present in literature describing artificial blocks which have a higher value of K_D for the no-damage criteria, due to the special interlocking effect The results of the model tests indicate that the critical value of K_D can be increased by a factor 2 to 3 in this way Pattern-grouting also increases the interlock between the various units, because of the three-dimensional effect, thus creating more or less irregular artificial units Moreover, the wave-attack is reduced as a result of the filled voids Obviously these aspects result in an increase in the stability of the structure which manifests itself in a higher K_D value
The upgrading-factor F is defined as the ratio

$$F = \frac{K_D \text{ for pattern grouted armour layer}}{K_D \text{ for non-grouted armour layer}}$$

4 4 Test results

4 4 1 Cement grouting

The first series of stability tests on grouted slopes was performed on a cross-section as indicated in Figure 6 As already mentioned, the cement grouting was used in these early tests

The patch-grouted test section was situated between - 8 5 ad - 3 m It has slopes of 1 2, 1 3 and consisted of rubble material from several weight classes during the various tests

The dimensions of the patches and the pattern of grouting was also varied under the following conditions

Slope	Stone Weight armour layer
1 3	1 - 6 ton
1 3	0 3 - 1 ton
1 3	0 5 - 3 ton
1 2	1 - 6 ton

The cross-section was subjected to regular waves with a period of 9 5 sec The wave height was increased from 3 to 8 m in steps of 1 m The duration of each step was 3 hours prototype After each step the damage was determined by counting the displaced stones and expressing this number in a percentage of the

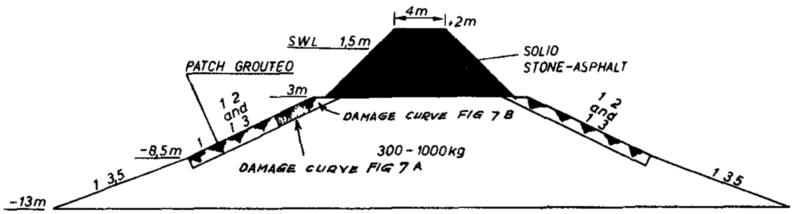


FIG 6

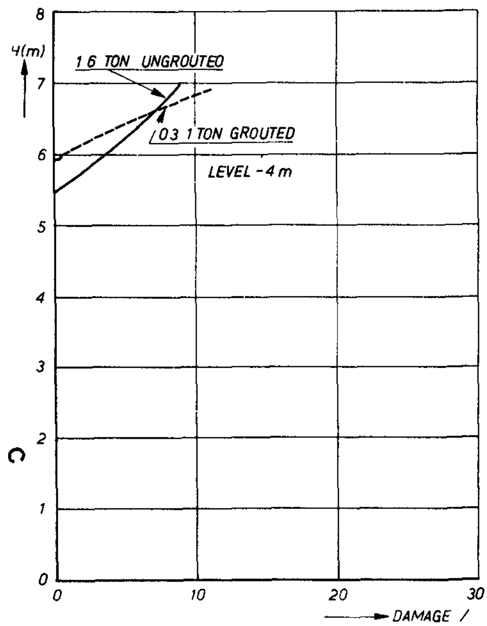


FIG 7 A

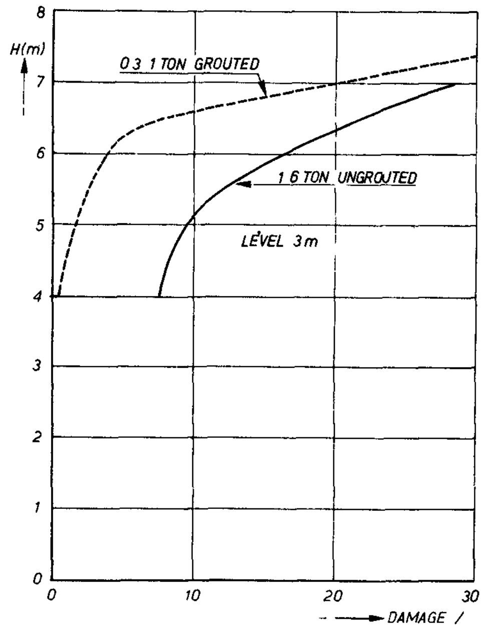


FIG 7B

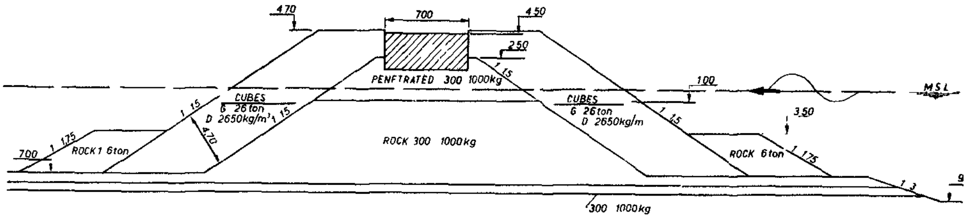


FIG 8A

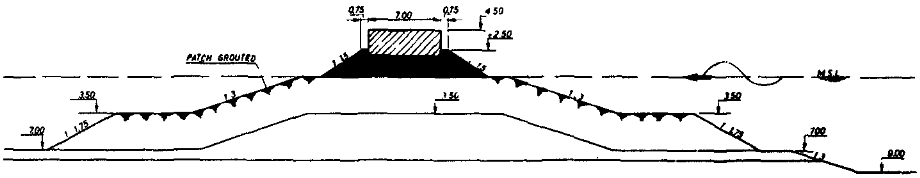


FIG 8B

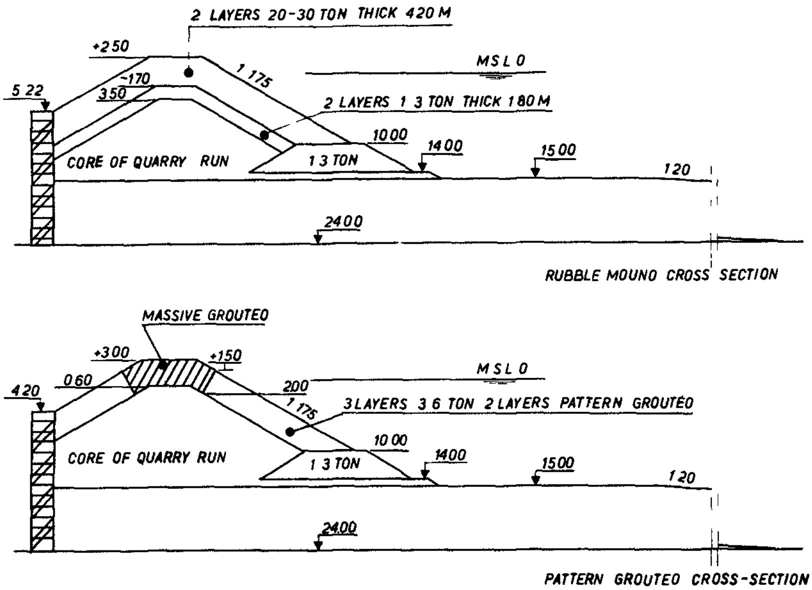


FIG 9

	Water-level - 0 5 H up to 5 m	Water-level + 0 5 H up to 5 5 m	Water-level + 2 5 H up to 6 m	Water-level + 3 5 H up to 7 m
Cross-section Fig 8A				
concrete blocks 26 ton	no damage	no damage	no damage	1 %
berm 1 - 6 ton	no damage	no damage	no damage	no damage
Cross-section Fig 8B				
grouted slope	no damage	no damage	no damage	no damage
berm 1 - 6 ton	no damage	no damage	no damage	moderate damage

Table 1 Results comparative tests with alternative design of Scheveningen breakwaters

tatal number of stones in the zone cancerned In this way it was possible ta establish a relationship between damage percentage and wave height

A comparison of the relatian between damage and wave height for grouted slapes with stones of 0 3 - 1 ton and non-grauted slopes consisting af 1 - 6 ton quarry (Figures 7A and 7B) shows that these situations are comparable for damage less than 10 % This leads ta the canclusion that far a practical range af damage percentage the upgrading factor can be assumed ta be about 5

This canclusion has been verified far an alternative design af the Scheveningen breakwater, where a cross-section in stone asphalt was campared with a method using cancrete blacks (Figures 8A and B) These tests were also performed in regular waves Data on the damage are presented in Table 1

In all conditions the grouted slope was equally as stable as the rubble-mound breakwater with cubes of 26 tans Only in the most severe conditions did the smaoth surface of the grouted slope increase the downrush in such a way that damage occurred to the non-grouted berms This draw-back could be remedied by making the grouting slightly deeper The stone-asphalt was also reproduced by a cement mortar in these tests

4 4 2 Asphalt grouting

Since grouting praved ta be a useful methad for stabilizing rubble-maund structures, some additianal tests were carried aut to study the stability of grouted rubble-mounds in waves Far that purpose a number of grauted sectians were campared with an equivalent number of traditional rubble-mound sectians consisting of stanes 5 times heavier than those used in the pattern-grauted section Using the formula of Hudson, the rubble-mound section was sa designed that no appreciable damage should accur under maximum wave attack

As stated in Sectian 4 2 special care was taken to reproduce both the geometrical and dynamical properties of the asphalt grout Since the strength af the material cauld not be reproduced carrectly, the tests were repeated with broken patches

Wave heights were increased step by step until maximum wave height were reached A review of cross-sections tested in the model is given in Figures 9 to 11, whilst a summary of test conditions and the test results is given in Table 2

Table 2 Test Conditions and Results for Various Grouted Sections

Profile	Fig	Scale of Test	Mixture	T (sec)	H_{max} (m)	Damage (%) [1]
A	9	30	I	10	8 5	NONE
B	10	30		10	8 5	NONE
B	10	30		8	6-7	NONE
C	11	30		8	6-7	NONE
B	10	30	II	10	7	NONE
B	10	30	II [2]	10	7	NONE

1) Domoge expressed as % of stone removed
2) Patches broken

Test results indicated that both the traditional and the grouted sections showed little or no damage, even when the patches were broken This proved the validity of an upgrading factor of 5

4 5 Run-up on grouted slopes

In many cases the crest level of a breakwater or a sea wall is determined on the basis of an acceptable amount of overtopping under extreme conditions, although sometimes the acceptable wave run-up is also used as a criterion

The level of wave run-up z above M S L has been investigated extensively for smooth impermeable slopes by several authors (See a o Reference 6, Reference 7 and Reference 8) An extract of these results is presented in Figures 12 and 13 for $d/H > 3$

For slopes covered with rip-rap and for rubble-mounds the run-up is much less because of the roughness of the surface and the porosity of the outer layers The reduction in run-up due to these effects is expressed by a reduction factor r, indicating the ratio between the run-up on the rough surface and the run-up on a smooth impermeable surface under the same conditions Though the scatter of the measured figures is considerable, authors from different origin indicate for r a value of 0 5 to 0 6 (References 9, 10, 11)

With a patch-grouted slope, porosity and roughness are reduced in comparison with rubble slopes Consequently the run-up must be expected to be greater This may lead to a higher crest level of patch grouted breakwaters which involves a higher cost

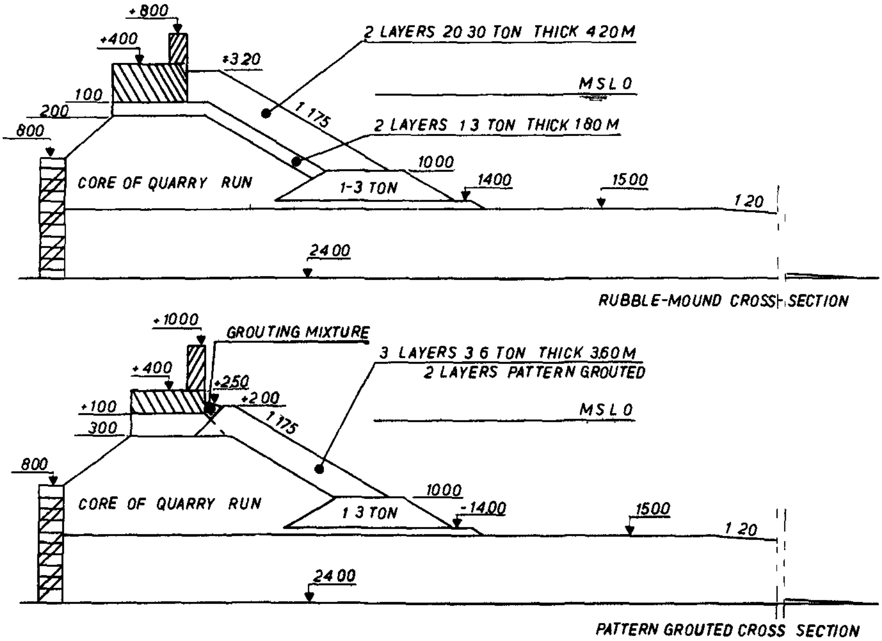

+800
+400
+3 20
2 LAYERS 20 30 TON THICK 4 20 M
M S L 0
100
200
800
1 1 75
2 LAYERS 1 3 TON THICK 1 80 M
10 00
CORE OF QUARRY RUN
1-3 TON
14 00
15 00
1 20
24 00
RUBBLE-MOUND CROSS SECTION
+10 00
GROUTING MIXTURE
3 LAYERS 3 6 TON THICK 3 60 M
2 LAYERS PATTERN GROUTED
+4 00
+2 50
+2 00
+1 00
M S L 0
3 00
1 1 75
8 00
CORE OF QUARRY RUN
10 00
1 3 TON
-14 00
15 00
1 20
24 00
PATTERN GROUTED CROSS SECTION

FIG 10

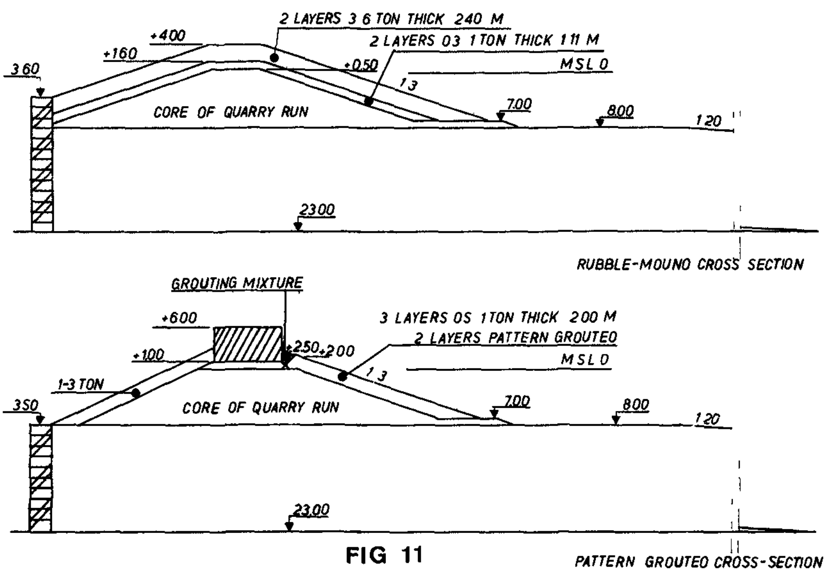

2 LAYERS 3 6 TON THICK 2 40 M
+4 00
2 LAYERS 0 3 1 TON THICK 1 11 M
+1 60
+0 50
M S L 0
3 60
1 3
CORE OF QUARRY RUN
7 00
8 00
1 20
23 00
RUBBLE-MOUND CROSS SECTION
GROUTING MIXTURE
+6 00
3 LAYERS 0 5 1 TON THICK 2 00 M
2 LAYERS PATTERN GROUTED
+2 50
+2 00
+1 00
M S L 0
1-3 TON
1 3
3 50
CORE OF QUARRY RUN
7 00
8 00
1 20
23 00
PATTERN GROUTED CROSS-SECTION

FIG 11

Model tests have been carried out on fully grouted slopes, eliminating porosity completely. Any reduction in run-up was due to surface roughness.

The run-up z depends on the following factors

$$z = f\,(H,\ L,\ T,\ d,\ \nu,\ \rho,\ g,\ k,\ p \text{ and } \alpha)$$

in which
- H = wave height
- L = wave length
- T = wave period
- d = water depth
- ν = kinematic viscosity
- ρ = density
- g = gravity
- k = roughness
- p = porosity
- cotg α = slope

Because k and p were kept constant during the tests (p = 0) the relative run-up can be expressed as

$$z/H = f(H/gT^2,\ d/gT^2,\ \text{cotg}\,\alpha)$$

H/gT^2 and d/gT^2 were varied from 0.0004 to 0.01 and from 0.009 to 0.095 respectively.

The slope cotg α was 1.75 and 2.25. The grouted quarry stones weighed from 100 - 250 kg (prototype). As a check also tests have been made on a smooth impermeable slope and on a non-grouted slope covered with quarry stone.

As the plot of z/H versus H/gT^2 showed a scatter which could not be explained by the differences of d/gT^2 only, the maximum values of z/H have been plotted as a function of H/gT^2, for both slopes separately (Figures 12 and 13).

To compare the actual model tests with the results of others, Figures 13 and 14 show also data obtained from References 7, 9 and 10.

From the Figures it can be concluded that reduction coefficients for the wave run-up can be used as indicated in Table 3.

<u>Table 3</u> Reduction Coefficients

Slope	r	Source
smooth impermeable	1.0	-
rip-rap covered	0.5 to 0.6	model tests + literature
rubble-mounds	0.5 to 0.6	model tests + literature
100 % grouted	0.6 to 0.8	model tests
patch grouted	0.6 to 0.7	interpolation

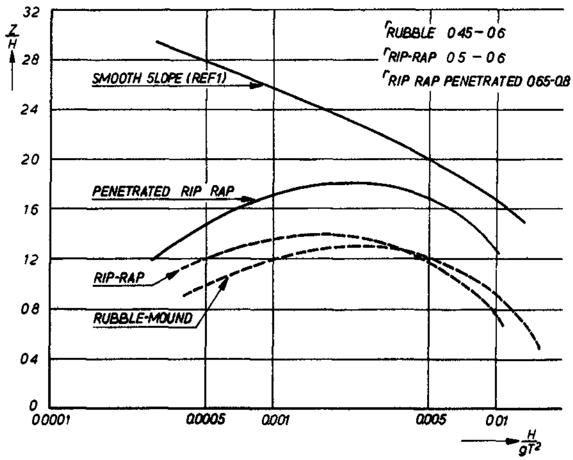

32
28
24
20
16
12
08
04
0
Z/H
RUBBLE 045–06
RIP-RAP 05 – 06
RIP RAP PENETRATED 065-08
SMOOTH SLOPE (REF1)
PENETRATED RIP RAP
RIP-RAP
RUBBLE-MOUND
00001
00005
0001
0005
001
H/gT²

FIG 12

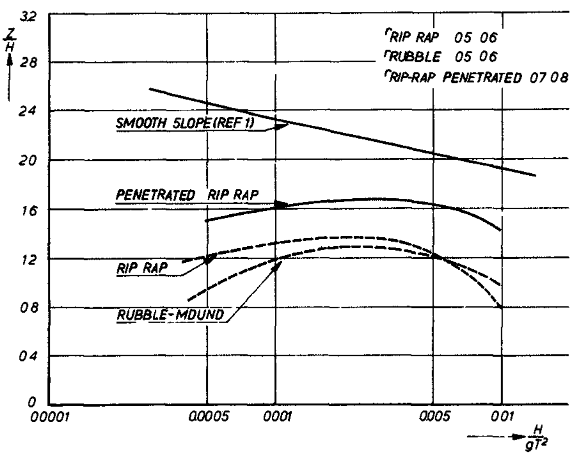

32
28
24
20
16
12
08
04
0
Z/H
RIP RAP 05 06
RUBBLE 05 06
RIP-RAP PENETRATED 07 08
SMOOTH SLOPE (REF 1)
PENETRATED RIP RAP
RIP RAP
RUBBLE-MDUND
00001
00005
0001
0005
001
H/gT²

FIG 13

5 SEPARATING JETTY HOOK OF HOLLAND

5 1 Design conditions

A recent example of the use of asphalt in breakwater constructian can be found in the so-called "Separating Jetty" at the Hook of Holland

This Separating Jetty (Figure 14) is situated in the new entrance being constructed far Rotterdam Harbour, separating bath traffic and the tidal density currents af the Ratterdam Waterway leading ta the inner harbaurs af Ratterdam, and the Caland Canal leading to the Eurapaart harbaurs far mammath tankers So its main objective is of a nautical nature, but by virtue of its exposed situatian perpendicular wave-attack up to H_{sign} = 6 m can be expected at the head, and oblique attack along the trunk

From the inner and going seaward the subsail is descending from above water down to approx M S L - 5 m Nevertheless it can hardly be regarded as a shallow water breakwater, due to the fact that the underwater banks will be dredged to 1 an 4 slapes gaing down beyand M S L - 20 m

5 2 Standard design

For the deeper part of the dam an embankment has been constructed cansisting of fascine mattresses pratecting the dam footing, and dumped stone and other waste materials from an ancient jetty to be cleared away, up to a level of M S L - 2 m

From an econamical point of view two materials came inta cansideratian for the dam core constructian above water mine-waste and lean sand asphalt The latter was chosen because of its low permeability, which will be explained later

In the standard design the armaur was designed as an almost traditianal fully grauted stane-layer, which is to be regarded as impermeable Fram investigations in an electric analogue, it appeared that in the lean sand asphalt dam care, being af the same permeability as the sand it is made of, pressure gradients develap during the tidal cycle which resulting in lower water pressures under the armaur than the dam care were made of mine-waste

This is why lean sand asphalt was chosen as the dam care material

In view of the geametry of the dam and the maderate permeability of the dam core, the influence af wave actian an the water pressures cauld be neglected

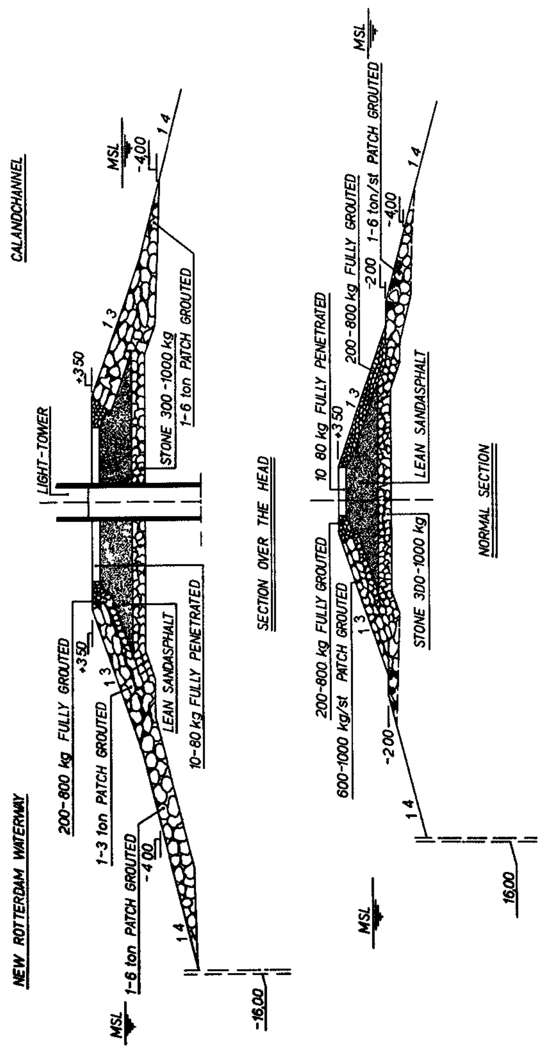

CALANDCHANNEL
NEW ROTTERDAM WATERWAY
LIGHT-TOWER
SECTION OVER THE HEAD
MSL
-4,00
1:4
1:3
+3,50
STONE 300-1000 kg
1-6 ton PATCH GROUTED
200-800 kg FULLY GROUTED
1-3 ton PATCH GROUTED
1-6 ton PATCH GROUTED
LEAN SANDASPHALT
10-80 kg FULLY PENETRATED
-16,00
NORMAL SECTION
10-80 kg FULLY PENETRATED
200-800 kg FULLY GROUTED
1-6 ton/st PATCH GROUTED
-2,00
LEAN SANDASPHALT
STONE 300-1000 kg
600-1000 kg/st PATCH GROUTED
16,00

FIG 14

During the actual construction of the dam, which started in December 1969, the amplitude of lean sand asphalt for the dam core proved to be succesful also as far as the stability under wave action during the execution of the works is concerned Negligible losses of dam core material were suffered, in spite of several heavy storms encountered during construction time

5 3 Test sections

As the Authorities (i e , Rijkswaterstaat) are very much interested in the recently-developed pattern-grouting system as described in Paragraph 3 3 two test sections will be constructed in the near future to investigate the merits of this method actually These sections are situated at the most exposed part of the dam, namely, the head and the adjacent part of the trunk

Of these two wave attack on the head is assumed to be frontal with a significant wave height of H_s = 6 m, whereas the trunk is mainly attacked by oblique waves of the same height

Using Hudson's formula with K_D = 2 9 for angular quarry stone as the head and an upgrading factor F = 5, 1 - 3 ton stones are used for the armour, to be pattern-grouted with 6 ton patches Model investigations have confirmed the stability of this armour, and from these it could also be deduced that the pattern-grouting had to be used down to a level of M S L - 4 m Below this level stone 1 - 6 ton is sufficiently stable without pattern-grouting

For economical reasons the recommended third layer is made of stone of a somewhat lighter class 200/800 kg This layer is applied on a layer of light permeable stone-asphalt, which in its turn protects the lean sand asphalt core

Wave attack on the trunk will be less than on the head, as it consists mainly of oblique waves Therefore the trunk armour is designed assuming K_D = 3 5 Because trunk sections are also considered as test sections and upgrading factor F = 10 has been applied which possibly may lead to some damage within a few years Together with the extensive wave measuring system of Rijkswaterstaat in the Europoort area it will be possible in this way to obtain insight in the behaviour of this type of structure under prototype conditions

6 CONCLUSIONS

Considering the development of the use of asphalt in breakwater construction it appears that in view of its favourable properties and its peculiarities, the most profitable application in this part of hydraulic engineering is the use as a grouting-agent

As a matter of fact, asphalt grouting already constituted the beginning of the development of fixating unstable slopes New techniques and working methods, however, grouting is developing into a system of internal armour protecting rubble-mound breakwaters and sea walls against the heaviest wave attack

In the pattern-grouting technique described in this Paper a new tool is given to the designer of a rubble-mound breakwater In practice it often happens that with the available rock-size from the quarry an armour-layer can be designed which is stable enough under wave attack of "normal" frequency, while just lacking stability in the exceptional design-storm Pattern-grouting provides that extra "upgrading" which is needed for the exceptional design wave, at relatively reasonable costs For design purposes an upgrading of 5 can be safely accepted

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