RIJKSWATERSTAAT COMMUNICATIONS

THE USE OF ASPHALT IN HYDRAULIC ENGINEERING

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The views in this article are the author's own.

Preface

Asphalt products have been used in the Netherlands in hydraulic engineering for a long time on a large scale, especially after the great disaster in 1953 when a large part of western Holland was flooded by the sea. After the disaster a great number of dikes had to be repaired very quickly and this was possible with the use of asphalt as a revetment material. Asphalt could be placed much faster then the materials most commonly used in those days. Further more asphalt is, when desired, more watertight than clay which is also scarce. Except for bitumen no foreign construction materials are necessary.

Skill, experience and knowledge were only slightly available in those days and had to be gained in the course of time. After some years it was thought useful that directives or recommendations were available. A first attempt to achieve this was made by the Working Group for Sealed Revetments who finished its work in 1961 with the 'Preliminary Report'. In 1979 the Technical Advisory Committee on Waterdefences decided to create new directives for the use of asphalt in hydraulics. The time was thought right because of the great developments and knowledge obtained since 1961. The building of the Delta Works including the Eastern Scheldt Storm Surge Barrier was a great contribution to this. In January 1984 the guidelines completed and now, one year later, an English translation is published.

The guidelines have been composed by a group in which representatives of public offices, companies and research centres worked together. It was attempted to make the content as complete as possible. Experience and knowledge were gathered, investigations carried out and design models made. It is inevitable however, that there are some shortcomings. Therefore it is intended to adapt the guidelines from time to time and to update them with new information. Comments and reactions of the reader are therefore very welcome.

The guidelines were written for Dutch purposes. This means that mostly applications to dikes, breakwaters and bed protections are considered. Other types as for instance reservoir and dam applications are only mentioned without specific details.
Also, reference is made to Dutch standards and directives only. Despite these aspects these guidelines will certainly contribute to a better design, execution, management and maintenance for hydraulic asphalt constructions, not only in the Netherlands but also in other countries, the reason for the translation into the English language.

The translation, which was made possible by the Dutch Ministery of Public Works and

Bitumarin B.V., was done by B. Wade. Many thanks to Dr. P. C. Barber (Ceemaid Serviced Ltd.), C. Davies (Metropolitan Borough of Wirral), Hydraulic Research Ltd., J. Harrison (Bitumarine Ltd.), D. D. Davidson (CERC-WES, Vicksburg, MS), Ir. J. P. J. van der Heide (The Association of Dutch Asphalt Contractors) and Ir. J. A. van Herpen (Oranjewoud Engineering Consultants) for their contribution and remarks. Last but not least mrs. D. van Eerd and mrs. W. Verhoeven should be mentioned.

I am convinced that these guidelines will help you to a better use of asphalt in hydraulic engineering.

Leiden, January 12th, 1984

Prof. Ir. P. A. van de Velde
Formerly Chairman of the
Technical Advisery Committee
on Water Defences

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0 Introduction

0.1 Format for guidelines

These guidelines are intended for persons and organizations who wish to inform themselves about the use of asphalt in hydraulic engineering. Designers, asphalt technologists, managers, government officials and contractors will find the information they need in these guidelines. The general form of the guidelines is such that they can also be used as teaching material.

Originally the subject of the guidelines was the use of asphalt products as a dike revetment material. Although this aspect is still covered, in fact the final version has a much wider scope and includes, namely, all hydraulic engineering applications of asphalt mixes under Dutch conditions. The title now is, therefore, more suitably 'The use of asphalt in hydraulic engineering'.
As the title suggests the reader is guided through all aspects involved in the application of asphalt mixes in hydraulic engineering. It gives guidelines, design methods, background information and recommendations. Absolute specifications are not given. Similarly tender specifications have not been included. For the latter reference should be made to the work of the Dutch Foundation for Rationalization and Automatization in Road Construction (RAW), which also publishes standard tender specifications for several kinds of hydraulic works, including bituminous applications.

Originally the intention was to limit the guidelines to a number of recommendations without any detailed explanation. The objectives of the guidelines, however, and the complexity of the subject and the lack of knowledge about many aspects did not lend themselves to a neat package. It was, therefore decided to publish these comprehensive guidelines after receiving comments from future users of the book.

The guidelines are divided into several parts, each comprising a separate topic:
Part A 'Composition and properties of asphalt mixes'. This part deals with the general technology of the asphalt mixes used in hydraulic engineering. The object is to provide knowledge and insight into the subject and to give the basic information needed for the other parts. Material parameters, used in Part C, are also presented here.

Part B 'Material technology'. The asphalt mix types most commonly used in hydraulic engineering are discussed and evaluated specifically in this part. The subjects covered include the basic materials, the way in which the mix composition is decided, the necessary mix design tests and the mix properties.

Part C 'Technical aspects of the design'. This part deals with functional requirements, the starting points and the loads which develop on and in the material, related to the design of hydraulic engineering asphalt constructions. Design methods are also discussed here.

Part D 'Execution'. Execution of the work is considered, for each type of asphalt, subdivided into production, transport and placing.

Part E 'Management and Maintenance'. This part deals with the management and maintenance of asphalt revetments. Methods of repair are given after a discussion about the causes, tracing and prevention of the various types of damage which can occur.

In appendices the theoretical backgrounds to several parts of the guidelines and the development of design models are given.

The asphalt types considered in the guidelines are:
- asphaltic concrete
- mastic
- grouting mortars
- dense stone asphalt
- open stone asphalt
- lean sand asphalt
- membranes

Subdivision of the directives between the several aspects, is used to make the subject more generally easy to grasp and apply. For practical purposes, however, a subdivision into material technology, design aspects, execution, management and maintenance might be desirable per mixtype. A separate table, Table 0 (page 21) had, therefore, been included in which the mixes are given, directly related to the areas of application. If, for example, only one particular mix type is being considered then it is only necessary to consult those sections of the guidelines indicated in the relevant column of the table.

0.2 Organization

In 1979 the Dutch Technical Advisory Committee on Water Defences (Technische Adviescommissie voor de Waterkeringen, TAW) decided to publish a set of directives on asphalt dike revetments. This aspect of hydraulic engineering falls within the scope of TAW Working-Group 4: 'Dike Revetments' and this group has, therefore, taken the work under its wing. The detailed work needed was allocated, in 1980, to Sub-group 4A: - 'Guidelines for Asphalt Revetments on Dikes'.
The members of this group originate from governmental departments and business organizations involved in the application of asphalt in hydraulic engineering.

As of 1st January, 1981, Ir. J. A. van Herpen was appointed as project leader to coordinate the work involved and to write the guidelines.
At the time of publication (of the Dutch version) of the guidelines the Working-Groups were composed as follows:

Working-Group 4 'Dike Revetments'

Prof. Ir. P. A. van de Velde, Chairman	Former professor at the Delft University of Technology, the Netherlands
Ir. E. H. Ebbens, Secretary	Netherlands Water Defences Research Centre
Ir. W. Bandsma	State Road Engineering Department
Prof. Drs. W. van Dijk	Delft University of Technology
Ir. J. A van Herpen	Delft University of Technology
Dr. Ir. P. A. Kolkman	Delft Hydraulics Laboratory
Ir. H. L. Koning	Delft Soil Mechanics Laboratory
Ir. R. C. Koole	Pavement Consultancy Services
Ir. P. C. Mazure	Netherlands Water Defences Research Centre
Ir. W. Meulenberg	The Office of the Dike-reeve, 'Het Noordhollands Noorderkwartier'
Ir. P. Ruijgrok	Delft Soil Mechanics Laboratory
Ir. H. Visser	The Walcheren Polder Authority
Ir. G. M. Wolsink	Delft University of Technology

On 1st April 1983, Ir. W. J. Heijnen (Delft Soil Mechanics laboratory) and Ing. J. T. de Vries (Netherlands Water Defences Research Centre) resigned from the Working-Group.

Sub-group 4A 'Directives for Asphalt Revetments on Dikes'

Ir. E. H. Ebbens, Chairman	Netherlands Water Defences Research Centre
Ir. J. A. van Herpen, Secretary/Project Leader	Delft University of Technology
Prof. Drs. W. van Dijk	Delft University of Technology
H. J. A. J. Gruis	State Road Engineering Department
Ir. J. P. J. van der Heide	The Association of Dutch Asphalt Contractors
Ing. K. A. van den Hoek	State Delta-Works Department
Ir. H. L. Koning	Delft Soil Mechanics Laboratory
Ir. G. L. M. Mulder	Bitumarin B.V.
Dr. R. C. Reintjes	Volker Stevin Wegen en Asfalt B.V.

On 1st January 1981, Ir. R. F. G. M. Zijlmans (Netherlands Water Defences Research

Centre) was replaced by Ir. E. H. Ebbens of the same organization and on 10th September 1981, Ing. K. A. C. Mouw (State Delta Works Department) was replaced by Ing. K. A. van den Hoek. On 25th May 1982, Ing. J. J. van der Plas (The Association of Dutch Asphalt Contractors) resigned in favour of Ir. J. P. J. van der Heide and on 26st Juli 1982, Ir. W. Bandsma (State Road Engineering Department), resigned in favour of H. J. A. J. Gruis.
On the resignation of Ing. J. T. de Vries on 1st April 1983 (Netherlands Water Defences Research Centre) the duties of secretary were taken over by Ir. J. A. van Herpen.

In addition to the members of Sub-group 4A many other people and organizations collaborated in the realization of the directives, two in particular, should be mentioned by name, Ir. H. Roos (Bitumarin B.V.) and Ing. C. C. Montauban (State Road Engineering Department).

0.3 The tasks and working methods of Working-Group 4A

Working-Group 4A was instructed to prepare guidelines for the design, execution, management and maintenance of asphalt revetments on dikes. To this end the following programme was executed:

1. Collection of available information on the subject of the use of asphalt in hydraulic engineering.
2. Preparation of the directives.
3. Initiation and supervision of research, with the object of increasing the knowledge available on the subject.

In order to realise the directives in a reasonable time a project leader was appointed for a 3 year period, beginning 1st January 1981. With the help of some of the members of the Working-Group a programme was drawn up listing all the aspects of the application of asphalt in hydraulic engineering. The first step was to assemble all available information and then for each type of asphalt a subdivision was made into design methods, material technology, execution and maintenance.
After the data had been collected an attempt was made to formulate practical rules for designing asphalt revetments. This information was then presented in an interim report to future users – principally dike managers – and members of Working-Group 4, for comments.
The 'Wave problems on Dikes' and 'Water Movements in Dikes' Working-Groups 1 and 2 of TAW and also specialists from the Delft Soils Laboratory and the State Road Engineering Department were consulted on certain aspects.
The guidelines were eventually, drawn up on the basis of the information obtained and a draft was presented to the various groups mentioned above for further comment.
Results of research executed byWorking-Group 4A have been used in the guidelines. This research is discussed further at the end of this introduction.

0.4 Recommendations for the future

During the preparation of the guidelines it became apparent that, for various reasons, it is possible to improve the present methods of application of asphalt in hydraulic engineering. A number of recommendations are therefore given below to assist in future work of research and development in this field.

Recommendations:

- In connection with construction design a good description is required of the physical/mechanical properties of the various asphalt mixes under typical loading conditions. Any gaps in information should be filled by investigations as soon as possible.
- When investigating the properties of an asphalt mix it is always preferable to determine the mix composition by extraction from a sample.
- Dynamic loading tests on the subsoil are recommended to obtain an impression of the behaviour of the subsoil, for a range of moisture contents, on which the asphalt construction will be placed.
- The possibilities of using gravel in asphaltic concrete in hydraulic structures should be investigated further. At present only crushed stone is prescribed in the Dutch Eisen 1978 (6) (see also Section 9.2.1).
- An asphaltic concrete revetment construction consisting of several layers is recommended if, under the usual methods of compacting cracking in a single layer cannot be avoided or if the required voids ratio cannot be obtained. It is recommended that the appropriate method of compaction is determined at the start of the work.
- More information should be obtained about the durability and resistance to currents of open stone asphalt and lean sand asphalt.
- Lean sand asphalt appears to be a very good material to use for bunds. When more is known about the 'soil mechanical' properties the application in this field can certainly be extended.
- Better methods for designing mastic mixes and grouting mortars are desirable.
- It is recommended that methods and equipment are developed for tracing damage so that management and maintenance can be done more effectively. A rational system for the management and maintenance of revetments and bank protection can then be gradually developed.
- Calculation models for construction design are, generally, still in the stage of development. More practical and usable models will assist efficient designing.
- The re-cycling of revetment materials should be investigated.

0.5 Investigations

In order to implement the above recommendations certain investigations will be necessary. A number of investigations have been formulated by Working-Group 4A, some in cooperation with other organizations.

These investigations include:

- An investigation to determine the best method of compacting asphaltic concrete dike revetments. At this moment part of this investigation has been carried out by the State Road Engineering Department.
- An investigation, related to a new mixture design method for mastic, will be completed shortly. This investigation is being carried out in cooperation with the State Road Engineering Department.
- Investigations into the resistance of open stone asphalt against wave attack. These include:

  1. A recently completed investigation in the Delta Flume of the Delft Hydraulics Laboratory, De Voorst, for Bitumarin Ltd.
  2. A recent investigation in the by-pass channel of the navigation dam at Lith, the Netherlands, in cooperation with the State Delta-Works Department.

- A preliminary investigation into the mechanical properties of open stone asphalt executed at the Delft University of Technology.
- Cooperation in an investigation into the soil mechanical properties of lean sand asphalt carried out by the State Delta-Works Department. This investigation will be completed shortly.
- Investigations into the resistance to currents and durability of lean sand asphalt are in preparation.
- General investigations into the mechanical properties of asphalt mixes are being carried out by the State Road Engineering Department.
- Investigations into the possibility of recycling asphalt mixes have been started in cooperation between Rijkswaterstaat and private industry.

Results, already available from the above investigations, have been included in the guidelines. Results still to come will be published and will be included within revised editions of these guidelines.
It should be noted that the above investigations will not cover all the points recommended for study, and some thought should be given to the remaining aspects for which information is required.

Table 0 Guideline reference table – The table relates the various uses of asphalt in hydraulic engineering to chapter and paragraph readings for the various material types.

			mix type								
					grouting mortars				lean sand asphalt		
			asphaltic concrete	mastic	fully grouting	pattern grouting	dense stone-asphalt	open stone asphalt	core material	layers	membranes
Part B Material Technology	basic materials		9.1	10.1	11.1		12.1	13.1	14.1		15.1
	mix design		9.2.1	10.2.1	11.2.1		12.2.1	13.2.1	14.2.1		15.2.1
	mix design tests		9.2.2	10.2.2	11.2.2		12.2.2	13.2.2	14.2.2		15.2.2
	mix properties		9.3	10.3	11.3		12.3	13.3	14.3		15.3
Part C Technical design	designing a dense asphalt revetment against hydraulic uplift pressures		20.1	20.1	20.1		20.1				20.1
	designing a plate type asphalt revetment against wave impacts		20.2	20.2	20.2		20.1	20.2	app. VI	20.2.2	
	designing underwater bed protection against uplift by waves and currents		20.3	20.3	20.3		20.3				20.3
	designing a loose element revetment against wave attack					20.4					
	designing an asphalt revetment against current		20.5	20.5	20.5		20.5	20.5	20.5		20.5
	designing an asphalt revetment against settlement and scouring		20.6	20.6	20.6		20.6	20.6	20.6		20.6
	determination of the maximum slope		20.7	20.7	20.7		20.7	20.7	20.7		20.7
	other 'loads'		19.3	19.3	19.3		19.3	19.3	19.3		19.3
Part D Execution	production		22.1	23.1	24.1		25.1	26.1	27.1		28.2
	transport		22.2	23.2	24.2		25.2	26.2	27.2		28.3
	placing		22.3	23.3	24.3		25.3	26.3	27.3		29.2
	technical design	joints	29.2	29.2	29.2		29.2	29.2	29.2		29.2
		between different materials	29.3	29.3	29.3		29.3	29.3	29.3		29.3
		onto structures	29.4	29.4	29.4		29.4	29.4	29.4		29.4
		toe constructions	29.5	29.5	29.5		29.5	29.5	29.5		29.5
Part E Management and Maintenance	possible damage		32.1	32.2	32.3		32.4	32.5	32.6		32.7
	Methods of repair		33.1	33.2	33.3.2	33.3.1	33.4	33.5	33.6.2	33.6.1	33.7

PART A

COMPOSITION AND PROPERTIES OF ASPHALT MIXES

Summary

Part A deals with the general aspects of the composition and properties of asphalt mixes which are important in their application to hydraulic structures. The object of Part A, is to introduce the subject to the reader and enlighten him, making the manual more easy to read, and also to present data about the materials which are relevant to Part C which deals with design techniques.

Part A comprises:
- A discussion of the individual components of asphalt and their influence on the material as a whole. Attention is paid to those mix properties which are most important to hydraulic engineering.
- A review of the most frequently used asphalt mixture types in hydraulic structures and how these types are applied.
- Finally, a discussion of quality control including mix-design tests, construction controls and completion checks prior to handing over the work to the Client. Included here is a discussion of the way in which samples are taken.

1 Mix considerations

1.1 Mix components

Asphalt is a mix of various components:

– bitumen;
– mineral aggregate;
– additives, if required.

The mineral aggregate is composed of crushed stone, gravel, sand or filler or a combination.
The choice of the most suitable composition for a particular application depends mainly on the requirements which the material has to meet and the associated mix properties, see Section 6.1.
The mix properties are specified by the composition, that is, the relative proportions of the various components, the properties of the components themselves, and the properties which result from the application and compaction method.

1.2 The degree of filling of the mix

The mineral aggregate mix contains voids. Initially, the bitumen coats and binds the various aggregate components together. If more bitumen is applied than is necessary for coating and binding then the pores will gradually be filled.
Mixes, in which the bitumen only serves as a binder, are referred to as 'underfilled' mixes, see Figure 1.1a. The properties of such a mix are directly related to the properties of the stone skeleton (4). If the proportion of bitumen is increased the voids in the mineral become filled and the influence of the bitumen on the properties of the mix is increased while that of the stone skeleton is reduced. With mixes in which the pores are almost filled with bitumen, see Figure 1.1b, both the stone skeleton and the bitumen contribute to the mix properties. This type of mix must be compacted, either mechanically or under its own weight.
'Overfilled' mixtures are those in which the volume of bitumen is greater than that of the voids in the mineral aggregate. In such a mix the properties of the bitumen predominate, the mineral providing only a certain amount of stiffening, see Figure 1.1c. This type of mix is impermeable and requires no compaction.

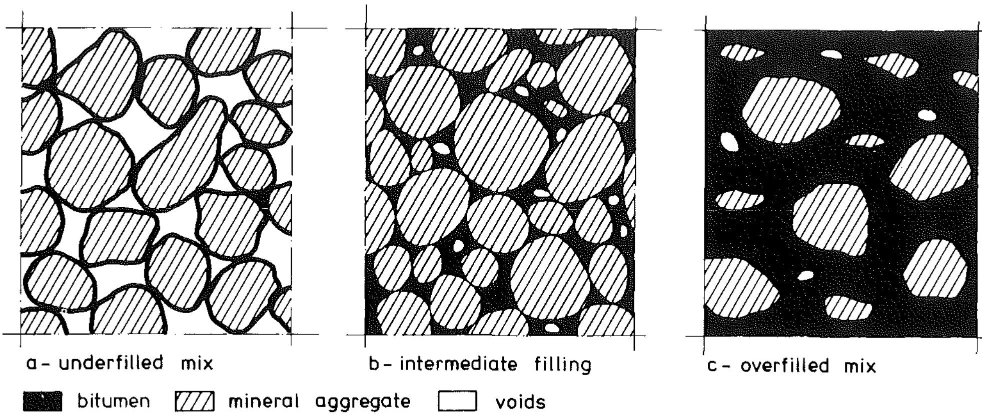


Figure 1.1 The degree of bitumen filling in the mineral aggregate.

1.3 Voids in asphalt mixtures

The term 'voids' refers to the volume of pores in the compacted asphalt (5). The voids ratio, VIM (voids in mix), is given by:

$$\text{VIM} = 100(\frac{d_\text{m} - d_\text{a}}{d_\text{m}}) \text{ vol \%}$$

d_m = density of the mix without voids (kg/m^3)
d_m = density of the mix with voids (kg/m^3)

In general, the smaller the voids ratio (HR) the more resistant is the mix to erosion and the greater its durability. A mix with a small voids ratio is better 'sealed' against external influences such as oxygen, light and water. If water penetrates between the bitumen and the mineral (through the material) there is a loss of adhesion which is referred to as 'stripping'.
Exposure to the atmosphere and light ages the bitumen. In this respect, the size of individual pores and degree of interconnection between voids are also important.
The voids ratio and the distribution of voids also determine whether or not the mix is sand and watertight. Although water impermeability is not always a functional requirement it gives a good indication of the durability of a mix.

To illustrate:
- A mix containing sand with 5%, by mass, of bitumen and a voids ratio of 25% is sand-tight. A mix, however, of open stone asphalt with 80%, by mass, of stone and 20%, by mass, of mastic with the same voids ratio is not.
- An asphaltic concrete with a voids ratio of 3% can be considered as absolutely watertight. In this case the voids are not interconnected. For the same reasons a mastic with the relatively large voids ratio of 10% is also watertight.

2 The mineral aggregate

2.1 General

It is important that there is good adhesion between the bitumen and the mineral aggregate. There are two kinds of adhesion:

1. Physical adhesion:
   This type of adhesion is better if the surface of the mineral aggregate is rough.
2. Chemical adhesion:
   Since bitumen is weakly acidic, better adhesion is obtained with mineral aggregates which are slightly basic. This property is generally to be found with minerals which contain a limited amount of silica oxide.
   It is also very important that dry aggregate is used.

A low voids ratio is ensured in an asphalt mix which is not overfilled by adjusting the coarse fraction in relation to the sand fraction and by using a well-graded mineral aggregate: the spaces between the larger particles are then filled by the smaller.
Increasing the filler fraction, provided that it does not expand the sand/-stone skeleton, can produce greater internal stability. The internal stability of the material in place can also be increased by using an angular material instead of round, for example crushed stone instead of gravel, crushed sand instead of natural sand.
The quantity of bitumen needed to bind the mineral aggregate depends on the specific aggregate surface. The specific surface is inversely proportional to the second power of the particle diameter.
Mineral aggregate can be subdivided according, to grain size. In the Netherlands the following terminology is used:

- stone fraction, larger than 2 mm;
- sand fraction, between 2 mm and 63 μm;
- filler fraction, smaller than 63 μm.

2.2 The stone fraction

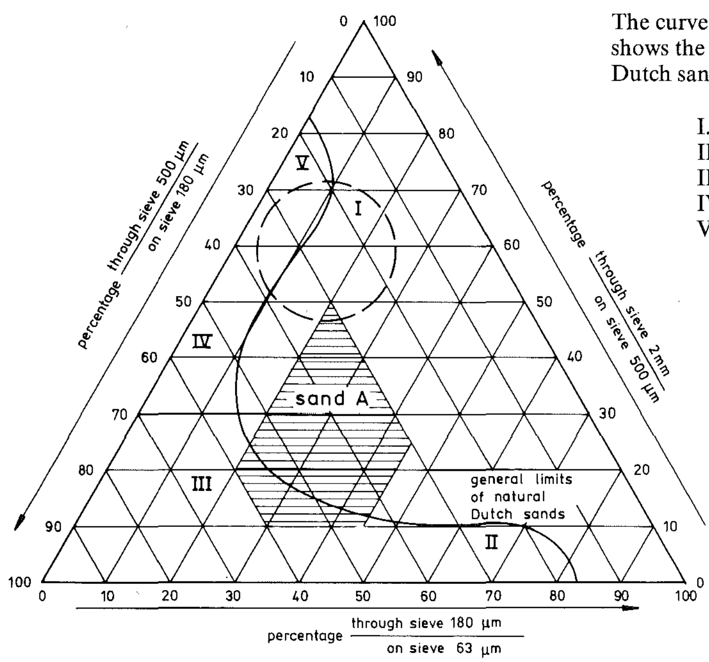
The stone fraction comprises crushed stone, that is a rough broken material, or a smooth round material such as gravel.
Crushed stone and gravel must be able to withstand impact and abrasion during mixing and compaction. This property depends on:

- particle shape; cubes are the best;
- the strength of the material.

Angular material has a higher interlocking strength.
The maximum particle size in an asphalt mix is related to layer thickness and the production and working methods. Generally a large grain size gives better stability but reduces workability. In addition the danger of segregation is greater. For the stone properties required, reference should be made to the Dutch specifications, Eisen 1978 (6).

2.3 The sand fraction

Natural sand, crushed sand or a mixture of both are used for bituminous mixes. The sand grain size distribution plays an important part in stability, voids ratio and the binder requirements for a mix. Sometimes the grain size distribution is specified. If this is not the case, laboratory investigations are carried out to determine the sand gradation which most economically satisfies the required mix properties. Often this can be achieved by mixing different varieties of sand.
Sand can be characterized by the mass percentages of sieve fractions lying between, for example, the 2 mm, 500 μm, 180 μm and 63 μm sieve sizes (numbers 1a, 30, 85 and 24c respective y). The composition can be shown graphically using the sieve fractions in, what is referred to as, the 'sand triangle', see Figure 2.1.



The curved line in the sand triangle shows the general limits of natural Dutch sands

I. sharp sand
II. very fine sand
III. medium fine sand
IV. medium coarse sand
V. very coarse sand

Figure 2.1 The sand triangle (5).

Sand is said to be better graded if it lies within the shaded area of the triangle. This sand is referred to as Sand A. According to Eisen 1978, the grain size fractions are:

Through Sieve size	On Sieve	Percentage by mass Desired	Limits
2 mm	500 μm	25	10–50
500 μm	180 μm	40	30–60
180 μm	63 μm	35	20–45

2.4 Filler

Filler (5):

1. fills the voids of the stone-sand mix producing a more uniformly graded material;
2. forms, together with the bitumen, the binder required;
3. has a stiffening effect on the binder which increases the viscosity and reduces the risk of segregation.

In general rather flexible mixes are required for hydraulic structures. 'Weak' to 'very weak' filler is used, therefore, which requires little bitumen binding and as a result more 'free' bitumen is obtained to provide flexibility. The preference is for filler which is hydrophobic and has a basic reaction, so that the adhesion between the bitumen and the stone is improved.
The quantity of filler and its voids ratio largely determine the quantity of bitumen required. For these reasons, and for optimum workability, the filler must have strictly controlled requirements such as:
- a constant absorptivity of bitumen;
- a constant nature and quality.

Limestone fillers should be used for preference.

Methods of characterising filler can include (63):
1. the Rigden dry compaction test, in which a particular weight of filler is compacted by a standard method and then its volume recorded.
2. the Van der Baan test to determine the bitumen number. In this test the volume of water required to bring the filler to a particular consistency is recorded (95).

For the properties required reference should be made to Eisen 1978 (6).

3 Bitumen

Bitumen is a very viscous, non-volatile material which principally consists of hydrocarbons or their derivatives (5). The viscosity of bitumen depends on the temperature; it is what is referred to as a thermoplastic material. Although it does not react to water and most chemicals it does dissolve in lighter hydrocarbons. Its performance under load is strongly dependent on its temperature and the duration of loading.

The bitumen can be defined by using empirical parameters:

- In the normal temperature range by the penetration and the softening point ring and ball. The penetration (pen 25°C) is the intrusion into a bitumen sample measured in units of 0.1 mm, of a standard probe with a weight of 100 gram at a temperature of 25°C during a period of 5 seconds. The softening point ($T_{r\&b}$) is the temperature, in °C, at which a slice of the material, held firmly in a ring under standard test conditions, undergoes a standard deformation under the weight of a metal ball.
  By temperature sensitivity is understood the extent to which the viscosity depends on the temperature. This dependency is described by the penetration index (PI).
  The higher the penetration index, the lower the temperature sensitivity.

$$\text{PI (pen, pen)} = \frac{20 - 500 \cdot A}{1 + 50 \cdot A}$$

in which:

$$A = \frac{\log 800 - \log \text{pen}}{T_{r\&k} - 25}$$

pen = penetration at 25°C (units of 0.1 mm)
$T_{r\&b}$ = softening point (°C). The penetration at the softening temperature for normal bitumen is about 800_{pen} (800 × 0.1 mm. = 80 mm)

The PI value and the softening point can be determined very simply from two penetration values using Figure 3.1.

For standard bitumen PI generally lies between +1 and −1. These bitumens are often referred to in the trade by the limits in which the penetration must lie, for example, bitumen 80/100.

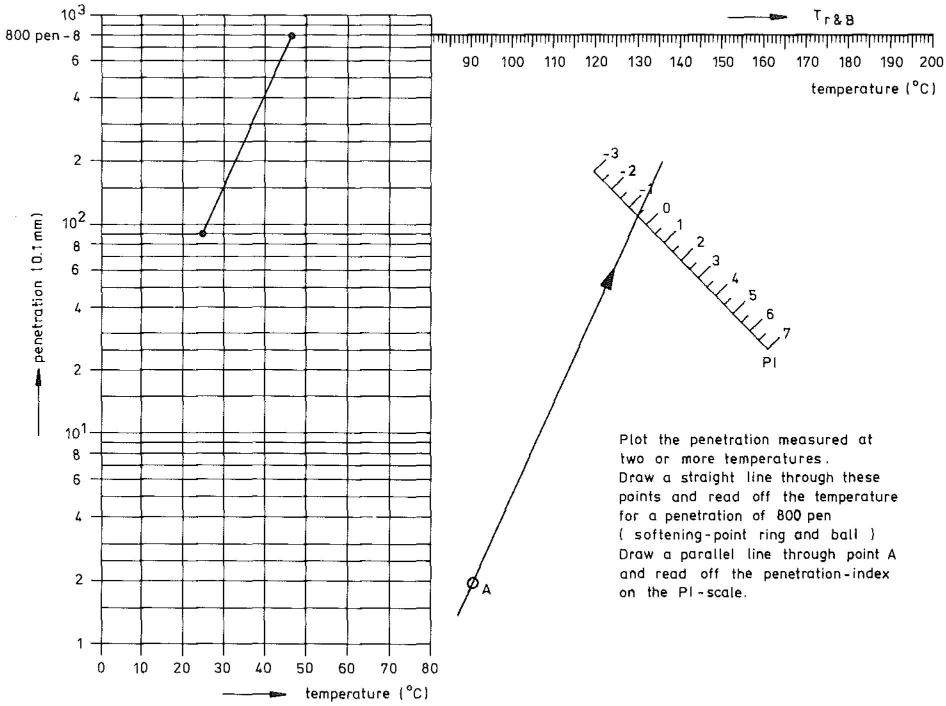


Figure 3.1 Determination of the softening point (5).

- In the low temperature range the Fraass breaking point, referred to as 'the brittle temperature' indicates the consistency.
  This number indicates the temperature at which a 0.5 mm thick layer of bitumen cracks under a bending load.
- The higher temperature range is important for the mixing and application of asphalt mixes. In this connection a certain viscosity is essential; the following values for kinematic bitumen viscosity have been determined for different operations (5):

  - spraying, about 20-50 mm^2/s
  - mixing with mineral aggregate, about 150-300 mm^2/s
  - pumping from a bitumen truck, about 1200 mm^2/s
  - placing and compacting, depending on the type of mix, about 300-2000 mm^2/s.

Remarks: The dynamic viscosity (η) is measured in Pa · s (Pascalseconds); the kinematic viscosity (ν) in m^2/s.
The relationship between the dynamic and kinematic viscosity is given by $\nu = \eta/\rho$, where ρ is the density of the bitumen in kg/m^3, see Table 3.1.

1 Pascal = 1 N/m^2
1 Poise = 0.1 Pa · s
The stokes unit is also used for kinematic viscosity.
1 Stokes = 10^{-4} m^2/s

Temperatures related to equal viscosities are referred to as Equi Viscosity Temperatures (EVT). So, for example, there is an EVT to which a bitumen must be heated to produce the mixing viscosity of 170 mm^2/s. This temperature is 160 to 170°C for bitumen 45/60 and 150 to 160°C for bitumen 80/100, see Figure 3.2.
When bitumen is exposed to the atmosphere it hardens, which lowers the penetration and raises the softening point ring and ball. Hardening is among others caused by the evaporation of the volatile elements and oxydation. The higher the temperature the faster the process.
Considerable hardening takes place during asphalt mixing and application. Hardening which develops during the course of time is referred to as aging.

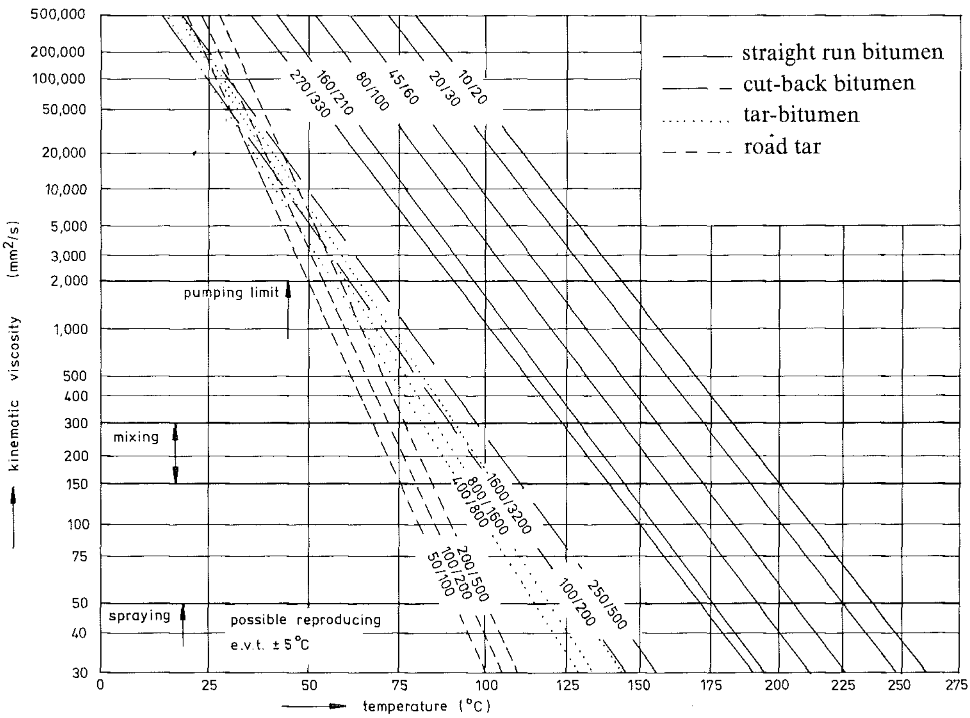


Figure 3.2 The viscosity of different bitumens as a function of temperature (5).

The quality of a bitumen must be such that hardening is not excessive. Investigations into hardening can be carried out in 'penetration after loss on heating' tests, in which a bitumen sample is heated at 163°C for a period of 5 hours. For thermostability there should be very little difference between penetration before and after the test.

After production at the refinery the bitumen must not be heated above 200°C since then the material properties can change and certain cracking processes can develop.

Bitumen must not be too hard; the working viscosity should be sufficiently low (about 0.2 pa . s at 140 tot 160°C) and the Fraass breaking point must not be reached under critical mechanical loads at low temperatures (8).

For requirements and tests related to bitumen, reference should be made to Eisen 1978 (6) or NEN 3902 (21).

Table 3.1 Density of penetration bitumen at various temperatures (5).

Bitumen (pen)	Temperature (°C)											
	25°	100°	110°	120°	130°	140°	150°	160°	170°	180°	190°	200°
	Density (1000 kg/m³)											
270/330	1,01	0,97	0,96	0,95	0,95	0,94	0,94	0,93	0,93	0,92		
160/210	1,02	0,97	0,96	0,96	0,96	0,95	0,94	0,94	0,93	0,93	0,92	0,92
80/100	1,03	0,97	0,98	0,97	0,97	0,96	0,96	0,95	0,95	0,94	0,93	0,93
45/60	1,04		0,98	0,98	0,97	0,97	0,96	0,96	0,95	0,94	0,94	0,93
20/30	1,05			0,99	0,98	0,97	0,96	0,96	0,96	0,95	0,95	0,94

4 Additives

Additives are used to improve adhesion by lowering the surface tension between the bitumen and the mineral aggregate. This limits stripping.
The effect of the additive is noticeable in the short term but, after about two years, is no longer apparent. Adhesion-improving additives have been used incidentically in hydraulic structures. It should be borne in mind that the viscosity can be affected.
Properties such as the viscosity can be affected by the addition of polymers, both chemically and physically. For the time being, polymers are not used in the hydraulic applications.

Additives can be added during production and/or before application of asphalt mixes. When adding to the bitumen, segregation must be prevented.

5 Mix composition

The mix composition is mostly designed on a choice, within certain limits, of particle grain size distribution and bitumen content, on the basis of investigations into the mechanical properties, and by application of the basic materials allowed and available. For different applications the composition is, generally, specified in the form of rough proportions by mass. The detailed composition can then be determined from laboratory and in-situ tests.

There are two general principles involved for determining mix composition (9):

1. Design for stability, if a mix of a certain strength is required. The bitumen content is, in this case, adjusted to the grading of the mineral aggregate.
2. Design for a particular viscosity if a flexible mix is required. In this case a certain overfilling of voids is essential.

6 Mix properties

6.1 General

As already stated the choice of mix composition depends on investigations into the mix properties. The most important properties for use in hydraulic engineering are:

- degree of permeability
- mechanical properties
- stability
- durability
- workability and compactability

6.2 The degree of permeability

A dike revetment – of which asphalt can form a part – must be completely sandtight. Sometimes a cover layer is also required to be watertight. The voids ratio of a mix and the size and orientation of the voids determine the degree of permeability of the mix as a whole. The factors which affect the voids are given in Section 1.3.
If a sand-permeable asphalt mix is chosen for a revetment then the sand seal must be provided by a filter construction. In the case of watertightness a stricter sealing criteria will be essential for asphaltic mixes used for sealing water reservoirs than for dike-revetments.
For example; a 5 cm thick plate of bituminous mix, with a voids ratio of 3% should not allow any water through when subjected to a water pressure of 3 atm for a few hours (12). For mixes with such a low voids ratio special thought must be given to the compaction operation because of the possibility of initiating cracks in the material.

6.3 Mechanical properties

Asphalt mixes and bitumen appear to have similar mechanical properties. An asphalt mix, in the same way as bitumen, is a visco-elastic material which under short duration loading and at low temperatures appears to be elastic; under long duration loads and higher temperatures it is viscous. The visco-elastic property of asphalt is an advantage for applications such as in dike revetments; it is stiff under short duration loads such as wave impacts but yields under long duration loads such as those occurring during settlement.

Remarks
The following sections give nomograms for determining the stiffness modulus, the initial

strain at break and the Poisson ratio of an asphalt mix. It should be borne in mind that these nomograms only give general values. If more accurate values are required it is better to carry out separate investigations on each mix, including, for example, three or four-point bending tests and creep tests.

6.3.1 *The stiffness modulus*

For an elastic material deformation is proportional to the stress applied. The modulus of elasticity, E, also referred to as Young's Modulus, is often used. This modulus is independent of temperature and loading duration.

$$E = \sigma/\varepsilon$$
$$\sigma = \text{stress } (N/m^2)$$
$$\varepsilon = \text{strain } (-)$$

There is a similar relationship for bitumen, which, however, is strongly dependent on temperature (T) and loading duration (t). The stiffness modulus, S, is used to characterize the material (5).

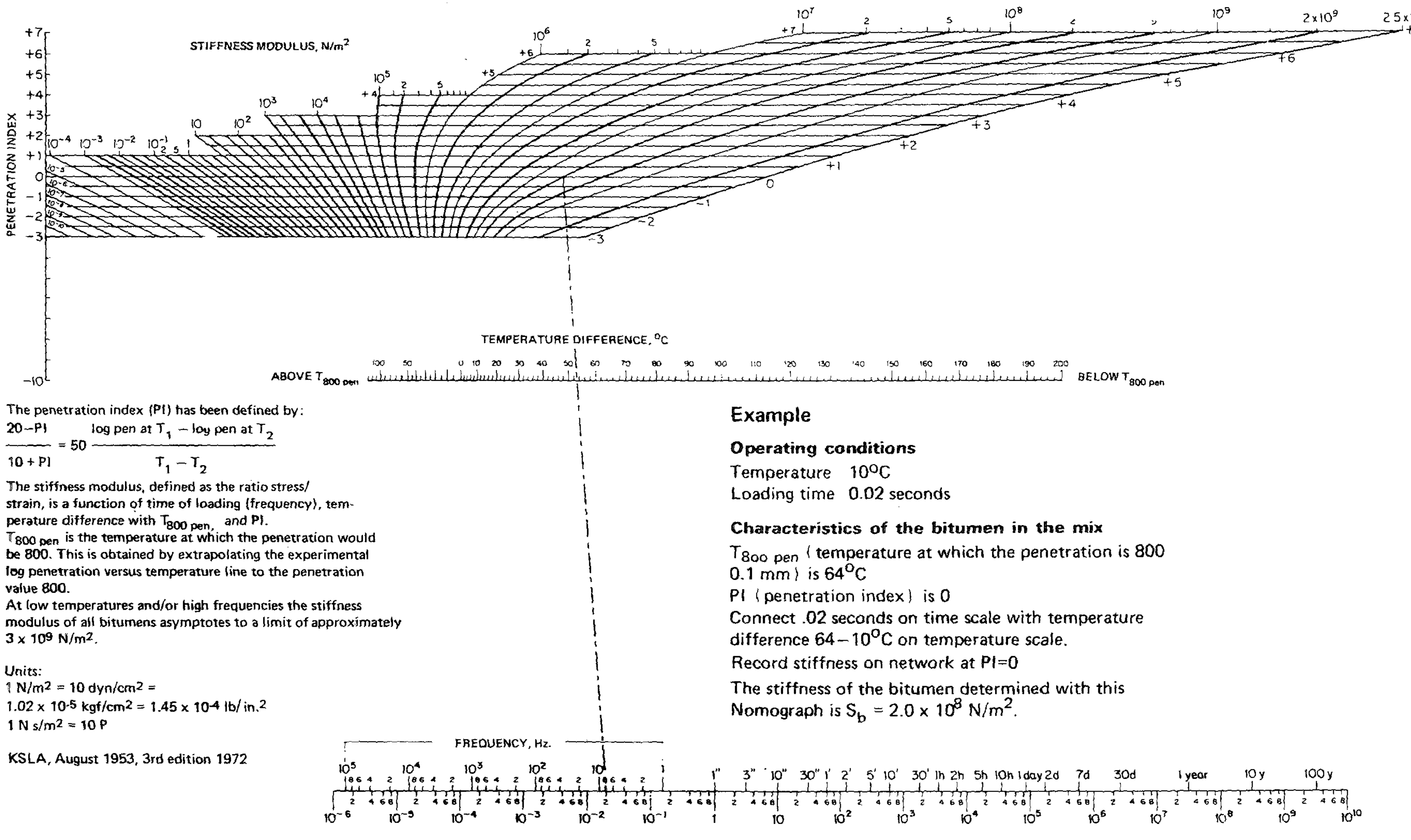
$$S(t, T) = \frac{\sigma}{\varepsilon}$$

For increasing values of t and T the values of S reduce.

The nomogram, prepared by Van de Poel, which can be used to determine the stiffness modulus of bitumen is shown in Figure 6.1.

Since an asphalt mix is visco-elastic the stiffness modulus can also be applied. From research it appears that there is a connection between the S-modulus of bitumen (S_{bit}) and that of an asphalt mix (S_{mix}). This relationship is determined, amongst other things by the volumetric percentages of the mineral aggregate and the bitumen. The relationship between S_{mix} and S_{bit} can be shown in what is known as a master curve. An example of such a curve is given in Figure 6.2.

Over the years various laboratories have carried out research and have developed nomograms for determining the stiffness modulus of asphalt mixes. The most recent example of these is give in Figure 6.3.



The penetration index (PI) has been defined by:

$$\frac{20-PI}{10+PI} = 50\,\frac{\text{log pen at } T_1 - \text{log pen at } T_2}{T_1 - T_2}$$

The stiffness modulus, defined as the ratio stress/strain, is a function of time of loading (frequency), temperature difference with $T_{800\ pen}$, and PI.
$T_{800\ pen}$ is the temperature at which the penetration would be 800. This is obtained by extrapolating the experimental log penetration versus temperature line to the penetration value 800.
At low temperatures and/or high frequencies the stiffness modulus of all bitumens asymptotes to a limit of approximately 3×10^9 N/m².

Units:
1 N/m² = 10 dyn/cm² =
1.02×10^{-5} kgf/cm² = 1.45×10^{-4} lb/in.²
1 N s/m² = 10 P

KSLA, August 1953, 3rd edition 1972

Example

Operating conditions

Temperature 10°C
Loading time 0.02 seconds

Characteristics of the bitumen in the mix

$T_{800\ pen}$ (temperature at which the penetration is 800 0.1 mm) is 64°C
PI (penetration index) is 0
Connect .02 seconds on time scale with temperature difference 64 – 10°C on temperature scale.
Record stiffness on network at PI=0
The stiffness of the bitumen determined with this Nomograph is $S_b = 2.0 \times 10^8$ N/m².

Figure 6.1 Nomogram for determining the stiffness modulus of bitumen (Van de Poel) (64).

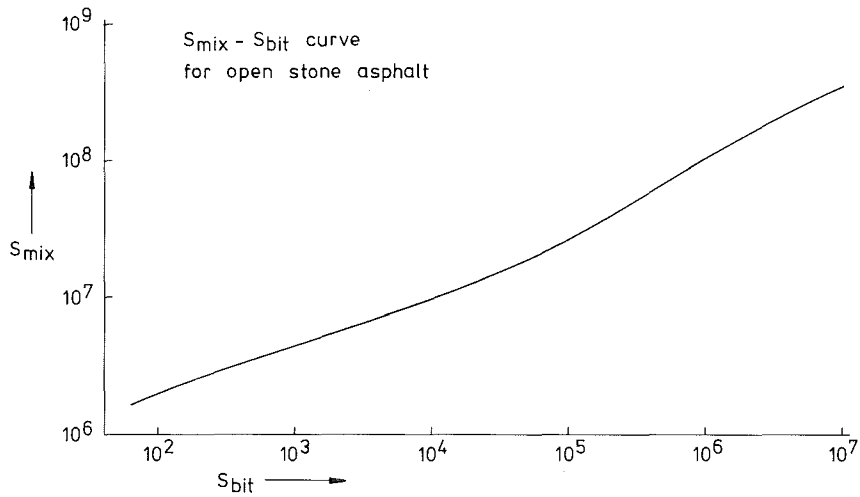


Figure 6.2 $S_{mix} - S_{bit}$ relation for open stone asphalt (master curve).

The boundary conditions required for Figure 6.3 are the volumetric percentages of bitumen and mineral aggregate. These can be derived from the mix composition in the following way:

$$V_b = d_a \cdot \frac{m_b}{d_b}$$

$$V_g = d_a \left(\frac{m_s}{d_s} + \frac{m_z}{d_z} + \frac{m_f}{d_f} \right)$$

$$HR = 100 \left(1 - \frac{d_a}{d_m} \right)$$

$$d_m = \frac{100}{\left(\frac{m_s}{d_s} + \frac{m_z}{d_z} + \frac{m_f}{d_f} + \frac{m_b}{d_b} \right)} \qquad \text{if } m_s + m_z + m_f + m_b = 100\%$$

in which:

V_b = volumetric percentage of bitumen
V_g = volumetric percentage of mineral aggregate
m = mass percentage
d = density (kg/m^3)
d_a = density of asphalt mix with voids (kg/m^3)
d_m = density of asphalt mix without voids (kg/m^3)
HR = voids ratio of the asphalt mix

The indices s, z, f and b refer respectively to stone, sand, filler and bitumen.

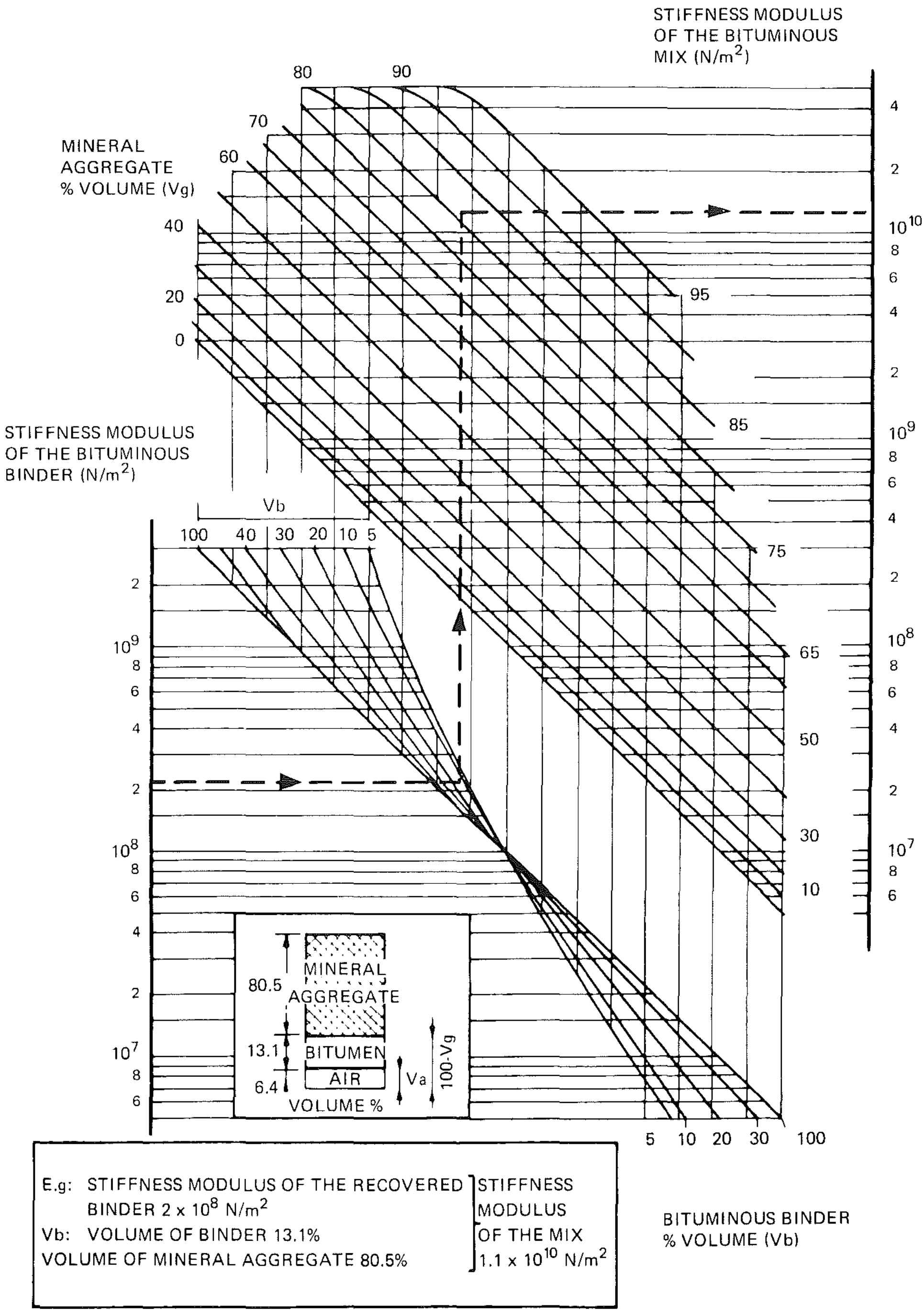


Figure 6.3 Nomogram for determining the stiffness modulus of asphalt mixes (Bonnaure et al) (65).

If the density of the various components is not known precisely the following general values can be used for initial guidance:

bitumen:	$d_m = 1020 \text{ kg/m}^3$
granite:	$d_m = 2850 \text{ kg/m}^3$
basalt:	$d_m = 2950 \text{ kg/m}^3$
limestone:	$d_m = 2700 \text{ kg/m}^3$
river and dune sand:	$d_m = 2650 \text{ kg/m}^3$

The nomogram, Figure 6.3., has an accuracy of the order of 1.5 to 2 and is valid for $S_{bit} > 10^6$ and $S_{mix} > 10^8 \text{ N/m}^2$.
For lower values of S_{bit} and S_{mix} other factors such as particle shape and particle size distribution of the mineral aggregate play an obvious role. The stiffness modulus can be determined from either static or dynamic tests.
For mixes with a voids ratio of upto 5% the stiffness modulus can be estimated from the relationship (66):

$$S_{mix} = S_{bit}\left(1 + \frac{2{,}5}{n} \cdot \frac{C_v}{1 - C_v}\right)^n$$

in which:

$$n = 0{,}83 \log\left(\frac{4 \cdot 10^{10}}{S_{bit}}\right)$$

$$C_v = \frac{\text{volume of the mineral aggregate}}{\text{volume of the mineral + bitumen}}$$

S_{bit} = stiffness modulus of bitumen (N/m^2)

6.3.2 *The strain at break*

In order to calculate an asphalt construction it is necessary to know the strain at break as well as the stiffness modulus.
Asphalt mixes are sensitive to fatigue and the value of the strain at break reduces the more often the material is loaded. The following general fatigue relationship can be applied:

$$N = k \cdot \sigma^{-a}$$

in which:

N = the number of load repetitions, of size σ, at which the material fails
σ = magnitude of the applied stress
k and a = constants for a particular type of mix with a certain stiffness modulus;

the factor a, in general, lies between 3 and 7 and stands for normal, slightly fat mixes, usually at 5.

The fatigue relationship between the number of load applications and the initial strain at failure can be determined from dynamic tests or from Figure 6.4.
The Miner's Law is used, for conditions involving varying loads. This states that for each applied N loading cycles of a certain load value on the material, which has a loading cycle number at failure of N, the amount of damage will be proportional to n/N. Failure will occur when the summation of the damage amounts reaches a value of 1. For a combination of N_j applied loading cycles of amplitude σ_j the failure limit is reached when the following condition is satisfied (67):

$$\Sigma \frac{n_j}{N_j} = 1$$

A nomogram has been developed (68) in which the initial strain at failure related to the asphalt mix parameters S_{mix}, V_b and PI, can be estimated in relation to the number of load applications, see Figure 6.4. In the nomogram loads which can be considered as producing constant strain are separated from those producing constant stress, see the two different S_{mix} scales.

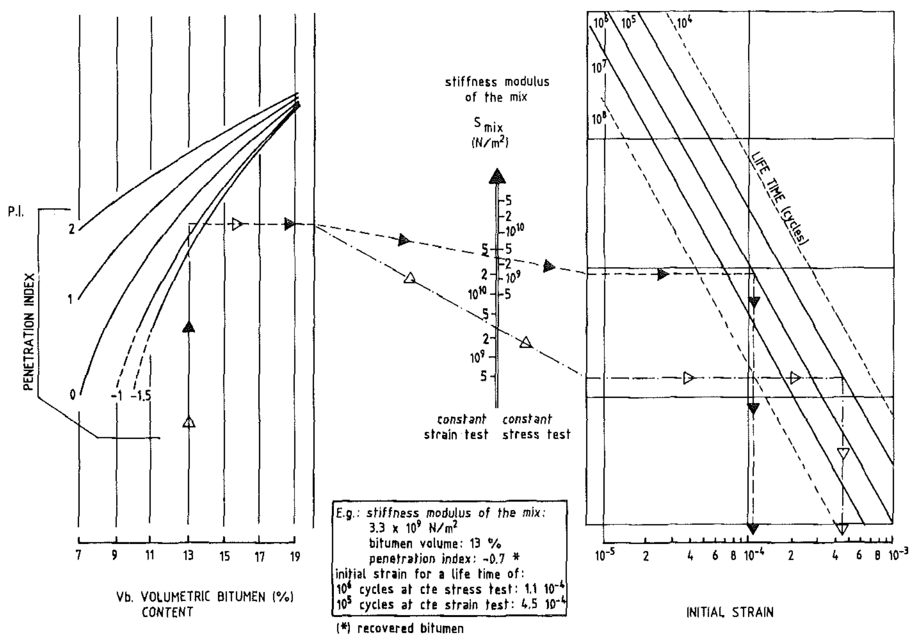


Figure 6.4 Nomogram for determining the failure strain of asphalt (68).

The basis for the nomogram are the formulas:

1. constant strain

$$\varepsilon_0 = (4{,}102 \times \mathrm{PI} - 0{,}205 \times \mathrm{PI} \times V_b + 1{,}094 \times V_b - 2{,}707) \times S_m^{-0{,}36} \times N^{-0{,}2}$$

2. constant stress

$$\varepsilon_0 = (0{,}300 \times \mathrm{PI} - 0{,}015 \times \mathrm{PI} \times V_b + 0{,}080 \times V_b - 0{,}198) \times S_m^{-0{,}28} \times N^{-0{,}2}$$

in which:

ε_0 = initial fatigue strain
PI = penetration index of recovered bitumen
V_b = volume of binder
S_m = stiffness modulus of the mix
N = number of loading cycles which cause the material to failure

If the loading stops after some time the asphalt (the mechanical properties) will recover to some extent. This aspect is known as healing.

6.3.3 *Permanent viscous deformation*

Under longer duration loads asphalt is viscous and permanent deformation can occur. The stiffness moduli in this range are low and cannot, in general, be obtained from Figure 6.3. To determine these parameters it is necessary to carry out static and dynamic tests, such as creep tests, on each particular mix.

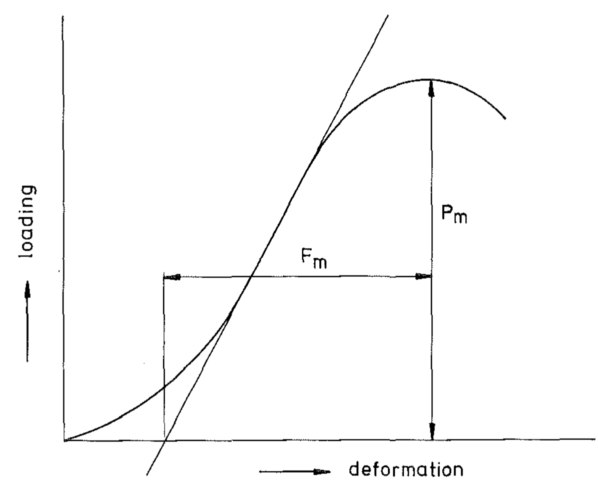


Figure 6.5 The Marshall diagram.

Much use is made of what is referred to as the Marshall test to determine the resistance to deformation (6). This test gives only values for comparison with other mixes and not specific quantities. In these laboratory tests, samples are subjected to specified loads and the deformation up to failure is recorded.
The load at failure is referred to as the Marshall-stability, P_m; the deformation which develops to failure as the Marshall-flow, F_m. The relationship between these parameters, recorded in the test, is given in the Marshall-diagram, see Figure 6.5.

6.3.4 *The Poisson-Ratio*

The Poisson-ratio gives the relationship between the strain in a sideways direction and the strain in the direction of the load. It can be estimated using values of $S_{bitumen}$ and the voids-ratio of the mix in Figure 6.6 (11).

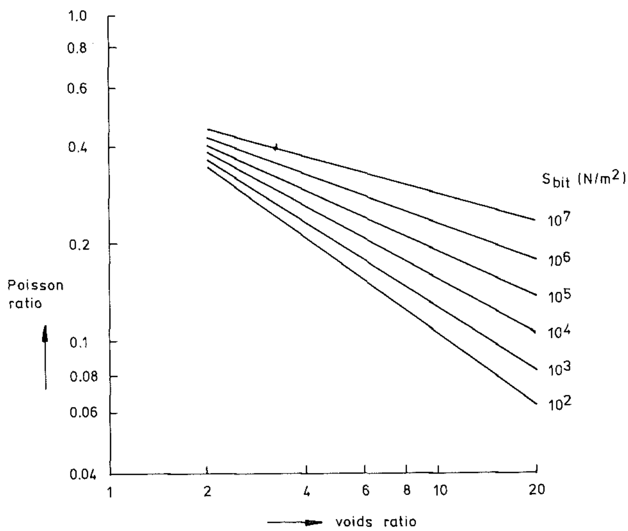


Figure 6.6 The Poisson-ratio.

6.3.5 *Temperature sensitivity*

In order to determine the properties of asphalt the surrounding temperature must be known. It is not sufficient, to consider only the maximum possible temperature, since, although the stiffness modulus is the lowest, the strain at failure is the highest. A solution can be to use the stiffness modulus and the strain at break for a range of temperatures and to choose the least favourable condition.
The temperatures can vary between some degrees below zero in sharp frosts or, if the cover is under water –2°C (supercooled seawater), to more than +50°C in direct sunlight.

6.4 Stability

6.4.1 *Stability of the asphalt*

If bitumen is laid in a layer on a slope, it will, because of its own weight and viscous properties, tend to flow down the slope, see Figure 6.7 and Appendix IX. An internal shear force must be mobilized to resist this flow.
Asphalt is a mixture of mineral aggregate and bitumen and the internal force which tries to resist the viscous flow is also developed by the friction between aggregate particles and is thus, to some extent, dependent on the normal pressure which the particles exert on each other. The bitumen has a lubricating action as a result of which the internal resistance is less than that found in the aggregate alone. The quantity of bitumen in the mix, therefore, has considerable influence.

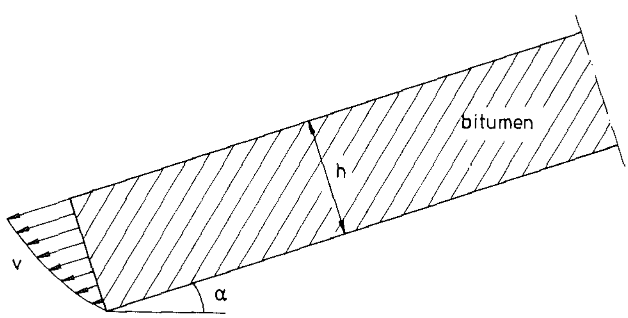


Figure 6.7 The viscous behaviour of bitumen on a slope.

6.4.2 *Stability of the revetment as a whole*

The revetment should be so heavy and extensive that it cannot move as a whole under the loads acting.

6.5 Durability

The revetment must, with the course of time, continue to fulfil its function. The characteristic mechanical properties should not deteriorate too much within a reasonable time. The following aspects will affect this.

6.5.1 *Aging*

Bitumen hardens with exposure to light and atmosphere. Material properties, such as the stiffness modulus, strain at failure and viscosity also change. This effect is more noticeable when the temperature is higher and the voids ratio larger. With a dense asphalt mix, such as asphaltic concrete, aging only takes place in very thin surface layer.

The hardening of bitumen which occurs during mixing, handling and placing must be taken into account when designing the revetment. In general a loss of bitumen penetration of 10 to 25% is to be expected, depending on the type of mix-plant.

6.5.2 *'Stripping'*

Water can strip the bitumen from the surface of the mineral aggregate and cause the mix to deteriorate.
The danger of stripping is less when the voids ratio is low. From tests (12) it has been found that, for mixes with a voids ratio of less than 6% there is no danger of stripping. For mixes with a voids ratio of less than 10% there is little danger.
A method of measuring the deterioration of a mix is the 'Immersion Compression Test' which originated in American road engineering (39). In this test the compressive strength of blank samples is compared with those that have been kept under water of a certain temperature for a given period. The ratio found between the compressive strengths of the blank and immersed samples is called the retained stability.

6.5.3 *Resistance to erosion*

Flowing water can erode asphalt mixes and, especially when solid matter is carried along the extent of erosion as a result of impact forces on the asphalt surface can be substantial. The stresses in the asphalt caused by such impacts, will increase with the hardness of the binder. The resistance of the material to these stresses, however, is not directly proportional to the degree of hardness. From these considerations the following rule has been formulated:
The lower the minimum temperature at which erosion can be expected the softer the bitumen should be (12).

6.5.4 *Biological resistance*

Organisms can, to a greater or lesser extent, affect asphalt.
In the tidal zone damage is often found which has been caused by algae and seaweed. In higher zones of the slope certain plants can damage asphalt with their roots and runners (13).

– Algae damage (wire weed).
  As an algae layer dries out it shrinks and exerts a shear force on the revetment which can damage the asphalt surface, see Photo 1.
  This can be prevented by:
  1. A seal coat of bitumen, emulsion, road tar or tar bitumen. The latter two have the added advantage that they are very resistant to oil products but have the disadvantage that they can, to a certain extent, damage the environment.
     The surface of the revetment is made smooth by the seal coat and there is less

opportunity for algae to attach itself. In or near the tidal zone it is therefore better if the seal coat is not blinded with chippings or shell grit.

2. Bladder weed. A covering of bladder weed slows down the drying out process of the algae.
3. Chemical treatments. These methods, unfortunately, are environmentally unacceptable.

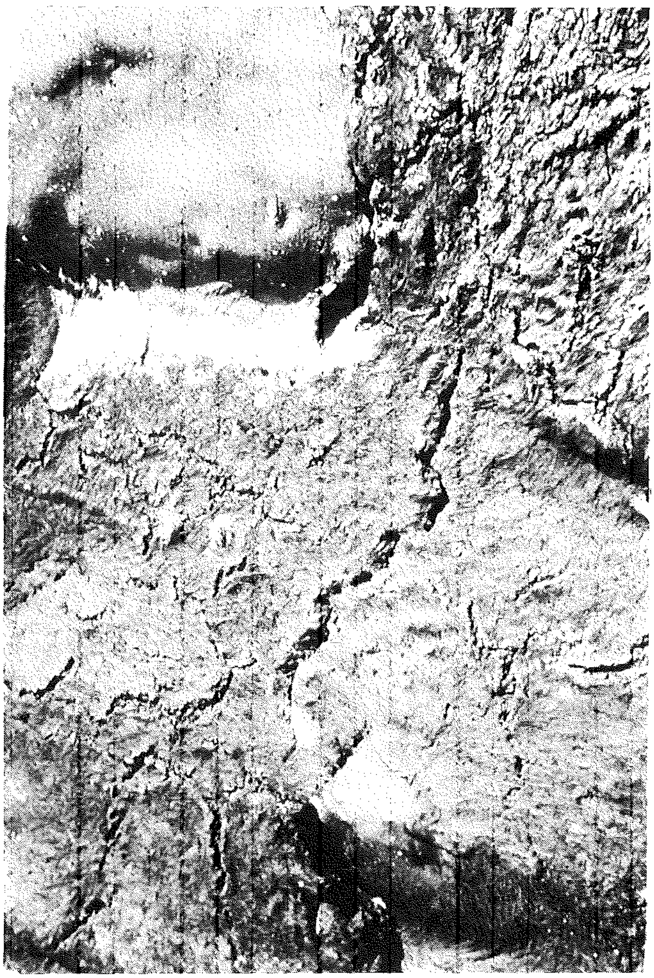


Photo 1 An asphaltic concrete revetment being attacked by algae

- Marine borers.

These animals which are to be found in the lower part of the tidal zone, exert during their growth pressure forces in the cracks and holes where they are established.

Mussels can then grow in the spaces which develop and so the damage progresses. This damage can be prevented by:

1. A seal coat as described above.
2. A thin layer of bitumen which kills the organisms.

- Plant damage.
  Plants can exert considerable force with their roots, and runners, see Photo 2.
  When there are seeds in the subsoil, plants can grow through the asphalt layer. This effect is strongly influenced by the type of subsoil; in dredged spoil there will be no or very few seeds. The environment can, also, be important, particular in salt or sweet water areas. It is possible to treat the soil with a planticide, but in this case the environment can be damaged.
  A smooth asphalt surface in which there are no cracks or holes offers very few attachment points for seeds which come onto the dike. A good seal coat can, therefore, be an important deterrent.

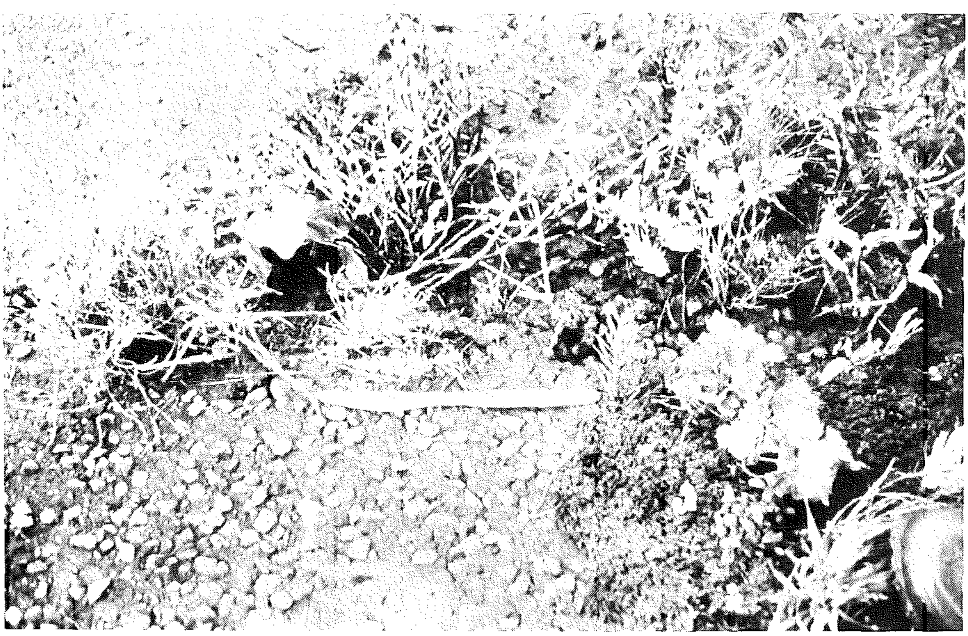


Photo 2 Plant growth damaging an asphaltic concrete revetment

6.5.5 *Chemical damage*

Bituminous materials are chemically inert apart from with some carbohydrates. The concentration of carbohydrates must be very high before they can cause real damage. A surface treatment of road tar or tar bitumen can be used for protection but this often is environmentally unacceptable.

6.5.6 *Other forms of damage*

Other forms of possible damage to the revetment are:
1. traffic, both during construction and operation;
2. vandalism, recreational activities, etc.;
3. floating debris;
4. vessels, anchors;
5. ice.

6.6 Workability

In order to obtain the best possible adhesion between the mineral aggregate and the bitumen the asphalt must be uniformly mixed. To obtain this the bitumen must have a low viscosity. This can be achieved by:

a. Heating the bitumen to 150-200°C (viscosity 150-300 mm^2/s); this is then referred to as 'hot-mix asphalt'. Hot-mix asphalt is prepared in an mix-plant in which the bitumen is heated to a specified viscosity and then mixed with dried and pre-heated mineral aggregate.
b. The bitumen can be diluted with a solvent, for example, a suitable petroleum distillate or can be emulsified in water; the solvent or the water has to vanish out of the material before the asphalt mix reaches stability.
   Under water or in a humid environment these methods are unsuitable. For this reason they are rarely used in hydraulic structures, except as an emulsion for seal coats and tack coats. Also the danger of stripping is greater than with hot-mix asphalt.

After production the mix is transported to the site and placed. The specification for mix composition generally depends largely on the workability, since total mechanised handling and compaction, using heavy equipment, is generally not possible.
The viscosity of the mix required for handling is strongly dependent on the viscosity of the bitumen and, therefore, requirements for the temperature are given. Hot-mix asphalt must be placed at the correct temperature on account of the time needed for handling and to achieve an optimal compaction.
For thicker asphalt layers the cooling period will be longer and, therefore, more time is available for placing and compaction. In this respect the weather conditions are important. When mechanical compaction is involved it should be borne in mind that the outer and inner surfaces of the layer will be cool while the inside is still hot. The thicker the layer the stronger is this effect. The possibility of building up the revetment in several layers can then be considered.

Overfilled mixes need not be compacted but underfilled mixes can be compacted in order to reduce the voids ratio. This process can be achieved by either mechanical compaction or compaction under the own weight of the material.

Mechanical compaction can be carried out by tamping, or by using rollers or vibrators. The degree of compaction can be affected by:
- the bitumen content;
- the content and type of filler;
- the mineral aggregate content;
- the particle shape of the mineral aggregate;
- the particle size distribution;
- the temperature of the asphalt and the surroundings;
- the layer thickness;
- the weight and type of compaction equipment.

Compaction is more difficult on steep slopes; the component of the weight of the roller vertical to the slope is smaller and the handling of equipment is more complicated.
In order to neutralize the roller weight parallel to the slope, the equipment can be attached to winches located on the crest. Compaction can be carried out effectively, without these special provisions on slopes more flat than 1 : 4, see also Section 22.3.1.
From an aesthetic point of view a smooth finish to the revetment is desirable. Sometimes, however, there is a tendency to prolong the compaction process in an attempt to obtain a certain smoothness. When this happens the asphalt can become too cool and (initial) cracks can develop. Rolling must be stopped if the asphalt becomes 'live', that is to say, if waves in the asphalt propagate in front of the roller.
If the mix is not very workable (initial) cracks can develop after, only a few passes of the roller, especially at the bottom of the slope where the material may start to flow. Rolling must then be halted to allow the material to cool down. At the same time the mix composition should be adjusted.
Generally it seems that the compactability is reduced as the stability of the mix is increased.

Compaction due to the weight of the material applies with what are known as, gap-graded mixes. These are mixes for which there is a 'gap' in the mineral aggregate grading curve; a particular particle fraction, usually the fine stone fraction, is missing. The coarse stone fraction forms the stable skeleton of which the voids are completely filled with sand mastic – in the case of a dense asphalt mix – or only coated in the case of an open asphalt mix.
The object is to obtain a mix which draws on the skeleton of the coarser stone to provide satisfactory stability against flow during handling and which, thanks to the viscous mortar, needs only a small amount of compaction energy to produce the required voids ratio. The stability of the mix in the completed stage must also be taken into account. This may possibly be in contradiction to the foregoing requirements.

Because they need not be compacted gap-graded mixes can be used at locations where compaction would be difficult.

6.7 Environmental aspects

The following points relate to the possible damaging effect of asphalt to the environment:

- The binder in the mix is generally bitumen.
  Normal straight-run or blown bitumen contains polycyclic aromatics (pca's) which are dangerous to health. The quantities involved are so small, however, that there is no danger to the environment, even when the bitumen is used in water catchment areas.
  Sometimes tar is used. This material contains, however, a much higher proportion of pca's than bitumen and generally is regarded as dangerous to the environment. Application in water catchment areas is not advisable.
  Bitumen is generally worked at high temperatures, and then vapour is given off which if inhaled excessively can lead to nausea. Under normal conditions vapour concentrations are below the danger level.
  Care must be taken for burns (82, 91).
- Mineral aggregates can also, depending on their place of origin, contain dangerous materials. An example is the use of fly ash originating in waste incinerators as filler. Normal sources in the Netherlands, however, have not caused significant problems.

7 Use of asphalt products in hydraulic structures

The most important mix types applied in hydraulic structures are:
1. Asphaltic concrete
2. Asphalt mastic
3. Grouting mortars
4. Dense stone asphalt
5. Open stone asphalt
6. Lean sand asphalt
7. Asphalt membranes

Mixes are defined by the constituents, the nature of asphalt mix and the binder material.

7.1 Asphaltic concrete

Asphaltic concrete is probably the best known mix type. It is a mixture of crushed stones or gravel, sand and filler in which the pores are practically completely filled with bitumen. The voids ratio is 3 to 6%.
In general the material must be compacted and is unsuitable for application under water or in the tidal zone. In view of the small voids ratio required, see Section 9.2.1., asphaltic concrete can be considered to be impermeable.
Asphaltic concrete is applied as a watertight dike revetment above the mean high water level, and as a lining for canals, reservoirs etc.

7.2 Mastic

Mastic is a mixture of sand, filler and bitumen. There is more bitumen available than necessary for filling the voids in the sand filler mixture. The mix, therefore, is naturally dense and need not be compacted. Mastic can be poured at working temperatures and is used, for asphalt slabs above and under water for lining or as bed and toe protection. When cold, mastic forms a viscous quasi-static mass.

7.3 Grouting mortars

Grouting mortars are hot-type mixes of sand, filler and bitumen of which there is more than required to fill the voids in the mineral; stone and gravel can be added if necessary. These mortars are used for grouing stone revetments above and below water-level, and also for slab construction.

7.4 Dense stone asphalt

Dense stone asphalt is a gap-graded mixture of stone, sand, filler and bitumen. The amount of bitumen slightly overfills the mixture. The material is, therefore, water impermeable.
It is used as bottom and slope protection and also in toe construction.

7.5 Open stone asphalt

Open stone asphalt is a gap-graded mixture of mastic and stone – a stone frequently used is limestone 20/40 mm. Mixing is carried out in two stages. First mastic is prepared and secondly it is mixed with limestone. The mastic binder only coats and connects the limestone particles together.
It is an 'underfilled' mix and, because of its open structure, should not be placed under water except in the form of prefabricated mattresses.

7.6 Lean sand asphalt

Lean sand asphalt is a mixture of sand, often locally obtained, with 3 to 5% bitumen. It is a greatly 'underfilled' mix and the function of the bitumen is simply to coat the sand grains and bind them together. After some time the permeability is very similar to the sand from which it is made.
It is used as a core material for reclamation bunds, filter layers and as permanent or temporary cover layer above and below water-level.

7.7 Membranes

Membranes are thin impermeable watertight layers of bitumen which are prepared in-situ or prefabricated. Membranes are used as impermeable linings for canals, banks and water courses.

8 Quality control

Quality control is essential in order to guarantee the quality of the work during the execution and on completion. Control includes mix design tests before the start of work, production control during the execution and checks on completion of a part or the complete work.
In road construction there are already many instructions and directives for carrying out quality control, for example, those given in Eisen 1978 (6), V.U.C.W. (18) and A.B.C.W. (19). There are, however, no such official instructions for the use of asphalt products in hydraulic structures. The construction requirements and the working conditions often appoint control checks required (20).

8.1 Mix-design tests

After the tender has been drawn up the definitive mix composition is decided, based on, (extensive) laboratory investigations. The type of investigation depends on the mix properties required and the application in prospect. Stability and voids ratio, for example, will be more relevant to asphaltic concrete revetments, whereas viscosity will be more relevant to a mastic slab.
During the investigations the suitability of the basic materials given in the tender is also examined.
The investigations should lead to a technical and economic optimum mix, taking into account the basic requirements and the materials available. Next the planned production and working methods are checked to confirm that they will yield the best results. The production method for the material and the execution method often determine the quality and cost of the work.

The mix-design tests on the basic materials are more or less the same for all asphalt mixes; they include, for example:
- an assessment of general information such as the type of material and identification of its source. The identification of its source must include the name of the supplier, location of the source, the nature of the material and the quantity for which the identification is valid. Each delivery of construction materials conform an identification of its source should be accompanied by a written authorization
- investigations into pollution aspects
- sieve analyses: stone/gravel; sand; filler
- determination of the bitumen number and sensitivity of the filler to water
- determination of the penetration and the softening point of the bitumen
- determination of the density of the mineral aggregate.

The design tests for the various mixes differ for each type of asphalt. Reference should be made to Part B, Chapters 9 to 15 inclusive.

8.2 Production control

Production control is necessary to guarantee the quality of the final product.
It includes:

1. Investigation of the quality of the construction materials, for example:
   - sieve analyses of:
     - stone/gravel
     - sand
     - filler
   - determination of the bitumen number, the voids ratio, and fraction <63 μm of the filler
   - determination of the penetration and softening point of the bitumen
   - determination of voids in sand/filler mixes, with the Engelsmann apparatus.
2. Check on composition of the mixture.
3. Check on properties of the mixture.
4. Check on production of the asphalt mix.
5. Check on handling, working and placing of the mix.

Production controls are normally carried out by the contractor. The principal can also carry out quality control during the execution of the work. In principle, however, from the very beginning, the principal should assume that the contractor has mastered the execution process and maintains good quality control. The control by the principal can include checking the methods used for production control. This can be in the form of tests on samples (by an independent organization). For these tests:

- the results should be available relatively quickly
- sampling and testing methods should be comparable to those used by other parties involved in the work
- supplementary agreements can be made concerning the test procedure.

The instructions, such as those given in A.B.C.W. (19), can be used a general guidelines for asphalt mix production control.

8.3 Acceptance control

The principal carries out acceptance control to confirm that the basic requirements have been satisfied. If the work has not been carried out satisfactorily then penalties should be imposed.
Acceptance control for road works are defined in Eisen 1978 (6) and V.U.C.W. (18). A part of this control can also be applied to hydraulic structures but obviously cannot be interpreted in exactly the same way.

Acceptance control is normally carried out after a part or all of the work is ready. If this is not possible the checks must be carried out during the work.
The parameters which are most frequently checked are given, per mix type, in Table 8.1.

8.4 Sampling

Good sampling technique is essential for reliable production and acceptance control. Samples are required of:

a. basic materials;
b. asphalt mixes.

8.4.1 *Construction materials*

Sampling and tests must be carried out sufficiently early so that results are available before the material is used.

GRAVEL, STONES AND SAND
Sampling of these materials should, for the time being, be according to the rules given in the Dutch standard NEN 3542. The sampling frequency is discussed in A.B.C.W. (19). Samples should be taken when the material arrives at the site, preferably while it is still in the transporter equipment or during unloading. Whilst there are no final standard rules on sampling, a specification is required for each construction.

FILLER
A.B.C.W. (19) can be used as a guideline on sampling frequency. Samples should be taken, on arrival, before the material is dumped in the silo. In order to verify the quality, samples can be taken out of the supply to the mixer.

BITUMEN
The sampling method should be as given in the Dutch standard NEN 3940 (21) and A.B.C.W. (19).

8.4.2 *Mixes*

Sampling is needed for:

- production control and to check the production control
- acceptance control.

Depending on the particular aspect being investigated, samples should be taken on one hand, at the mix-plant, and on the other hand, on site.

There are two methods for mixer sampling:

- at or in the skip
- out of the loaded truck.

The method of samples can be taken in accordance with A.B.C.W. (19) or any other systems specified in the particular tender.

The method of sampling, out of the completed work, is often dependent on the accessibility of the material in the structure and the possibility of obtaining samples with a saw or by taking cores. Usually cores are taken for road construction works, but this is not always possible for hydraulic structures. Taking cores is, if possible, in general the prefered method of sampling (Photo 3).

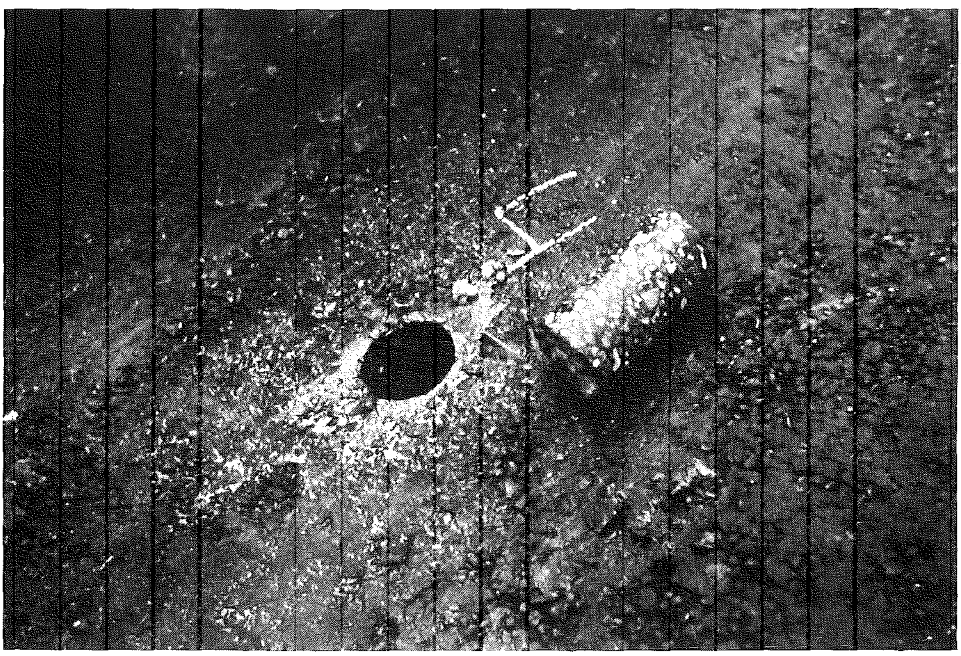


Photo 3 Taking a sample out of a asphaltic concrete revetment by core boring

Table 8.1. Completion checks for various asphalt mixes.

mix		Completion check: layer thickness	mix composition: bitumen content	mix composition: sieve analysis	mix composition: voids ratio	depth of penetration	sagging	joints	quantity of placed material/m²	sealing between different layers	possible sampling method[4]
asphaltic concrete		x	x	x	x						core boring
asphalt mastic		x[1]	x	x							core boring bulk sample
penetration mortar			x	x		x[2]					bulk sample
dense stone asphalt		x	x	x							core boring bulk sample
open stone asphalt	in-situ	x	x	x			x				core boring
	prefab.	x	x	x			x				bulk sample
lean sand-asphalt		x									core boring bulk sample
mem-brane	in-situ								x	x	bulk sample
	prefab.[3]	x						x			

[1] Check on layer thickness by core borings or by special thickness probes, also carried out under water.
[2] Desirable but often impossible to obtain.
[3] Testing of the specifications of the manufacturer;
Execution tests, specially joints;
Test on functional requirements:
- impermeability of the whole surface, including joints
- resistance to root growth
- ability to deform without leaks developing
- durability
- flexible joints with other structural components.

[4] Core borings are preferred; these, however, are not always possible. An alternative is bulk samples per quantity of in-situ material.

PART B

MATERIAL TECHNOLOGY

Summary

Part B discusses aspects of material technology of the various asphalt mixes which are most used in hydraulic structures.
These are:

- asphaltic concrete;
- asphalt mastic;
- grouting mortars;
- dense stone asphalt;
- open stone asphalt;
- lean sand asphalt;
- asphalt membranes.

To make this part of the manual more readily applicable it has been subdivided into the different asphalt types. So that separate sections can be read without cross-reference sometimes information has been repeated.
Three separate aspects have been discussed for each asphalt type, namely:
- The basic materials
  The materials, of which the asphalt type is composed, are discussed together with related construction parts.
- Mixes
  The composition of the asphalt and the way in which the mix is designed is described. The method of mix-design testing is also discussed. These are the investigations which must be carried out to determine the most suitable mix composition before the work can be started.
- Mix properties
  The choice of a particular asphalt type is based on the mix properties. Since the components and the composition vary, the properties of each mix type also vary. The relevant mix properties are described for each mix type. These properties form the basis for Part C of the manual which deals with the technical design.

9 Asphaltic concrete

9.1 Basic materials

9.1.1 *Crushed stone*

Crushed stone can be from local sources or imported.
Limestone gives a good adhesion, but can be expensive if not locally available.
The requirements for crushed stone, used in bituminous mixes for hydraulic structures, are given in Eisen 1978 (6).

9.1.2 *Gravel*

Because of the fact that gravel possibly gives less adhesion and the internal stability of the mix is lower in the hot condition, for some time the use of gravel was abandoned. Since the lower internal stability never has been proved sufficiently, however, there is, for the present, no reason not to use gravel.
Reference should be made to Eisen 1978 (6) as far as this is related to hydraulics.

9.1.3 *Sand*

In principe every kind of sand can be used. A well-graded sand is desirable since otherwise high percentages of filler must be added in order to reduce the voids ratio. This can lead to mixes which are difficult to handle and are more expensive.
A sand, that lies in the optimum area of the sand triangle, Figure 2.1, is well-graded and satisfies the requirements for Sand Type A, given in Eisen 1978 (6).
The following points relate to sands lying outside the optimum area:

- There is no experience with sand that lies to the right of the line for natural sand, see Figure 2.1. As far as it is known such natural sands do not occur in the Netherlands.
- Sands with a grading such as that in Area II in Figure 2.1 contain the finest fractions and are very uniformly graded. Because of this the mineral mix will have a high voids ratio and a relatively large amount of additional filler and bitumen would be required to produce a dense asphalt mix. As a result the workability would be adversely affected.
- Sands lying in the lower left hand corner of the sand triangle produce good results provided that the filler content, as indicated by a sand-filler compaction test (Engelsmann method), is increased. Then workable mixes are obtained with a low voids ratio.

– Sands laying in Area V of the sand triangle, Figure 2.1, can also produce mixes with good workability. The use of such sands, however, is rather expensive.

Stability during the construction phase can be improved if some of the natural sand is replaced by crushed sand; this is a more angular material. On the other hand the workability can be influenced adversely by this.

9.1.4 *Filler*

'Very weak' fillers are used in hydraulic structures, being fillers with a very low voids ratio which require relatively very little bitumen. Ideally limestone fillers should be used because they give better adhesion in a humid environment.
From experience it appears that mix workability is increased by using 'weak' rather than 'very weak' filler, while there is no noticeable affect on the required voids ratio.
For further information, reference should be made to Eisen 1978 (6).

9.1.5 *Bitumen*

Concerning the required flexibility of the revetment it has been found that bitumen 80/100 pen is the most suitable in the Netherlands. For bitumen requirements, see Eisen 1978 (6).

9.1.6 *Other constructional aspects*

Asphaltic concrete is, preferably, covered by a seal coat. This can consist of a sealing layer and/or a surface dressing. The function of the seal coat is to:

1. cover surface cracks and open texture;
2. limit temperature affects due to the sun;
3. improve the aesthetic appearance.

The surface dressing comprises a sprayed layer of bitumen emulsion, Type Unstable, about 1.0 kg/m^2, which, above the tidal zone is blinded with chippings, for example, 10 to 12 kg/m^2 of size fraction 4/8 mm, or shell grit (16).
The seal coat is preferably preceded by a protection layer which must be applied as quickly as possible after the asphaltic concrete has been placed. This layer comprises sprayed bitumen emulsion, Type Unstable, about 1.0 kg/m^2. For aesthetic and practical reasons it is recommended that the emulsion is sprayed from the top of the slope downwards.

9.2 Mixes

9.2.1 *Mix design*

The asphaltic concrete used in hydraulic structures is a hot-prepared mix of crushed stone or gravel, sand, 'weak' or 'very weak' filler and bitumen 80/100 pen. The object is to design a mix which has the lowest possible voids ratio and still is sufficiently stable. The voids ratio of the mix depends, irrespective of compaction, on the voids ratio of the mineral aggregate and the bitumen content.
It appears that the minimum voids ratio in the sand-stone mixture, without filler, is between 50 and 60% by mass of normal crushed stone or gravel.
For a sand-filler mix there is also a minimum voids ratio for a certain sand-filler proportion (Figure 9.1).

In principle the filler content can be chosen so that the voids ratio in the sand-filler mix is minimal. In practice, however, for economic reasons, a lower filler content of 7 to 8% m, of the complete asphalt mix, is often chosen. Around the minimum voids ratio value changes in the filler content have only a limited effects on the voids ratio (Figure 9.1). It is also practical to choose a filler content somewhat to the left of the minimum since filler contents to the right of the minimum value shown on the curve result in an undesirable increase to the volume of the voids.

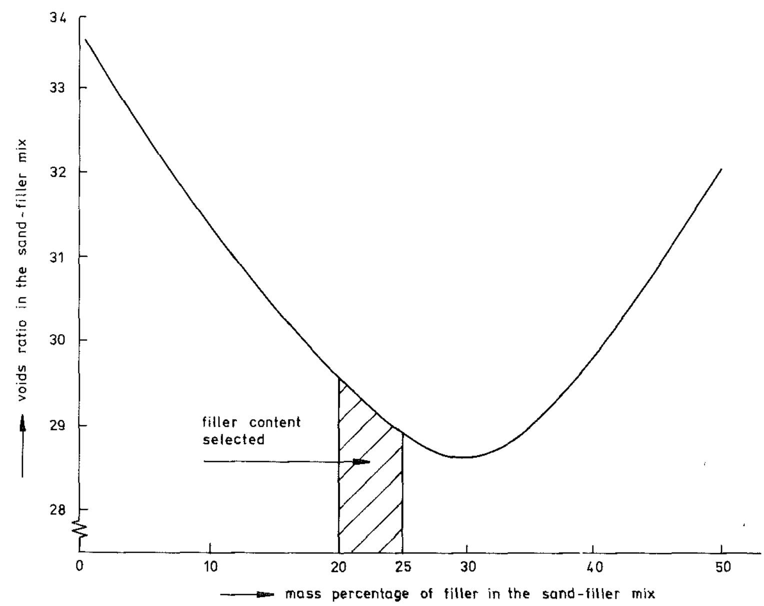


Figure 9.1 Relationship between voids ratio in a sand-filler mix and the filler content in the mix.

The composition of the mineral aggregate content is determined in the way described above.
The bitumen content selected depends on:

1. The form of the curves which relate voids ratio of the mix to the bitumen content, see Figure 9.2.
2. The required voids ratio compared with the voids ratio found in Marshall samples, compacted by 50 to 100 strokes on one side, see Figure 9.2.

Whether or not a mixture with the chosen bitumen content is also suitable to be placed and compacted, also on slopes, can only be found out in practise. Only limited information can be obtained about this aspect by doing slope tests in the laboratory.
With asphaltic concrete used in hydraulic structures, containing for example, Sand A and/or fine sand it appears that increasing the bitumen content, and thus lowering the viscosity, lowers the voids ratio to a certain limiting value. A mix with a bitumen content of 7.5% m in 100% m mineral is difficult to handle because it is insufficiently stable in the hot condition.

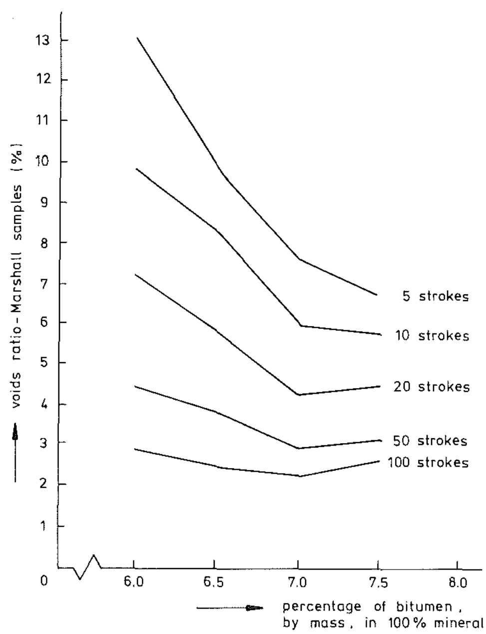


Figure 9.2 One-sided compaction-Marshall samples (16).

Investigations by the Commissie Verdichting Asfaltdijktaluds (Dutch commission for the compaction of asphalt dike revetments) (16) indicate the following suitable mix composition (see also Eisen 1978):

- Asphaltic concrete for hydraulic structures should be a hot-prepared mix of crushed stone 6/16 mm, sand, very weak filler and bitumen 80/100 pen for layers less than or equal to 150 mm thick. For thicker layer crushed stone 6/22 mm should be used.
  In practice, in the last few years, a bitumen of 6.5% m has often been used to improve the workability, especially the stability in the hot condition.
  When asphaltic concrete is in place the voids ratio should not be greater than 6% m on slopes steeper than 1 : 4 and not greater than 5% m on slopes of 1 : 4 or less.

Table 9.1 Asphaltic concrete mix composition used in hydraulic structures.

	% by mass			
thru sieve on sieve	required	min.	max.	tolerance
C 22.4 2 mm	50	48	55	
2 mm 63 μm	42	37	47	±5
63 μm —	8	7	10	±1
bitumen content in 100% mineral	7	6.5	7.5	±0.5

The voids ratios above give, in general, mixes which are durable and have good workability.
In practice often for aesthetic reasons, mixes are compacted too long and too heavily so that initial cracks develop (59, 89). This can normally be prevented by using a good compaction method, see Section 22.3.1. If the required voids ratio is so small or the layer thickness is so large that cracking cannot be avoided even with a good compaction method then the revetment must be laid in several layers. In general, the thinner the layer the better the compaction. Naturally good cohesion must then be provided between the layers. This can lead to problems during execution if dry sands blows onto the surface being prepared. Dry sand and other pollutants must, therefore, first be removed. At locations where dry sand is not a problem the method using several layers produces good results and is to be recommended when compaction problems are anticipated. The layers can be built up of the same material or from different materials which are matched. For this aspect reference should be made to the various cover layer constructions used for dams and roadworks.
For conditions in the Netherlands the first solution is often adequate.

9.2.2 *Mix-design tests*

In order to find suitable mix compositions with a voids ratio as low as possible and which are sufficiently stable when cold it is necessary to carry out laboratory tests.
When these two criteria cannot be satisfied in combination, generally the voids ratio requirement is given first priority. In practice, as far as it is known, stability problems under normal circumstances have never occurred.

In Eisen 1978 only the investigation of voids ratio is required; after compaction according to a Marshall test, the voids ratio should be not more than 4%. In order to achieve better compaction results and to have more time available for compaction the mix should be such that it can be compacted at the highest possible temperatures.
Mix-design tests can include a number of sections, such as:

- investigation of the voids ratio of the sand-filler mix with, for example, the Engelsmann apparatus;
- determination of the voids ratio of the asphaltic concrete mix;
- comparative investigations by Marshall tests.

Mix properties can be investigated by varying the mass percentages of the various components within the given limits. Normally, however, only the effect of the bitumen content is investigated.

From the results of these investigations the mix can be designed. When the mix has been decided the influence of the tolerances should be investigated, for example by carrying out two tests on mixes with the designed mineral composition; one with 0.5% by mass more bitumen and one with 0.5% by mass less bitumen. If it appears that problems may develop then the mix composition should be modified.
A written statement must be given of the composition of the asphalt to be delivered to the site. The tolerances, for use with this composition, as given in Table 9.1, should also be declared. If it appears that the composition of the asphalt has to be changed during the execution, then these changes should be reported in writing and the tolerances should be restated.
Reference should be made to Section 8.1 for the investigations to be carried out on the basic materials.

9.3 Mix properties

9.3.1 *Mechanical properties*

During mixing and handling the bitumen hardens and, as a result, the penetration decreases and the softening point increases (6). In order to determine exactly the extent of this effect a sample of the bitumen must be recovered.
The stiffness modulus of the bitumen can be determined from the Van de Poel nomogram, see Figure 6.1. The penetration-index and the softening point of the recovered bitumen and the related loading period and temperature determine the value of the stiffness modulus. The Poisson ratio of the asphalt mix can be estimated using Figure 6.6. The stiffness modulus of the mix can be estimated with Figure 6.3. for values of $S_{bit} > 10^6\ N/m^2$ and $S_{mix} > 10^8 N/m^2$. Preferably, however, the S-modulus should be determined by experiment.

Figure 6.4. gives general values for the initial strain at failure. This value can also be determined for each stiffness of modulus by experiments.
Since the Marshall test only gives an indication of the stability and flow properties of mixes it is only usable for comparison purposes, for example, in connection with construction and control.

9.3.2 *Permeability*

Permeability is largely dependent of the voids of ratio.
It appears that, for voids ratio below 3%, no water can be forced through a test plate 5 cm thick, under a pressure of 3 atm. For mixes with voids ratios of about 8% the permeability coefficient is about 10^{-8} m/s (12).
Usually the voids ratio of asphaltic concrete, used for slope revetments is between 5 and 6% and the material considered to be water impermeable. If this permeability does not appear to give a satisfactory seal then water permeability tests should be undertaken.

9.3.3 *Durability*

AGING AND HARDENING
During mixing and handling, the bitumen hardens, a phenomenon which can be measured using a thermostability test, see Chapter 3.
Changes in the penetration can vary from 10 to 25%, depending on the type of mix plant and size of the batch. The conditions set out in Eisen 1978 (6) should be adhered too. In view of the relatively low voids ratio and the high density of asphaltic concrete, aging of the bitumen, during the lifetime of the revetment, only takes place in a very thin surface layer; the phenomena is, therefore, of little importance. It can be reduced even further by a seal coat.

LOSS OF ADHESION DUE TO WATER; 'STRIPPING'
With a dense mix such as asphaltic concrete there is little danger of stripping especially if the revetment is given a seal coat.

CHEMICAL RESISTANCE
Bituminous materials are chemically resistant except to a limited number of carbohydrates. Before these can cause any damage to a dense asphaltic concrete mix the concentrations in the surface water must be very high. A seal coat of road tar or tar-bitumen can limit the effects. The environmental aspects, discussed in Section 6.7, are then also relevant here.

BIOLOGICAL RESISTANCE
See Section 6.5.4.

10 Asphalt mastic

10.1 Basic materials

10.1.1 *Sand*

Provided the mix properties are satisfactory, in principle, all sand types can be used. For the sand requirements reference should be made to Eisen 1978 (6).

10.1.2 *Filler*

Filler should be weak or very weak. Since the filler strongly determines the viscosity of the mastic it should have a uniform quality and a constant absorption capacity. See also Eisen 1978 (6).

10.1.3 *Bitumen*

The type and quantity of bitumen greatly influence the viscosity of the mastic. A bitumen which is no harder than pen 80/100 is recommended. See also Eisen 1978 (6).

10.1.4 *Other constituents*

Polymers are sometimes added to restrict flow in cold conditions. It is advisable, in such instances, to use a different mix composition.

10.2 Mixes

10.2.1 *Mix design*

Mastic is a mix of 60 to 70% by mass sand, 15 to 25% by mass filler and about 20% by mass bitumen. It is an overfilled mix which, in itself, has the properties of a highly viscous fluid. Mastic mix design is based, therefore, not on stability but on viscosity.
Mastic requirements, see (23), include:

1. The mix must be pourable at high temperatures. This depends on the viscosity at placing temperatures; in practice the viscosity is between 10 and 200 Pa . s.
   The mastic must have such a viscosity that, for example, it can be poured through a pipe.
   If placing temperatures are too high vapour-filled bubbles develop in the material when it is applied under water. These subsequently heal by mastic flow.

Calculations show that, depending on the pipe and nozzle diameters, the mastic viscosity should not be higher than 200 Pa . s for it to flow through a vertical pipe under its own weight and without the excess pressure becoming too large (23), see also Appendix VIII. On the other hand, however, the viscosity must not be so low that water can intrude into the nozzle. Naturally both upper and lower limits to viscosity depend on the water depth.

When working with a closed hopper bucket or a crane the viscosity should be 30 to 100 Pa . s under water and 50 to 200 Pa . s above water. The viscosity should not be so low that, on slopes of 1 : 10 above water and 1 : 7 under water, excessive hot flow develop down the slope. On the other hand the flow should be sufficient to form a continuous layer.

A viscosity between 80 and 150 Pa . s at the placing temperature is sufficient to ensure this. (see Table 10.1).

2. After cooling down to normal temperatures the mastic viscosity should be sufficiently high to ensure that it only flows within particular limits. For this temperature, slope and layer thickness have to be taken in account. From calculations it appears that, for a 10 cm thick plate on a slope of 1 : 10 to flow a few decimetres in 7 years, the viscosity would have to be at least 10^9 Pa . s, see also Appendix IX. (See Table 10.1).

Table 10.1 Required asphalt mastic viscosities at various temperature stage.

Asphalt mastic		Required viscosity (Pa . s) min.	max.
In the equipment:	170-100 (°C)		
pipe		30	150
bucket, crane: under water		30	150
above water		50	200
During execution: hot flow of a plate on a slope of 1 : 10	170-100 (°C)	80	1000
After execution: cold flow of a plate	10 (°C)	10^9-10^{10}	

Some examples of mastic mixes used in the Netherlands for slab construction are:

– Grevelingen Dam
  - sand 63-65% by mass
  - filler 15-17% by mass
  - bitumen 280/320 20%

– Eastern Scheldt Storm Surge Barrier

• sand	62 %	by mass	sand	64 %	by mass
• filler	20.5%	by mass	filler	17.5%	by mass
• bitumen 160/210	17.5%	by mass	bitumen 80/200	18.5%	by mass

Viscosity at 130°C, 30 to Pa . s.

10.2.2 *Mix-design tests*

Mix-design tests to determine the mix composition are mostly based on the Kerkhoven Method.
First of all the optimum sand/filler proportion is found using the Engelsmann apparatus and based on this value a somewhat smaller filler content is selected, see Section 9.2.1. The tests with the Engelsmann apparatus are carried out to determine the smallest voids ratio in the sand/filler mix in order to ensure that the related bitumen content is the smallest possible. The bitumen content is determined from the Vsf-factor, which, in its turn, is found from Figure 10.4., see Section 10.3.1.
The viscosity at high temperatures, 120 to 170°C, is found using the Kerkhoven viscosity meter (Figure 10.1.). With this meter, the time taken for one litre of mastic to flow out is measured, in seconds.

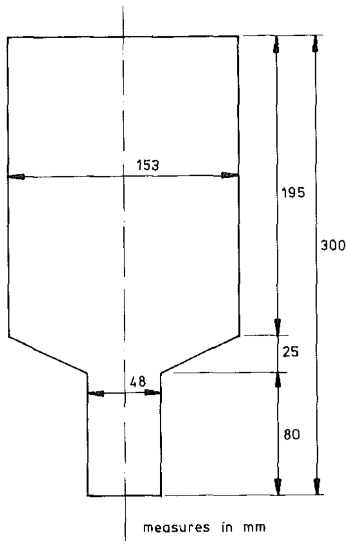


Figure 10.1 Kerkhoven viscosity apparatus.

Since the flow is laminar the viscosity can be calculated using:

$$\eta = 43.5 \cdot 10^{-5} \rho g t \text{ (for the type of viscosimeter shown in Figure 10.1)}$$

in which:

η = viscosity (Pa . s)
ρ = bulk density (kg/m^3)
g = acceleration due to gravity (m/s^2)
t = outflow time (s)

The following can be investigated:

1. the influence of variations in mix composition within given limits, on the mastic properties.
2. the influence of variations in the grading of the sand on the mastic properties.

In practice the sensitivity to temperatures (Figure 10.2), of the mix is important in connection with the execution. This property is determined by the mix composition and the properties of the components.

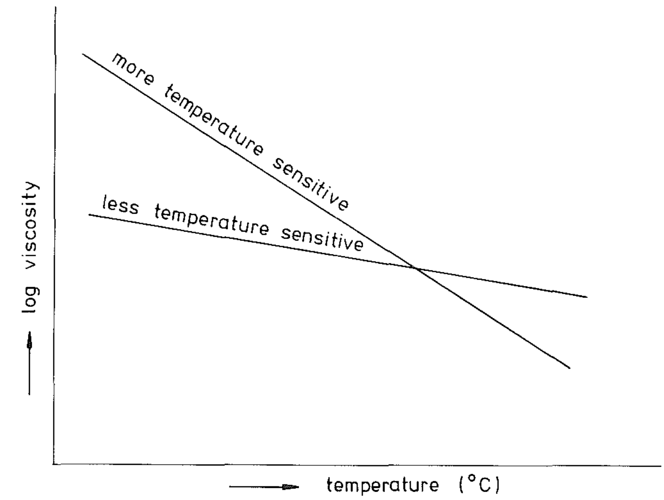


Figure 10.2 Viscosity of a mastic mix related to temperature.

Remarks

The Kerkhoven method of mix design has a number of limitations:

- the tests give only the viscosities at the test temperatures and not at the normal temperatures
- the measuring accuracy is not very great.

The designer and the user should have a more usable method to supplement the Kerkhoven method. Research into this is almost completed.

10.3 Mix properties

10.3.1 *Viscosity*

Measurements of mastic viscosity have been made by Kerkhoven (22). With this the measured relative viscosity, η_m/η_{bit} (η_m = viscosity of the mix, η_{bit} = viscosity of the bitumen) is related to the bulk volume of the sand/filler mix (Vsf).

The Vsf can be determined using:

$$\text{Vsf} = \frac{100}{100 - \text{Hm}} \cdot \frac{F + M}{F + M + B} \cdot 100\%$$

in which:

Hm = voids ratio of the compacted sand/filler mix
F = volume of filler
M = volume of sand
B = volume of bitumen

The Vsf represents the solid phase of the mix, that is, the bulk volume of the sand/filler mix in which the voids are filled with fixed bitumen, see Figure 10.3.

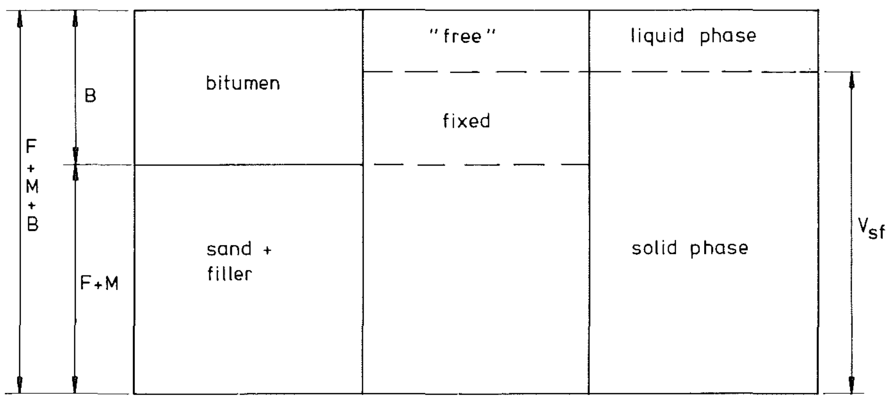


Figure 10.3 The different phases of a mastic mix.

Measurements have been made, using the Kerkhoven method, on a large number of mastic samples made up from four different sand types, including dune and river sand, mixed in different proportions with filler and bitumen 50/60 and tested at 140°C and 40°C (22).
Kerkhoven used the Marshall compaction hammer to compact the sand/filler mix, a method which produced a lower voids ratio than the Engelsmann apparatus. The latter method, however, gives better agreement with values obtained in practice (4), see Figure 10.4.
It appears that the viscosity of a mastic, as a first approximation, can be determined using the Vsf formula. A simple dry compaction test, based on Engelsmann, can then be used to determine the voids ratio of the mineral aggregate. The relative volumetric proportion of filler to bitumen only influences the viscosity at higher temperatures and only then when the proportion is 0.20 to 0.25 or less. These low proportions are, in fact, seldom

found. With a low relative volume of filler to bitumen it is possible to design a mix with a relatively low viscosity at working temperatures and a high viscosity at normal temperatures.

$$F/B = \frac{\text{Volume of filler}}{\text{Volume of bitumen}}$$

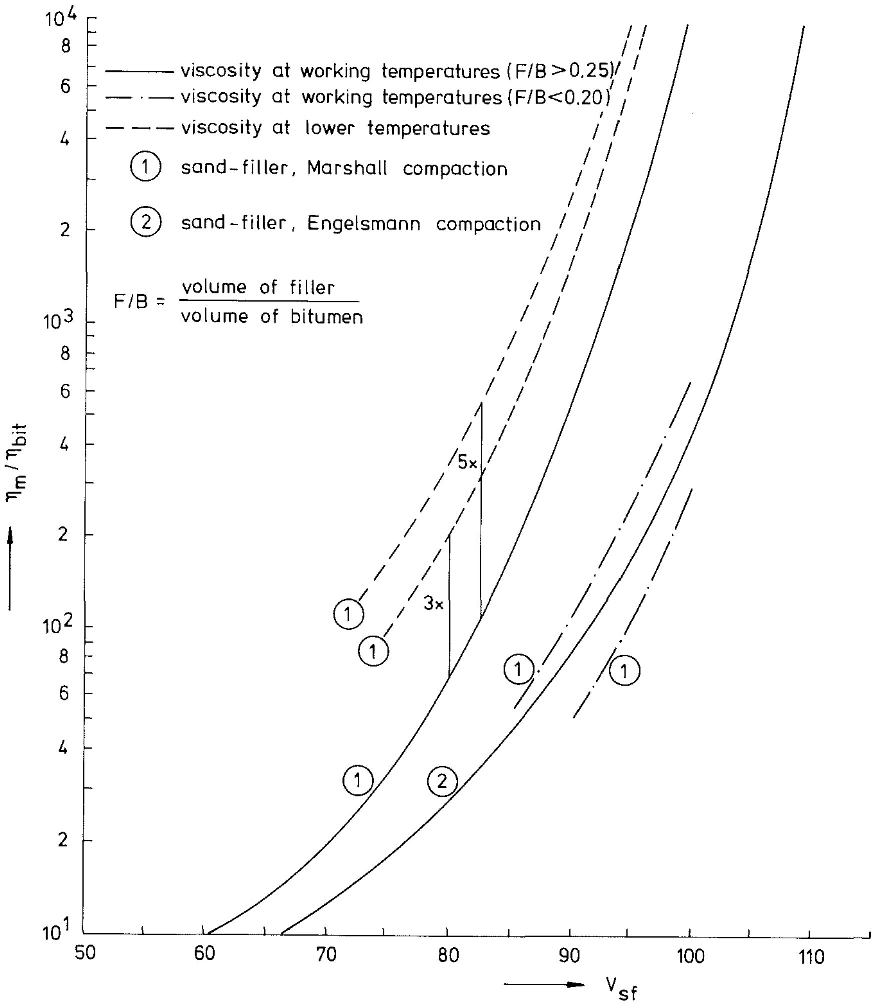


Figure 10.4 The effect of Vsf on the viscosity of mastic mixes.

The viscosities at working and normal temperatures are very closely related. An easily workable mix will, in general, have greater flow at low temperatures. The flow characteristics of mixes at normal temperatures, therefore, can only be compared when the mixes have the same viscosities at working temperatures.

10.3.2 *Mechanical properties*

The stiffness modulus and the strain at break of a mastic mix can be determined, very generally, using the methods discussed in Sections 6.3.1 and 6.3.2.

10.3.3 *Permeability*

Mastic is an overfilled mix and is, thus, dense. Air bubbles can form in the mix as a result of the preparation process which generally do not affect the impermeability. There may well be a voids ratio of 7 to 12% but the voids are not in contact with each other.
Air bubbles can, however, have an effect on the viscosity. An increase in the air content can cause a reduction of the relative viscosity.

10.3.4 *Durability*

Because the mastic is overfilled, the mix is, in itself, sealed and therefore very durable.

11 Grouting mortars

Three ways in which an asphalt mortar can be used to grout stone are:

1. Surface grouting. This is also referred to by the German term 'Verklammerung'. In this method a certain quantity of grouting mortar is applied uniformly to the whole surface, see Figure 11.1a. The construction is not completely sealed since the mortar only penetrates the surface layer.
2. Pattern/partial grouting. In this method the whole layer of stone is penetrated on a predetermined pattern. As a result, in fact, lumps of stone are formed, see Figure 11.1b.
3. Fully grouting. In this method the voids in the stone are completely filled with grouting mortar forming a continuous homogeneous revetment, see Figure 11.1c.

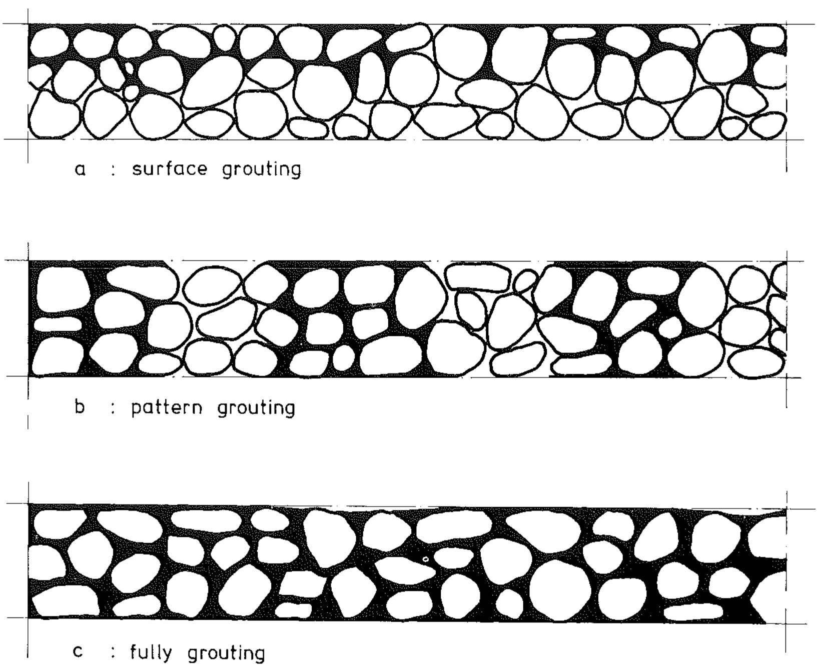


Figure 11.1 Different methods of grouting.

11.1 Basic materials

11.1.1 *Sand, filler, bitumen*

See Chapter 10, Mastic.

11.1.2 *Gravel, stone*

See Eisen 1978 (6).

11.1.3 *Other constructional aspects*

1. The grouted stone

Crushed stone is normally specified by the mass of the 'smallest', and 'largest' stone size, for example quarry-run 10/60 kg.
The stone must, in the first place, satisfy the general conditions for dumped stone (49):

- it must be sufficiently hard;
- it must be frost/weather resistant;
- it must have less than 3% by mass below the minimum stated size or, when it is to be dumped under water, less than 1%;
- there must be no flat pieces; the relationship between the breadth and the length, and the heigth and length must be larger than 1/3;
- it must be preferably uniformly graded;
- there must be very little breakage during transport and handling.

An important aspect to be considered is the effect of different properties such as stone size distribution and the percentage of flat stones, and the viscosity of the asphalt mortar on the grouting depth and spreading.

2. Filters

A sand and watertight construction cannot always be guaranteed even with fully grouted stone and, therefore, also in relation to the execution of the work a filter is recommended. When a filter cloth is used the temperature of the grouting mortar, which will touch the cloth, should not be too high. For a polypropylene cloth the maximum temperature is about 140°C.

11.2 Mixes

11.2.1 *Mix-design*

A grouting mortar is an asphalt mastic or an homogeneous mix of mastic with gravel or stone. The use of gravel or stone is dependent on the dimensions of the stone to be penetrated. Gravel or stone serve to:

1. reduce the quantity of mastic needed;
2. prevent the mortar sagging onto the slope;
3. limit, sometimes, the grouting depth.

Mix-design is often a question of experience. During execution the mix viscosity should be sufficiently low, in the range 30 to 80 Pa . s, for the mortar to be handled and the stone grouted without it sagging onto the slope.
After execution the viscosity should be so high that flow at ambient temperatures is limited. Calculations indicate that, to reduce the flow of mastic in rubble 80/100 kg, on a slope of 1 : 5 in a 7 year period to a minimum, a mortar viscosity of 10^9 Pa . s is needed. In general the viscosity required will lie between 10^6 and 10^9 Pa . s, depending on the temperature, slope angle and dimensions of the stone (23).
The flow properties depend on the composition of the grouting mortar in relation to the dimensions of the stone and the slope-angle. If the flow of the hot mastic in the voids between the stones is regarded as laminar flow through a pipe, the following relationship can be used to find the grouting depth (78):

$$l = \frac{Cd^4}{\eta_0}$$

in which:

l = grouting depth (m)
d = specific stone size, d_{20} (m)
η_0 = initial viscosity of the mastic (Pa . s)
C = constant, determined by experiment (Ns/m^5)

The design of the grouting mortar is directly related to the weight and dimensions of the stone to be grouted. The type of application is also important, for example, when grouting is to be carried out under water since the mix cools down quickly and the time available for grouting is shorter. Experience gained when constructing the breakwaters at IJmuiden indicates that, in order to obtain a grouting depth of two layers of stone, the relationship between the maximum particle size in the mix, d_{85}, and the minimum particle size of the stone d_{15}, should be (27):
– above water-level, d_{15}/d_{85} = 5 to 10;
– below water-level, d_{15}/d_{85} = 10 to 20.

The design of the mastic is carried out in the way described under mix design tests, see Section 10.3.1. The optimum sand/filler proportion is determined from tests with the Engelsmann apparatus, the grouting depth required being achieved by varying the bitumen, gravel/rubble, sand and filler proportions, the mixing temperature and the bitumen hardness.
In order to achieve a satisfactory grouting depth a minimum of 50 to 55% mastic is

required in the mortar, depending on the particle size distribution of the gravel or stone and the other components (60).
The maximum particle size of the stone or gravel of the grouting mortar is determined by the mixing equipment to be used. For normal asphalt mix-plants the maximum particle size is 50 to 60 mm. For coarser materials special equipment is needed.
For good 'surface' penetration at least 1/3 of the voids of the revetment should be filled (the upper layer being completely filled) (37). For 'pattern' penetration 50 to 80% of the revetment surface should be covered. With fully grouting all the voids are filled.

A grouting mortar has the tendency to segregate, see Figure 11.2. When used for slab construction it is essential to distinguish between the properties of the mastic and the properties of the segregated grouting mortar.
The performance of a segregated grouting mortar is determined by the mineral particle skeleton and the degree of self-compaction. If this compaction is limited the mastic properties predominate. With good compaction, however, it is the particle skeleton which predominates.

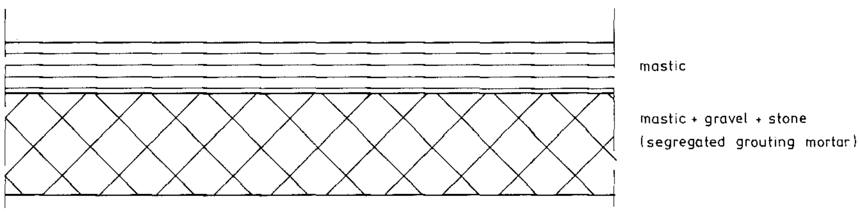


Figure 11.2 Segregated grouting mortar.

As an example a viscosity of 40 to 80 Pa · s is required for grouting stone 5/40 kg. A grouting mortar which is often used is given in Table 11.1.

Table 11.1 Grouting mortar often used for stone 5/40 kg.

	by mass %
gravel 4/16	30
sand A	61
very weak filler	9
bitumen 80/100 on 100% mineral	

11.2.2 *Mix design tests*

Mix design tests, in general, include:

1. An investigation of construction materials, see Section 8.1.
2. Laboratory investigations of the mix, the mastic, see Section 10.2.2, which includes, the effect of variations in mix composition, within given limits, on the properties of the asphalt mortar.

11.3 Mix properties

11.3.1 *Stability*

A grouting mortar should be so stable that:

1. Any subsequent grouting, when cold, is limited to a minimum.
2. The mix is not sagged onto the slope.

By adding crushed stone or gravel a good quality mix can be produced which has only a limited cold flow. This can be investigated by large scale slope tests. These tests should be of sufficiently long duration and the test site well chosen.
Above water-level slopes of upto 1 : 1.7 can be grouted, below water-level the maximum slope is 1 : 3. By grouting carried out in layers these slopes can be increased to 1 : 1 and 1 : 1.5.

11.3.2 *Viscosity*

The viscosity of grouting mortar is determined in the same way as the viscosity of mastic, see Section 10.3.1. The working viscosity will, in general lie between 10 and 100 Pa . s.

11.3.3 *Permeability*

Grouting mortar, being overfilled, is, in itself impermeable. Permeability of the revetment is dependant on the degree of penetration. One can only speak of 'impermeable', however, in relation to fully grouting and even this cannot be guaranteed in all cases. When designing for hydrostatic uplift complete water impermeability should be assumed.

When a permeable revetment is required, then preferably not more than 70% of the crushed stone surface should be grouted. Good mix composition and execution is essential to ensure that the grouting mortar does not remain in the top layer or sags and thus forms a dense layer.

11.3.4 *Durability*

The durability of the grouting mortar is determined by that of the stone and that of the mortar itself. For the stone the requirement is that it is, in itself, durable (49). For the durability of the mortar reference should be made to Section 10.3.4.

12 Dense stone asphalt

12.1 Basic materials

12.1.1 *Crushed stone, gravel*

Since dense stone asphalt is a slightly overfilled mix it is, on the whole, not important what kind of stone is used. Good adhesion is obtained with limestone.
Limestone 20/40 mm is often used. Use of this size is based on the fact that:

- it is the maximum size which can be used in conventional asphalt mixers (maximum about 60 mm);
- it is a type which is generally available;
- it is suitable for the layer thicknesses typically used;

The stone or gravel requirements are based on (49):

- the oversize;
- the undersize;
- the proportion of filler;
- the quantity of flat pieces;
- the frost and weather resistance;
- hardness;
- purity;
- a uniform grading;

For further details, see Eisen 1978 (6).
The stability of very thick layers, both during the hot and cold phase is increased by using heavier stones. A dense stone asphalt with stones of a maximum 10/60 kg was for example applied for the construction of the breakwaters at IJmuiden. A special mix-plant is then required.

12.1.2 *Sand, filler and bitumen*

See Section 10.1.

12.2 Mixes

12.2.1 *Mix design*

Dense stone asphalt is a slightly overfilled mix with some gap-grading, a result of which

is that compaction takes places under its own weight. Gap-grading is important for underwater applications where the cooling phase and, thus the time available for placing the mix is relatively short.
The ability to form a dense stable revetment by self-compaction depends on:

- the viscosity of the asphalt mortar (mastic or grouting mortar);
- the proportion of stones to asphalt mortar.

The stones form a skeleton which gives a certain stability to the mix in the hot phase, while the mortar has a binding and load-spreading function. The viscosity of the mortar is important in both the hot and cold phases.

- A relatively low viscosity is essential at working temperatures to ensure that the stone asphalt has good workability.
- After cooling down the viscosity must be relatively high in order to prevent any excessive cold flow.

Generally, depending on stone size and grading, the mix comprises 50 to 70% by mass stone and 50 to 30% by mass mortar. With lower stone contents a stiffer mortar should be used. For mortar design, see Section 10.2 and 11.2.
The layer thickness is also a determining factor in the choice of stone size.
Examples of a few dense stone asphalt mixes used are given in Table 12.1.

Table 12.1 Percentages by mass of dense stone asphalt and mortar mixes used for the Eastern Scheldt storm surge barrier and IJmuiden breakwaters.

		IJmuiden	
	Eastern Scheldt	below water	above water
Asphalt mix (mass %)			
– limestone 10/60 kg	–	46.0	–
– limestone 1/10 kg	–	20.0	74.5
– limestone 20/40 mm	60.0	–	–
– mortar	40.0	34.0	25.5
mortar (mass %)			
– well-graded mineral	–	66.0	67.0
– local sand	64.0	–	–
– very weak filler	17.0	15.0	15.5
– asphalt bitumen 80/100	19.0	19.0	17.5

12.2.2 *Mix design tests*

Mix design tests can include:

- Investigations of basic materials, see Section 8.1.
- Investigations of mix composition, production and execution.

The laboratory design of the mix, generally is as follows:

- The stone size is selected, in principle, based on the layer thickness required. A number of mixes are then made with various stone contents and different mortar compositions and, thus, different viscosities. If the stone size is too large for laboratory investigation then tests can be made at the mix-plant.

In order to obtain an impression of the hot-flow characteristics slope tests are carried out at the required slope.
For asphalt mortar investigations reference should be made to Sections 10.2.1 en 11.2.1.

12.3 Mix properties

12.3.1 *Stability and viscosity*

Dense stone asphalt is an overfilled mix of asphalt mortar (mastic with or without crushed stone or gravel) and crushed stone which gets its stability from a limited amount of segregation.
The stability and workability are, in fact, two contrary properties which are dependent on:

- stone content;
- maximum stone size;
- stone size distribution;
- the viscosity of the asphalt mortar at working temperatures.

By increasing the stone content and the stone size and using a low viscous asphalt mortar, the stones settle more quickly under their own weight on the slope, there by producing, more quickly a larger interlock in the stone skeleton. In this way it is possible to design a mix which is easier to work with and in which the stones settle quickly producing a greater stability in the hot condition than with a mix of smaller stones and a more viscous mastic.
Use of larger stones gives a rough surface which has more holding points for waves and currents.
At normal temperatures the material is viscous, except under very short period loads. Application on steep slopes can, because of the limited stability, be difficult.
The lines in Figure 12.1 show viscosity related to percentage of stone for various mixes under the same conditions.
Table 12.2 gives the composition of the various mixes referred to in Figure 12.1.
Tests have been carried out by the Delta Department of Rijkswaterstaat with stone asphalt, layer thickness 0.5 m, on a slope of 1 : 4, at normal temperatures. In these tests, a limestone/mastic mixture, relative proportions 65/35, even after a few weeks, showed no noticeable deformation.
A mix with limestone/mastic proportions of 60/40, however, did show some deformation. The compositions of the mixes, which were tested, are given in Table 12.3.

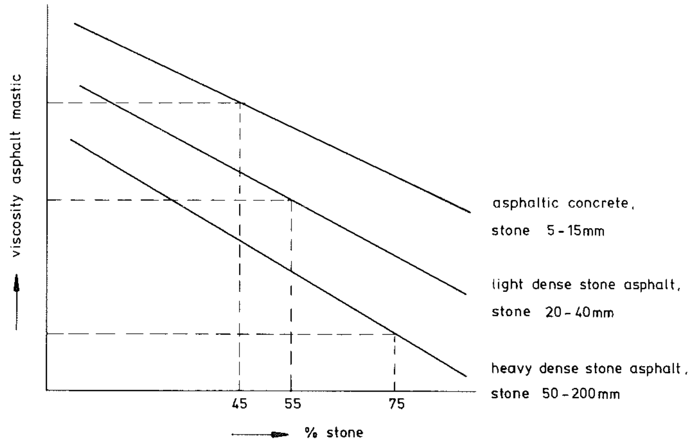


Figure 12.1 Stability (workability) and composition of some asphalt mixes (14).

Table 12.2 Composition of the mixes used in Figure 12.1.

	percentages by mass		
Mix composition	heavy dense stone asphalt	light dense stone asphalt	Asphaltic concrete
Stone 5/15 mm Graded sand Very weak filler Asphalt bitumen 80/100 pen on 100% mineral			48,0 44,5 7,5 7,5
Stone 50-200 mm Stone 20-40 mm Asphalt mastic – graded sand – very weak filler – asphalt bitumen 80/100 pen	75,0 25,0 67,0 15,5 17,5	 55,0 45,0 69,0 15,5 15,5	
Working temperature	110°C	140°C	140°C
Slope	1 : 2	1 : 3	1 : 3

12.3.2 *Stiffness and breaking strength*

With very thick layers and large stones, there is generally no need to consider tearing under wave attack. Stiffness and strength, in this case, are only secondary factors (22). Stiffness and strength are, however, of importance for light stone asphalts in thin layers and, in this case, it is essential to evaluate these properties. There has been, however, little investigation in this field and actual values are, generally, not available. The properties of the mastic, to a large extent, determine the stiffness and strength of the stone asphalt.
See also part 6.3.

Table 12.3 Composition of stone asphalt mixes, percentage by weight (24).

mix composition		weight percentage
Mix 1	limestone 20/40 mm mastic	65 35
	mastic: Sand A Filler Bitumen 80/100	62 19 19
Mix 2	limestone 20/40 mm mastic	60 40
	mastic: Sand A Filler Bitumen 160/210	62 18 20

12.3.3 *Durability*

The mix is dense and, therefore, has a good durability. In this respect the density of the layer surface is very important.

13 Open stone asphalt

Open stone asphalt is used in the tidal zone and above high water and, also, in the form of pre-fabricated mattresses under water. The product is patented under the name 'Fixtone'.

13.1 Basic materials

13.1.1 *Crushed stone*

Because of the need for good adhesion, only limestone is used in open stone asphalt. The gradation most frequently used is 20/40 mm. See also under dense stone asphalt, Section 12.1.

13.1.2 *Gravel*

In view of the lower adhesion obtained with gravel it is not used in open stone asphalt.

13.1.3 *Sand*

In principle every kind of sand can be used provided it does not contain any foreign matter.

13.1.4 *Filler*

Preferably the filler must be weak. Limestone filler is preferred. See also Eisen 1978 (60).

13.1.5 *Bitumen*

Normally bitumen 80/100 pen is used.

13.1.6 *Other constructional aspects*

a. Filter material
   Open stone asphalt is so permeable that it cannot be considered as sand-tight and a filter must, therefore, be used. This can be in the form of:
   - a granular filter;
   - a filtercloth
   - a filter of lean sand asphalt.

The filtering properties of lean sand asphalt and filtercloth best kept in hand. A filtercloth can, if it acts as reinforcement, have a negative effect, see Section 20.6.

b. Reinforcing is generally used with open stone asphalt mattresses to prevent breaking during handling. The reinforcing comprises metal gauze or similar material. Steel cables or synthetic ropes are placed under the mattress for carrying. These are removed when the mattress is in place. Lifting cables which are incorporated into the mattress can also be used.
   Mattresses must be manufactured very carefully in such a way that there can be no direct contact between the reinforcing and the filtercloth. Rust, from steel cables has a negative effect on the filtercloth ageing process. The minimum thickness of the mattress, when using limestone 20/40 mm, is 12 cm; the reinforcing is then 6 cm above the filtercloth.

13.2 Mixes

13.2.1 *Mix design*

Open stone asphalt is an underfilled mix of limestone and mastic. Manufacture is in two phases: first the mastic is prepared and then it is mixed with the preheated stone fraction. By using this method of mixing the mastic can be spread very homogeneously over the surface of the crushed stone and all the contact surfaces between the stones can be fixed with a minimum quantity of the mastic. The production of open stone asphalt requires accurate control of the mix composition; especially the mastic is very important for the overal quality. In addition, it is important that the stone does not contain any undersized pieces.

The stone/mastic ratio in the mix is largely based on the viscosity of the asphalt mastic. If the viscosity is low, mixing is easier but, then, only a limited quantity of mastic is retained by the stones. The excess of the mastic sags and seals the lowest part of the layer. With a stiff mastic mixing is more difficult but more mastic is retained and the tendency to sag is reduced.

The lower limit of the asphalt mastic/stone ratio is determined by the minimum quantity of mastic which is needed to completely coat the stones. This quantity depends, to a large extent, on the mastic viscosity and the mineral grading. The upper limit of the asphalt mastic/stone ratio is determined by the extent to which the mastic tends to sag (20).

In general the composition, is:
- mastic 20% by mass
- limestone 20/40 80% by mass.

For mastic design, see also Section 10.2.

In practice, the mastic/stone ratio is determined by a sag test in which mastic is added to the mix and the quantity noted at which the mix is still ‘open’. A particular quantity of

the mix is placed in a tall tube and, after cooling, the amount of asphalt mastic above and below is recorded using extraction methods and visual observations. In practice the mastic percentage lies between 17 and 21% by mass.
Typical open stone asphalt mixes are given in Table 13.1.

Table 13.1 Composition of open stone asphalt mixes used for the Eastern Scheldt Project.

	composition percentage by mass		
Mix component	desired	min	max.
Limestone 20/40 mm	81	79	83
Asphalt mastic	19	17	21
Asphalt mastic – for mattress			
Sand	67	63.5	70.5
Filler	14	12	16
Bitumen	19	17.5	20.5
Asphalt mastic – for filterconstruction			
Sand	64.0	60.5	67.5
Filler	17.0	15.0	19.0
Bitumen	19.0	17.5	20.4

13.2.2 *Mix design tests*

Mix design tests can include:

1. Investigations of basic materials, see Section 8.1.
2. Investigations of the mix
   a. For mastic, see Section 10.2.2.
   b. For open stone asphalt the object of the investigations is to limit the tendency to segregate at handling temperatures.

13.3 Mix properties

13.3.1 *Stability and viscosity*

The stability of the mix is determined by:

1. the stone skeleton;
2. the adhesion between the mastic and the stone.

The stone skeleton of the mix has less stability than a loose pile of stones but the mastic add a viscous property which, depending on the deformation speed, gives extra

stability so that the total stability is larger than for loose stones. When deformation occurs quickly the material reacts elastically.
An impression of the stability can be obtained by moulding a quantity of the mix using wooden formwork at the slope required and noting how it performs at working temperatures and in cold conditions.
It appears that:

1. Under hot conditions (110°-120°C) mixes with limestone are stable on steeper slopes (1 : 1) than mixes with gravel (1 : 1.5). This is due to the greater interlocking of the limestone fragments.
2. For long-period normal conditions (30°-40°), however, gravel is more stable. This is because the contact surfaces between gravel particles, being round, are larger than for the angular limestone and the adhesion, therefore, is greater. With good compaction, however, the stability of limestone mixes can be improved. In practice compaction is not carried out.

The viscous behaviour of the mix is strongly determined by the viscosity of the mastic. The upper limit of the mastic percentage is determined by the degree of saggin after placing or by the minimum voids ratio required, see Section 13.2.1. For mastic a viscosity of 40 Pa . s at 140°C is desirable. The limits to viscosity, depending amongst other things on the measuring method, lie between 30 and 80 Pa . s.

13.3.2 *Mechanical properties*

Mix properties such as breaking strain, stiffness modulus etc., values which are necessary for calculating an open stone asphalt revetment can be estimated, generally, with the help of Figures 13.1 and 13.2.
These figures are based on investigations into the properties of open stone asphalt (72).

13.3.3 *Water permeability*

The voids ratio of open stone asphalt is, generally, between 20 to 25%. The material is, thus, very permeable. The water permeability which is of the same order of magnitude as that of loosely dumped stone of the same nature can be measured using a permeability test carried out with an apparatus designed for large samples.
The permeability depends on:

- the stone grading;
- the mastic/stone ratio.

Investigations indicate that, under normal conditions, the flow in open stone asphalt is between laminar and turbulent. If open stone asphalt is used with a filtercloth, the permeability of the structure will be largely determined by the permeability of the cloth.

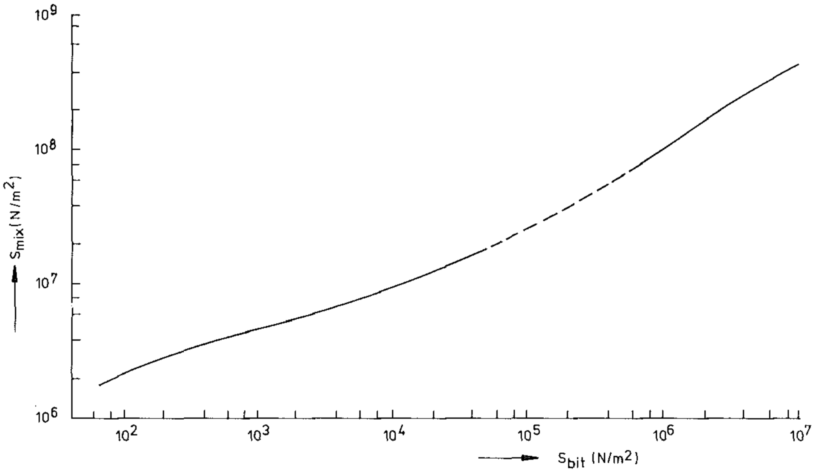


Figure 13.1 S_{mix}-S_{bit} relationship for open asphalt, determined from the results of creep and dynamic tests.

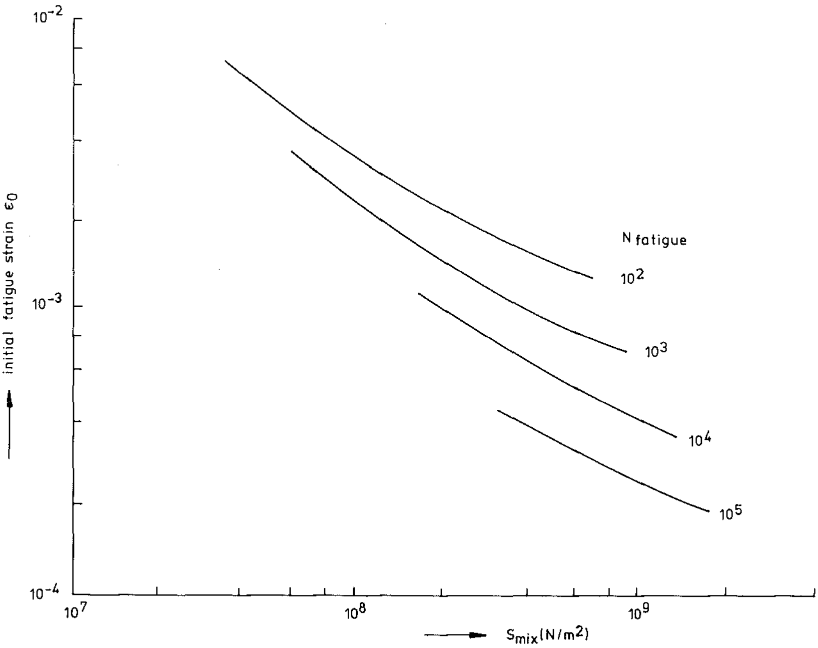


Figure 13.2 The relationship between the stiffness modulus for n = 10, the initial strain and the number of loading cycles required to break.

13.3.4 *Durability*

Open stone asphalt, as the name suggests, is an open mix and the durability therefore, is of great importance.

The durability is also determined by that of the stone for which there are standard durability requirements.
For the mix itself the following must be taken into account:

a. Aging
   This manifests itself in
   – reduced adhesion between the stone and the mastic ('stripping');
   – aging of the mastic layer itself;
   – sensitivity of the total mix.
b. Erosion. This can be investigated with flow tests.
c. Biological attack. This should be investigated with plant penetration tests.
d. Chemical attack, by, for example, polluted water which:
   – reduces adhesion;
   – attacks the mortar.

Some petroleum products weaken bituminous materials. This process will occur more quickly with open mixes. Marshall tests on cylinders of open stone asphalt, which have been placed for a certain time in a bath containing hot water with 1.5% m oil, indicated only a limited change in the mechanical properties (30).
The laboratory of the Bundesanstalt für Wasserbau at Karlsruhe has carried out various tests related to temperature resistance durability (62):

1. A sample, placed on a slope of 1 : 3, was subjected to the equivalent of 3000 hours sunshine at a surface temperature of 60°C. No flow phenomena were noted.
2. A sample, placed on a slope of 1 : 2, was subjected to ultraviolet rays, equivalent of 4000 hours of sunshine and, again, no flow phenomena were noted.

Freeze/thaw tests on open stone asphalt cylinders did not show any damage to the material.

14 Lean sand asphalt

14.1 Basic materials

14.1.1 *Sand*

See Eisen 1978 (6).
From experience it appears that, in order to obtain the same quality of lean sand asphalt with different types of sand, different bitumen percentages have to be used. Factors which influence this are, amongst other things, the specific surface and the shape of the sand particles.
When lean sand asphalt is to be used as a filter material it is, in connection with the permeability, best to use the same sand as that upon which the filter is to be laid for permeability performance.

14.1.2 *Bitumen*

A bitumen which is too hard can lead to higher working temperatures. For application under water, however, this is undesirable. At temperatures over 100°C the water will boil. This can neutralize the adhesion required between the bitumen and the mineral, see Section 14.3.4.

14.1.3 *Other constructional materials*

Gravel and filler can be used for certain applications.
Gravel can be used if:

- a greater stability is required;
- a faster mixer production is desired;
- a transition must be made onto coarser material, such as quarry stone.

The use of filler, and the associated change in bitumen content, gives, in general, a stronger material with greater resistance to erosion which can be used as a permanent revetment. The water permeability is greatly reduced and, since compaction is then necessary it cannot be applied under water.

14.2 Mixes

14.2.1 *Mix design*

Lean sand asphalt is generally a mix of sand and bitumen, the most frequently used

bitumen content being 2 to 6% by mass. The bitumen functions as a binder and tends to concentrate on the contact surfaces between the particles. In order to obtain a certain film thickness, the total surface area of the mineral particles has to be taken into account as well as the bitumen content (34).
So, for example, in a hot mix lean sand asphalt, with a mean particle diameter of 0.1 mm and a bitumen content of 4% m, the particles have a film thickness of 1 to 2 μm.
The pattern of the voids of the original sand is changed by the addition of the bitumen; the extent, however, is unknown. Adding of filler above a certain level results in enlargment of the voids. Only by heavy compaction the voids between the sand grains are filled in such a way with a stiffer mortar, that they are reduced noticably.

Mix design, aimed particularly on durability and water permeability, is based mainly on experience. Factors which can influence the properties of lean sand asphalt are (32):

1. The bitumen content. This is the critical factor, since:
   a. The strenth increases as the bitumen content is decreased;
   b. The resistance to wear increases with increasing bitumen content;
   c. The permeability decreases with increasing bitumen content;
   d. When altering the bitumen content the deformation characteristics change.

   A bitumen content of 3% m is definitely the lower limit. It must not be any lower, since then the durability is strongly reduced. A bitumen content of 5% m is the maximum since a higher percentage produces a lean sand asphalt which is too 'fat' which, depending on the application, reduces strength too much. A bitumen content of 3% m is adequate for temporary work; for permanent work and filter layers the content should be between 4 and 5% m.
2. Degree of compaction. A high degree of compaction increases the wear resistance and the deformation and strength properties but lowers the permeability.

14.2.2 *Mix design tests*

In addition to investigations into basic materials mix design tests can comprise a comparison with known lean sand asphalt. These investigations can include:

1. Permeability;
2. Erosion tests (by rolling or by water-jet);
3. Retained stability;
4. Marshall tests;
5. Triaxial tests.

14.3 Mix properties

14.3.1 *Stability*

The stability, in relation to forces aplied slowly is less than for natural sand. With forces which are applied more quickly the stability is greater, see also Section 20.7.

14.3.2 *Mechanical properties*

The stiffness modulus of the mix is dependent on the duration of loading and the temperature and is related to the mineral content and the stiffness modulus of the bitumen.
Because of the high voids ratio of lean sand asphalt the bitumen hardening process is faster than for most dense mixes. Creep tests, carried out on samples taken from a 1 : 1 scale test which comprised 96% m fine sand and 4% m bitumen at temperatures of 15° and 20°C, indicated the values shown in Figures 14.1 and 14.2 (32).

14.3.3 *Permeability*

Because of the limited quantity of bitumen in the mix the sand grains are only covered by a thin film of bitumen a few microns thick. The bitumen tends to concentrate on the contact surfaces between the grains. This is the cause of the fact that, depending on the compaction, grain composition and shape, the lean sand asphalt has a high permeability. With the low degree of compaction which is found in practice the permeability tends to be similar to the sand of which it is made. With dry lean sand asphalt an 'initial resistance' has to be overcome before the ultimate permeability can be reached.

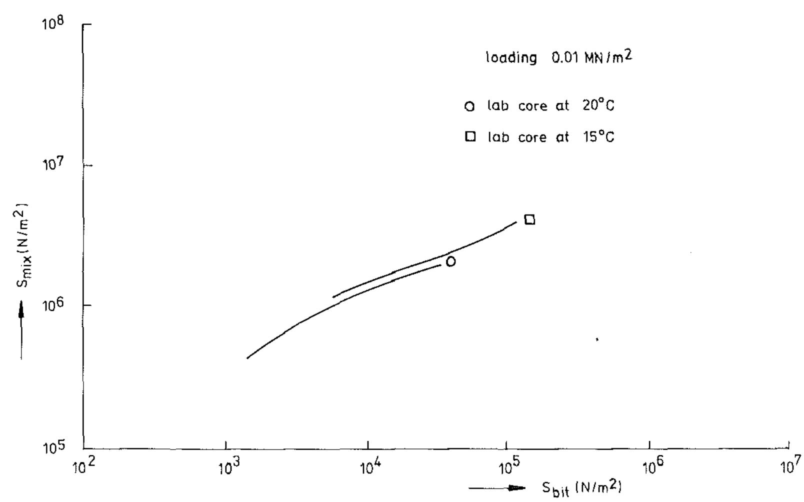


Figure 14.1 Stiffness moduli of lean sand asphalt.

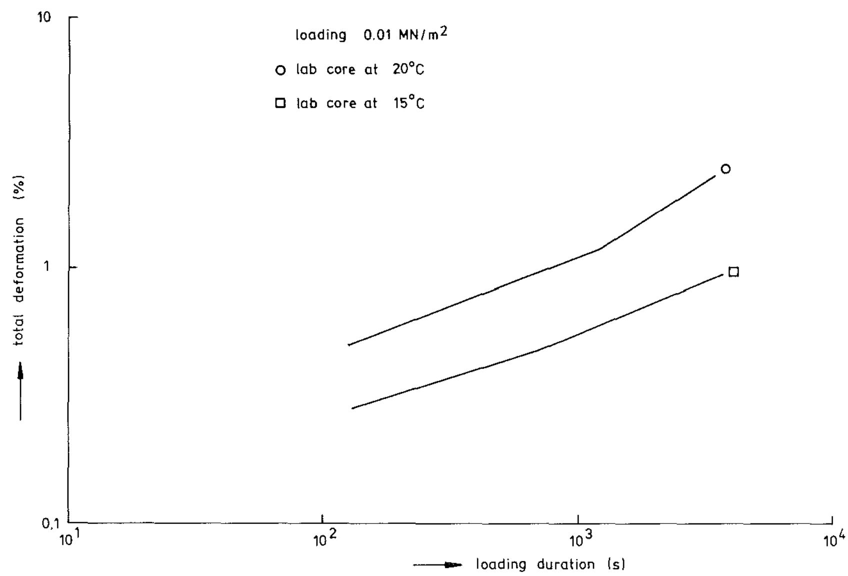


Figure 14.2 Deformation as a function of the loading duration.

14.3.4 *Durability*

The durability of the lean sand asphalt depends on the permanent binding effect of the bitumen.

For applications under water working temperatures above 100°C have an undesirable effect on durability, see Section 14.2.2.

The following phenomena and properties are relevant to durability of lean sand asphalt.

a. Aging is relatively fast because of the open character of the mix.
b. The adhesion between the sand and the bitumen is reduced when water is forced into the mix ('stripping'). This effect can be measured with a retained stabilty test (39) or some other water sensitivity test. If the stability is measured over a period of time it appears that it eventually reduces to a value of about 30%, see Figure 14.3, depending on the mix composition and the physical conditions (32, 33). A higher bitumen content and a thicker film on the grains tends to improve this effect.
c. Good chemical resistance particularly the reaction to polluted water.

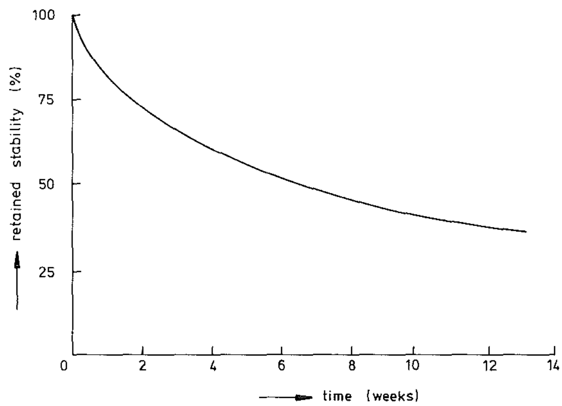


Figure 14.3 The effect of stripping on lean sand asphalt.

15 Bitumen membranes

Bitumen membranes are used for making canals, watercourses and banks watertight. There are two methods of execution:

- in situ;
- pre-fabricated.

15.1 Basic materials

15.1.1 *Filler*

Some times filler is used to increase the stability of membranes. The filler has to be in connection with the other materials used.

15.1.2 *Bitumen*

The selection of the bitumen type depends on the application – for example, at low working temperatures or for a certain desired deformation. For application of blown bitumen, see (35) and Section 15.2.1.

15.1.3 *Additives*

- plastic rubber to increase stability;
- materials to prevent plant penetration.

15.1.4 *Other constructional aspects*

Prefabricated membranes include:

1. Reinforcing (sometimes also used in membranes made in situ). Generally the reinforcing is a synthetic cloth which has the following properties
   - strength in warp and weft directions;
   - strain at break;
   - strength in relation to temperature effects;
   - workability (can be handled easily).
2. Thin sheets, also referred to as inlays
   - to resist root penetration;
   - to protect sealing strips.

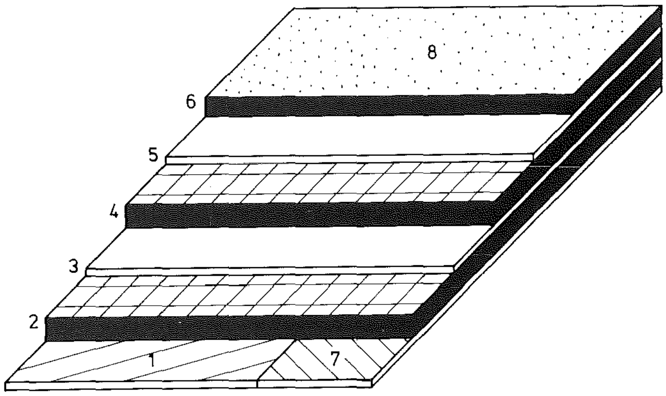
3. A wearing surface finish, amongst other things to increase the friction.
   This can be achieved by scattering the surface with:
   - sand;
   - fine stone particles.
4. It is advisable to use blown bitumen for welding the membranes together.

15.2 Design and design tests

At present membrane design is done empirically, using 'trial and error' methods. Mix design involves tests on the membrane in the conditions in which it will be used.
There are two types of membranes:

1. Membranes prepared in situ.
   The membrane consists of two or three layers of bitumen, R85/50 or R85/40, with a total thickness of 8 to 12 mm, sprayed onto the subsoil (35, 69). The quantity of bitumen depends on the subsoil texture and ranges from 6 kg/m^2 for fine subsoils to 8 kg/m^2 for rough. The maximum slope angle of the subsoil should be 1 : 2.5.
   To protect the membrane it should be covered with at least 30 cm soil or, if available, gravel. If vegetation is anticipated the protection layer thickness should be increased to 50 cm so that weeds can be removed without damaging the membrane. The subsoil should be well-compacted and smooth without stones or other obstacles and should be pretreated with a planticide.
   The disadvantages of membranes, prepared in situ, are:
   a. they are prone to plant penetration;
   b. they are prone to settlement of the subsoil;
   c. adhesion between the layers of bitumen cannot be guaranteed;
   d. the quality is only moderately reliable;
   e. the subsoil must be prepared very carefully.
2. Prefabricated membranes.
   Because of the disadvantages of in situ membranes prefabricated membranes have been developed. Prefabricated membranes often comprise a bearer or reinforcing of synthetic cloth between layers of bitumen or bitumen/filler-mix. Because of the reinforcing the membrane is less likely to settlement damage and, because of the anti-plantpenetration sheet it is not necessary to treat the subsoil with a planticide.
   The subsoil must be smooth and free of sharp objects. In order to reduce the danger of the protection layer sliding off the membrane, it is sprinkled with sand or fine rubble (29).
   The protection layer can be of earth, crushed stone, concrete asphalt etc. Depending on the type of protection an 'non-woven' can be used to reduce the effects of the temperature and mechanical loadings on the membrane.

An example of a prefabricated membrane, patented under the named 'Hypofors', is shown in Figure 15.1 (36).



1. paper
2. bitumen
3. reinforcing
4. bitumen
5. polyester film
6. bitumen
7. polyethylene strip
8. sprinkled sand

Figure 15.1 Hypofors (prefabricated membrane-patented).

15.3 Membrane properties

15.3.1 *Stability*

It is necessary to know the stability of the membrane in relation to the sliding forces to which it is subjected. Investigations can be made with the help of a rheometer.

15.3.2 *Viscosity*

The viscosity of the bitumen is important in relation to:

1. the stability of the membrane and the protection layer;
2. the workability.

For the viscosity of bitumen reference should be made to Chapter 3.

15.3.3 *Permeability*

The membrane and the seams must be and must remain watertight under the maximum specified loads. This property can be investigated using a vacuum meter.
The membrane must also remain impermeable under deformation.

15.3.4 *Durability*

The membrane must be durable. In this respect the following properties are relevant:

- Ageing of the bituminous layer.
- Biological resistance; tests may be needed on plant penetration.
- Chemical resistance; resistance against agressive percolation.

Membranes are generally protected with a cover layer.

PART C

TECHNICAL ASPECTS OF THE DESIGN

Summary

Part C deals with the technical aspects of the design of asphalt revetments used in hydraulic engineering.
Such revetments must satisfy a number of requirements in order to function properly. A review of these functional requirements is given; the designer should take note of which of these requirements to apply to a particular situation.
The functional requirements are partly deduced from the loads which can be exerted on an asphalt revetment. The loads can be distinguished into loads related to the hydraulic conditions such as water-levels, waves and currents, loads arising from the settlement of a dikebody and the subsoil and erosion of the foreshore and other loads. The influence of these loads on an asphalt structure are discussed.
Dike design is based on a number of starting points. In the Netherlands for some situations these are established; in other situations, however, not. A summary of the present 'state of the art' is given.
A very important factor in the design and construction of an asphalt revetment is the nature of the dike body and the subsoil. A paragraph is devoted to the aspects which play a role in this subject. An attempt is made to indicate in which way the situation can be influenced favourable.
An asphalt revetment is normally dimensioned on the basis of the loads which result from the hydraulic conditions, settlements and scouring and, sometimes, on other loads. The dimensioning methods for the most relevant conditions are given. The following situations are discussed:

- design of a relatively impermeable asphalt revetment against hydraulic uplift;
- design of a plate-type asphalt revetment against wave loads;
- design of an under water bed protection against uplift pressures caused by currents and waves;
- design of a surface- or pattern-grouted stone revetment against wave action;
- design of an asphalt revetment against currents;
- design of an asphalt revetment against irregular settlements and scouring;
- the determination of the maximum slope angle.

16 The functional requirements of an asphalt revetment

The function of a dike is to protect the area behind from floading. In order to protect the dike body, which is generally built up from soil, against erosion it is often provided with a revetment. Since the revetment is, at the same time, a part of the dike it can also fulfil other functions as, for example, watertightness. In general, for design purposes, the revetment may not be used to increase the safety of the dike body, in itself, against slipping.
An asphalt revetment must, in order to function satisfactorily, fulfil various requirements; requirements which stem partially from the loads which can be exerted. These requirements are:

1. The revetment must be so that material from the dike body cannot pass through it. Sometimes the requirement is that the revetment should be watertight.
2. The revetment must be able to withstand:
   - waves generated by wind and ships;
   - currents and material carried with it such as sand, stones and driftwood;
   - uplift water pressures (only applicable to a relatively watertight revetment);
3. The revetment must be able to adjust to settlements and scouring, within limits, and must remain in contact with the subsoil.
4. The revetment must be stable.
   It should not slip down from the slope, also, during construction. It should, also, be stable, as a whole, so that it cannot be carried away by the loads which act upon it. This implies a certain dimension and weight.
5. The revetment must be weather and water resistant to erosion, corrosion, light, wind, temperature, and ice. The environment should not be able to damage the revetment or vice versa.
6. The revetment must be durable, that is, it must continue to function throughout its design life.
7. The revetment should, preferably, be aesthetically acceptable. This condition, however, if it is contrary to other requirements, should never be overriding.
8. In addition the following should be taken into account and guarded against:
   - biological damage by plants, animals and sea organisms;
   - chemical damage by polluted or salt water;
   - possible land traffic, during construction and when completed;
   - vandalism;
   - recreational activities;
   - vessels and anchors.

The above requirements must, in principle, be satisfied. This is possible by composing the revetment of one or more materials. Often, because the various requirements demand different revetment properties, a compromise must be sought.
The combination of the revetment and the dike body determine whether or not the dike functions satisfactorily, also in the coarse of time. Also it should be possible to construct the revetment under to local conditions. The dike body itself should be such that a reasonable cheap, well-functioning revetment can be achieved.

17 The dike body

An important factor to take into consideration when dimensioning and constructing a dike revetment is the nature of the dike body, that is, the sandbed.*
The following aspects are important:

a. The bearing capacity of the dike body determines among others the performance of a revetment under wave attack and other forces, and, therefore, plays a large role in the dimensioning. If the bearing capacity is large then often the thickness of the revetment can be reduced.
The properties of the soil such as the modulus of elasticity, the modulus of subgrade reaction and Poisson's ratio are important, see Appendix V. They themselves are influenced by the amount of compacting of the dike body. These properties can be determined from, for example, plate bearing tests.
b. A high degree of compaction can, amongst other things, avert the softening of a saturated or almost saturated soil by impact loads, for example wave attack, which can cause loss of bearing capacity.
A relative proctor density of 95-100%, down to a depth of about 2 m, can, in general, in sand reduce the possibility of softening to an acceptable minimum.
Bad, permeable, wet soil is prone to weakening; the presence of mud, in this context, is undesirable.
c. The permeability of the sandbed is important in connection with groundwater flow in the dike body and, consequential uplift pressures under a relatively watertight revetment. It is also important in connection with likely softening of the sand body.
d. The placing above water of an open asphalt mix on a saturated sandbed can, through the influx of water, result in the early development of stripping. Vibration compaction can soften a loosely packed saturated sandbody. Under impermeable mixes, as asphaltic concrete, uplift pressures can develop while the asphalt is still soft. This situation can, for example, occur by the delivery water from hydraulic filling work.
e. The dike body should have sufficient bearing capacity to support construction activities. If the sandbed has little resistance to deformation it is difficult to compact and, construction equipment can cause track impressions ('rutting').
After the sandbed has been compacted and made smooth it should not be driven over or care should be taken to ensure that it is not disturbed.
The dike body is often formed of sand which is reasonably easy to compact. A

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* Although this chapter is directly related to the dike body the contents are also applicable wherever an asphalt revetment is constructed on an earth structure.

good compacted subsoil gives fewer problems while compacting the revetment, initial cracking is limited and the voids ratio required easier to achieve.

In some areas in the Netherlands it has been common practice to only smoothen the sandbed with a bulldozer. This, however, only produces a limited improvement on the original density. On several dike projects, tests have shown that, even at considerable depth, the compaction was low. It is, in fact, better to build up the sandbody in thin layers, with bulldozers. The dike face can then be profiled accurately, also with bulldozers. If this does not produce the compaction required, then a vibration roller should be used. For this process it is recommended that the sandbed is first sprayed with water (Photo 4).

Photo 4 Compacting the sandbed using a vibratory roller

The best dike body construction method, however, is to dump an excess of material, and then, after this has been compacted, to produce the profile required by grading. The application of a clay underlayer is not recommended because of its weak consistency and the possibility of 'frost heave'. If an asphalt revetment is to be built on an existing clay revetment this should be of a good quality, otherwise it should be removed. Investigations of the quality are desirable.

There are generally no problems with underlayers of inert mine waste or lean sand asphalt. There can be some problems when laying an asphalt revetment on mine waste in connection with obtaining an equal revetment thickness.

Subsoil can be improved by:

- mechanical compaction;
- physical/mechanical treatment; an improvement of the particle skeleton in combination with mechanical compaction. This method has not yet, generally been used in hydraulic structures, but perhaps it will be in the future.

f. After constructing the revetment the dike body will tend to settle. If it has not been well compacted or if there are clay or peat layers in the subsoil, the settlement can be large and irregular. If the bed, is at the same time highly permeable then it is possible that the grain stress only recovers slowly and that the bearing capacity of the bed temporarily appears to be insufficient. This effect must certainly be taken into account with clayey subsoils; good drainage in this case is essential (17). With very permeable material the situation does not develop.

The bearing capacity of a ground mass can be measured using C.B.R. tests, soundings, or plate-bearing tests. It is recommended that laboratory investigations are carried out to determine values of permeability, proctor density, friction factors, etc. Sufficient measurements should be made to obtain representative values.
For the present the Rijkswaterstaat makes the following recommendations for compacting subsoil: The minimum compaction should be 95% of the maximum proctor density. The average compaction should be 98%.

18 Design basis

18.1 Safety aspects

The philosophy of dike revetment design stems from several starting points. Although these starting points vary from situation to situation, in the Netherlands, generally, the following can be posed:

- In the report of the Delta Commission (83) for sea dikes, the most extreme conditions are established by the design level. If the water-level reaches this design level then, according to the Delta Commission, there should still be a considerable factor of safety against breaching. This implies that a revetment, as far that it is essential for the functioning for the dike, must be able to resist a design water-level with a large probability. During a storm in which the design level is reached not all parts of the dike are loaded to a maximum. The maximum and most critical wave impact loads develop a little below still water-level. The maximum uplift pressure under the revetment will occur some hours after the maximum water-level has been reached.
- Dune toe protection.
  No general basic rules have been formulated for the use of asphalt in dune toe protection. The effect of dune toe protection on erosion during the design conditions is still to a large extent unknown.
  When designing against dune erosion the basis should be the same as that used for sea dikes. The most important function of dune toe protection was often to limit dune coast erosion.
- River dikes.
  The following is stated in the report of the River Dikes Commission (84): 'The commission recommends that improvements to river dikes should be based on a leading water-level related to a Rhine discharge of 16,500 m^3/s at Lobith. This discharge occurs with a frequency of about 1/1250'. In the directives for river dikes (Leidraad Rivierdijken), published by the Dutch Technical Advisory Committee on Water Defences (Technische Adviescommissie voor de Waterkeringen) in 1984, it is stressed, as in the report by the Delta Commission, that there should be a large factor of safety against possible breaching in design water-level conditions.
- Canal dikes and embankments, etc.
  Dikes and embankments along canals, lakes, reclaimed areas and polder reservoirs form a type of protection works of which the object is to protect against a permanently high water-level. These dikes are often of limited dimensions. With the exception of bow waves from passing ships, there is generally very little wave action on these structures. Exceptions to this are the dikes around large lakes, such as the IJsselmeer,

where there can be considerable ware attack and high water-levels can develop in relation to wind set-up.

18.2 The decrease of structural strength

The fact that the strength of a structure can decrease with time must be taken into account in the design.
There are two different ways of estimating this problem:

1. The structure can be 'over-designed' so that it continues to function satisfactorily throughout its life.
2. Structural strength can be preserved by regular control and maintenance procedures.

The choice between the two depends on the experience of the designer: does he accept that a certain amount of risk is delegated to the control and maintenance by other parties, and also by economic factors: lighter structures cost less to build but more to maintain.
At the same time, the safety of the structure during the construction period should be borne in mind and the possibility of structural failure in this period should be anticipated in the design and method of construction. A solution can often be found by choosing a favourable time of the year, construction sequence and method of execution.

18.3 Theoretical background of safety aspects

A dike revetment is dimensioned with the help of theoretical models in which certain basic variables are used to determine design dimensions, such as the revetment thickness. The basic variables describe, on one hand, the different loads on the structure: waves, water-levels, structural weight, etc.; and, on the other hand, the structural properties: material properties, geometry, etc.
If the basic variables are specific deterministic quantities there are no problems related to the calculation, apart from the choice of a good theoretical model.
The theory states that the structure in that case will not fail if the – deterministic – loads are less than the – determenistic – strength.
The difficulty lies in the fact that the basic variables are stochastic quantities, that is, variables. Loads and stresses are, therefore also stochastic properties. The problem can be defined as follows: the structure must be designed in such a way that the probability of failure is acceptably small. For this type of design there are three different methods available (2):

1. The (semi) – deterministic method
   In this method the basic load-related parameters are assumed to be adequately high and thus 'safe' values. Often average or safe values are selected for the strength properties. The choice of 'safe' loading is mainly based on experience.

2. The quasi-probabilistic method
   In this method 'safe', what are referred to as 'characteristic' values are assumed for all basic variables. The characteristic load is, for example, defined as the load which has 5% probability of being exceeded during the lifetime of the structure. The characteristic strength properties are then those values which have a 5% probability of being less than the lower limit, see also Figure 18.1.
   In the building and construction world the margin between characteristic loading and strength can be given by using partial safety coefficients. Partial safety coefficients, as defined in I.S.O. 2394, are then transposed by multiplication into the total safety coefficient:

$$\gamma_{\text{total}} = \prod_{i=1}^{m} \gamma_{\text{partil i}}$$

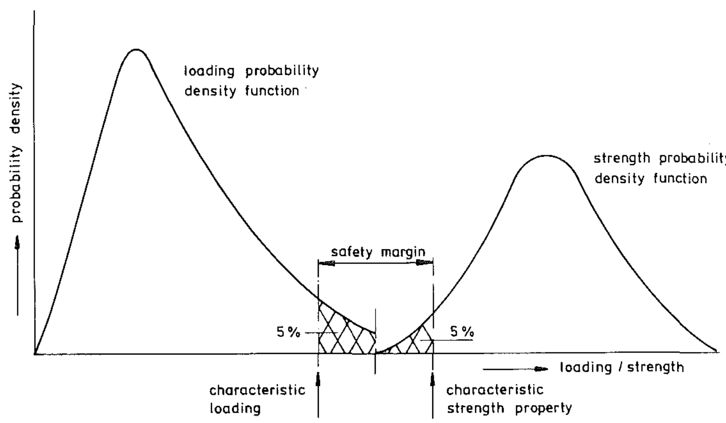


Figure 18.1 Example of a choice of characteristic values

3. The probalistic method
   This method, which is still being developed, is the most advanced. All basic variables are specified as probability density functions. The probability of structural failure is then determined by integrating the probabilistic density functions of the loading and strength properties. The probability density of structural failure is then used as a criterion for judging design. More information on this subject is given in (75, 76, 77).

In practice it appears that, at present, the method most used is somewhere between deterministic and quasi-probalistic. Variations in the loads are taken into account but not the structural strength properties.
For loading variables values are mostly selected which have an extremely low probability of being exceeded. Average or characteristic values are often assumed for strength properties.

19 Loads

A number of different kinds of loads can be exerted on an asphalt revetment. These can be roughly subdivided into loads caused by hydraulic conditions, loads resulting from settlement and scouring and loads due to other factors. In general, revetment design is based on the first two categories and also on other loading types which develop regularly or can be expected to occur with reasonable probability. Sometimes it is not practical to design for certain loads, because, although they are very large it is cheaper to repair the revetment or to take preventative measures.
The most important conditions and the related loads are treated generally in the following sections. The most common types of loads and methods of design are discussed in subsequent chapters.

19.1 Hydraulic boundary conditions

A particular water-level can be anticipated for a dike either permanent or temporary. The type of protection, the absence or presence of a foreshore and its height, and the water-level and its variation are all important to this aspect. Waves and currents can be expected to occur in the water in front of the dike.
In order to dimension a revetment for specified hydraulic conditions it is necessary to know the actual loads which will occur. Ideally, therefore, the waves, currents and water-levels, which will occur on the dike itself, should be known so that the loads can be calculated directly. Those types of data are however mostly available for deep water, some distance from the shore, and have to be converted to data at the coast itself. Waves, for example, can be transferred by refraction calculations.

19.1.1 *Water-levels*

The water-levels in front of a dike may vary. For dikes on the coast, in inlets and estuaries this variation will mainly be caused by tides, winds or both in combination. In the upper reaches of rivers the water-level is mainly determined by the discharge; this can also have effect in lower reaches. In canals, waterways and polder reservoirs water-level variations are due, principally, to inflow and outflow through locks, to rainfall, and to waves caused by ships; in lakes and reservoirs water-level variations are caused by the in- and outflow of water and by wind (1). Wind generated waves also give short-term water-level variations.
The duration of a change in water-level can vary greatly. Variations due to wind and ship waves last only a few seconds, tidal variations on the other hand have periods of several

hours, long-term wind effect can last a few hours or days and river discharges can affect the water-level for days to weeks on end.
A phreatic line is set up in the dike body in connection with the water-level in front of the dike. Variations in the water-level produce changes in the phreatic level. The extent of these changes depends, amongst other things, on the duration and extent of water-level changes, the geometry and permeability of the subsoil and the dike body, and the relative permeabilty of the cover layer. When the water-level outside the dike is lower than the phreatic line and water cannot flow freely out of the dike because the revetment is relatively impermeable then uplift pressures can develop under the revetment. This will tend to lift the revetment. In this situation the revetment should be designed as discussed in Section 20.1.
The soil under the phreatic line is saturated. Weak soil under the cover layer can, under a sudden load, be softened and lose its bearing capacity. Although this phenomenon should be taken into account in the dike design, in general it is not practical for it to form a basis for revetment dimensioning.
The maximum allowable slope angle is also dependent on the water-level in front of and in the dike, see Section 20.7.
Finally the water-level also affects the wave climate and the currents at the dike.

19.1.2 *Waves*

Waves, at least those dealt with in this section, are generated by wind. The factors affecting waves include the wind speed, the fetch, the wind duration and the water depth. A wave can be expressed in terms of:

1. The wave height (H);
2. The wave period (T);
3. The speed of propagation (c).

The wave length is: $L = c \cdot T$.
In the area where the wind disturbs the sea a very irregular wave field can develop which comprises a whole range of waves moving in several directions with different heights and periods. When the influence of the wind deminishes the waves propagate out of the wave field in different directions. The short period waves damp out before the long period, leaving a more regular swell.
One method of defining a wave field is that wich is referred to as a spectrum, by which the wave period is related to a parameter deduced from wave height. A wave spectrum is normally composed from a number of measurements made at several locations over a long or short period. From these measurements a series of stochastic quantities are obtained over a limited time period which can be treated statistically to long-term predictions. More information on this topic can be obtained from the considerable literature available and also from Appendix I.
A parameter which is frequently used to describe a wave in deep water is the signif-

icant wave height (H_{sig}). This is the average of the highest third of the observed waves. This value agrees well with the wave height which, based on a Rayleigh distribution of waves in a wave field, is exceeded by 13.5% of the waves.

Roughly three areas can be distinguished:

1. Deep water
   The wave movement is not influenced by the waterdepth; the speed of propagation is related only to the wave period. The movement of water particles in the wave at the bottom, is not noticeable.
2. Shallow water
   The waves are greatly influenced by the bottom; the wave celerity is determined by the water depth alone. The horizontal velocity of the water particles in the wave is independent of the vertical position of the particle and is very strong at the bottom.
3. Intermediate water
   The wave movement is influenced by both the wave period and the water depth.

Generally these categories are expressed as:

- Deep water $d/L_0 > 0{,}25$
- Shallow water $d/L_0 < 0{,}05$

L_0 is the wave length in deep water and d the waterdepth.

As the waterdepth (d) decreases a wave of height H travelling from deep to shallow water will increase in height to a certain limiting value given by $H/d = \gamma$, γ is the breaker index, after which the wave starts to break, see Figure 19.1.
A wave approaching a dike slope will, depending on the water depth in front of the dike, the wave length, and the wave height have already broken or will break on the dike face. In the case of the latter the breaking process will be influenced also by the dike slope. There are a number of breaker types:

1. Spilling breakers:
   The wave front, in this case, is steep but not vertical. The wave begins to collapse at the crest and the breaking mass of water runs into the trough of the preceding wave and up the wave slope.
2. Plunging breakers:
   A 'tongue' extends out from the crest and falls freely into the preceding wave trough or onto the dike face.
3. Surging breakers:
   These develop with low waves and steep dike slopes. The wave builds up like a plunging breaker but before the crest tongue has formed the toe of the breaker surges up the slope and the breaker develops no further.

It is possible to get an impression of which type of breaker will develop with the help of the so-called ξ parameter.

$$\xi_0 = \frac{1}{\sqrt{2\pi}} \cdot \frac{\text{tg}\, \alpha}{\sqrt{H_0/gT^2}}$$

in which:

α = slope of dike face (dgr)
H_0 = deep water wave height (m)
g = acceleration due to gravity (m/s^2)
T = wave period (s)

$\xi < 0{,}5$: spilling breakers
$0{,}5 < \xi < 3{,}3$: plunging breakers
$\xi > 3{,}3$: surging breakers

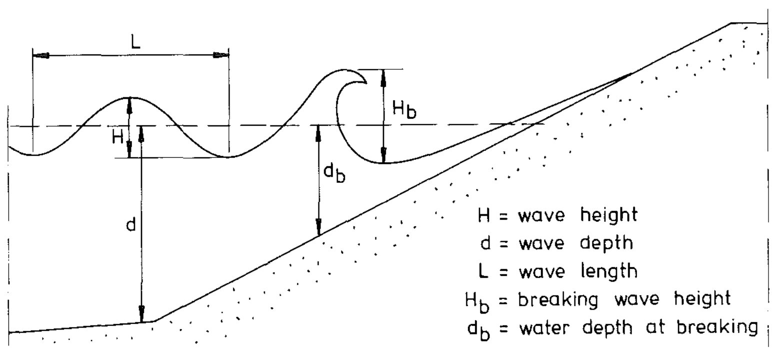


Figure 19.1 The breaking of a wave as it approaches a dike slope.

A dike revetment must be able to resist the forces exerted by waves. In the design distinction is made between plate-type structures and structures built up of loose elements.

Plate-type revetments are, for example, asphaltic concrete revetments, mastic slabs, fully grouted rubble layers, open and dense stone asphalt revetments and layers of lean sand asphalt.

The largest loads which waves exert on a plate-type revetment are impacts from plunging breakers. The dimensioning method is given in Section 20.2. Structures built up from loose elements are for example pattern grouted riprap revetments. Riprap slopes, with surface grouting form, an intermediate type of structure between fully grouted riprap and pattern grouted riprap.

Water flowing on and off the revetment can cause shear which has an abrasive effect. This aspect is discussed in Section 20.5.

At the same time pressure differences can be built up by wave movement across a relatively waterimpermeable revetment. This results in upward pressures which tend to

lift the revetment and reduces the resistance to slipping. The pressure differences can also produce a hydraulic gradient in the groundwater which can cause the movement of sand along and/or through the revetment. The result can be a cavity under the revetment.
There are no specific dimensioning methods to deal with this phenomenon. A saturated subsoil can be softened by a sudden loading, for example, a wave impact, and lose its bearing capacity.
The water movements in a wave, whether or not it has broken, can erode the bed at the toe of a dike. In order to prevent this a bed protection is often constructed. If the bottom protection is extended to where the wave movements are too weak to transport the sand particles, there will be no problems provided that there are no other strong currents present. To guard against erosion the bed protection should be sufficiently long so that any scouring stops at some distance from the dike toe. This aspect is discussed further in Section 20.6.2.
A dense underwater bottom protection can be lifted up by differences in pressure, above and below, caused by translatory or standing waves. This phenomenon is discussed further in Section 20.3.2.
It is usual to design a revetment for extreme conditions referred to as the design boundary conditions. What should not be overlooked, however, is that wave forces, although they are not always extremely large, can occur frequently and that a lot of minor damages can eventually lead to failure. Since asphalt is sensitive to fatigue – the strength reduces in relation to the loading frequency – this property should not be overlooked in the design.

Waves, acting on a revetment of loose elements, such as riprap, will exert certain forces. The flow pattern caused by wave action on the revetment is complicated and, as a result, it is not possible to give a good theoretical assissment of the problem. Formulas for determining the weight of revetment elements under wave attack are wholly or partly empirical. An example is the Hudson formula, see Section 20.4.

19.1.3 *Currents*

Currents can be distinguised into:

- stationary currents;
- quasi-stationary currents, for example, those caused by the movement of a ship;
- non-stationary currents, for example, the run-up and run-down on a dike face caused by wave action.

Flowing water can have an eroding effect on an asphalt revetment apart from that caused by material which it is transporting. Pieces of wood, stone, etc., being carried by the water, can hit the revetment and cause damage, especially in the breaker zone, see also Section 7.5.3.

Flowing water can get hold on the side of thin asphalt plates to flap. At the same time the pressure differences which develop on a platetype, watertight bed protection can cause it to lift. Methods of design against this are given in Section 20.3.

As is the case with wave action it is not possible to adequately describe the loads on a loose element revetment due to currents, theoretically. The weight of the revetment elements must be determined using formulas developed empirically.
Non-steady flows, because of the fluctuations can induce fatigue loads onto an asphalt revetment. The effect of currents on the different types of asphalt is treated in Section 20.5.
Finally, currents can erode the bottom in front of a dike and the toe can be undermined. This can be prevented by using a bottom protection, see Section 20.6.2.

19.2 Settlement and scouring

19.2.1 *Settlement*

Settlement is caused by the deformation of the dike body and the subsoil. The revetment must, within limits, be able to adjust to irregular settlement. Since it is a viscous material the ability to adjust is determined by its composition. Overfilled mixes, such as mastic, are very flexible, whereas mixes in which the voids are not completely filled with bitumen, such as asphaltic concrete, are less.
If the speed of irregular settlement is great then the asphalt plate cannot adjust immediately and a cavity will develop under the cover layer. Then, if the bending stress caused in the material exceeds the maximum allowable, the structure will fail. This manifests itself in crack development around the edges of the settlement. Large settlements can cause significant changes in the thickness and material properties of the original revetment. Simultaneously other forces, for example, a wave impact on the revetment before it has adjusted to the settlement of the subsoil can cause extra tension and possible structural failure. It is very important that such settlements remain as small as possible. A method of designing a plate-type revetment to cope with settlements is given in Section 20.6.1.
Settlements of the dike body itself, can be limited by using appropriate construction methods: construction of the sandbed in layers, preferably with a certain amount of excess material to be removed by grading, will ensure that it is well-compacted, see also Chapter 18.
Subsoil settlements, to some extent, must be accepted. If unacceptably large settlement is expected, then it may be possible to improve the subsoil or to excavate and replace it. It is important that any subsoil discontinuities, for example, old watercourses, which could lead to uneven settlements, are taken into account (Photo 5, 6, 7). Also joint construction, should not be overlooked in relation to this. Because of faulty construction, material can vanish from the dike body and cavities can develop under the revetment.

In general an asphalt revetment is as most materials unable to adapt to cavities which develop quickly. Ideally the dike design should be such that cavities cannot appear.

19.2.2 *Scouring*

Scouring mostly develops when material in front of the dike is eroded by, for example, waves or currents. Erosion can propagate under the revetment. It can be so extensive and fast that revetments, for example asphaltic concrete and grouted riprap, cannot adjust. The dike should be designed in such a way that scouring cannot develop, with, for

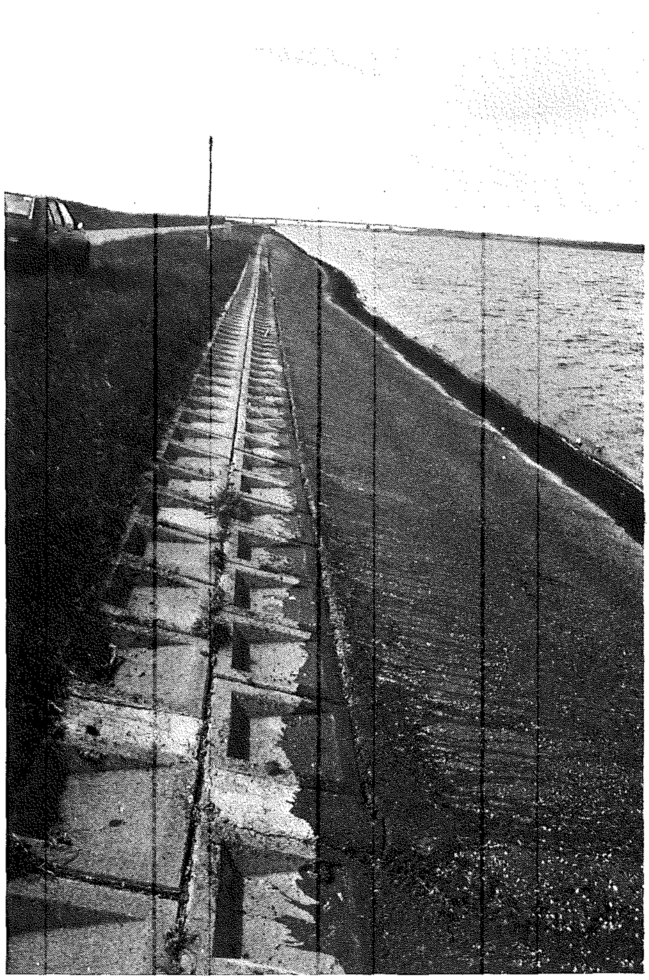


Photo 5 Deformation of an asphaltic concrete revetment caused by irregular settlement of the dike body

Photo 6
The cause of the settlement

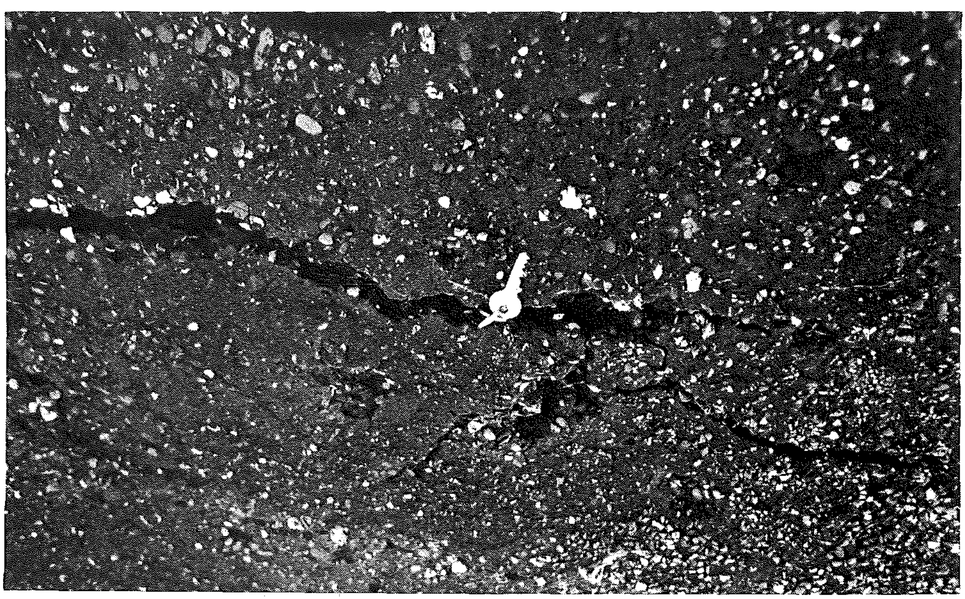


Photo 7 Detail of local damage due to the settlement

example, a suitable toe and bed protection. Asphalt mastic, being a fairly viscous material which can adapt to relatively large scouring, is often used as bed protection. Design methods for this type of application are given in Section 20.6.2., see also Photo 8.

19.3 Other types of 'loads'

Table 19.1 reviews other important 'loading types' which can affect the various revetment materials. The numbers in the table refer to the remarks given below. These factors must be taken into account. In general, however, there are no definite rules which can be applied.

Table 19.1 Review of other load types on an asphalt revetment.

	asphaltic concrete	mastic	pene-tration mortars	dense stone asphalt	open stone asphalt	lean sand-asphalt	mem-branes
biological/chemical attack (1)	x	x	x (1.1)	x	x	x	x (1.2)
collisions	x	x	x	x	x (2.1)	x	
anchors (3)	x	x (3.1)	x (3.1)	x	x	x	
floating debris/ ice (4)	x	x	x (8)	x	x (10)		
recreational activi-ties/vandalism (5)	x		x (8)		x (10)	x	
traffic: (6) during execution after completing	 x x	 x	 x	 x	 x	 x	 x
long-term loads (7)	x	x	x	x	x	x	

Remarks

1 For biological/chemical attack see Section 6.5.
1.1 Affect of the grouting mortar between the stones.
1.2 For membranes made in-situ, which are sensitive to plant penetration from below, the bed should be pre-treated with a planticide. When tubers are present in the soil, however, planticide has been found, generally, to be ineffective.
Plant-resistant foils are often built into prefabricated membranes.
2 Impact loads from colliding ships can not only damage the revetment, but also soften the subsoil so that it loses its bearing capacity and the revetment may slide off. Dimensioning is maybe possible by the models given in 20.3 and Appendix 1.1.1.
2.1 To illustrate:
Ship tests have been carried out in the Rhine-Main-Donau Canal on an 18 cm thick layer of open stone asphalt overlying a 12 cm layer of lean sand asphalt. In the tests a Europa class ship (76.5 × 12 m; total weight = 1,720 ton) was driven at a degress to dike face of 1 : 2.5 at a speed of 1.2 m/s into a dike face; as a result the ship came about 90 cm out of the water. After a few minutes it pulled itself free. Investigations showed that the ship had made a shallow groove, 1 to 2 cm deep, 4 cm wide and 12 to 15 m long, in the dike face. No cracks in the surface were found. Vessels travelling at slower speeds and smaller approach angles didn't cause any noticable damage (29).

3 An anchor which is dragging or has hooked up on the edge of the revetment can cause damage. Falling anchors can punch a hole in the revetment. If the asphalt structure is designed as a watertight layer then this type of damage cannot be allowed and anchoring here must be forbidden. Alternatively the revetment should be designed to resist these loads.

3.1 Sunken pipelines, syphons and underwater mains are occasionally protected against anchors by using a grouted rip-rap or mastic cover.
It has been found that a 1350 kg Danforth anchor will not break through a 900 mm thick layer of rip-rap (10/60 kg). The danger of breaking through is much less with a fully grouted layer of stone. An anchor tends to slide on a mastic apron and, therefore, do no harm. Investigations indicate that the thickness of the protection construction, related to a layer of rip-rap 10/60 kg with a thickness of a m depending on anchor size, should be (26):
- fully grouted a thickness of 0.5a m
- mastic slab a thickness of 1/3a m.

4 Floating debris and ice can hit the revetment and damage it. Damage can be avoided, as much as possible, by:
- using a revetment with a suitable bitumen hardness, a smooth surface, and adequate thickness;
- the timely removal of debris from the dike face slope.

5 Normally a dike is accessible for recreational activities and the design should, therefore, include amongst others:
- special parking places if the revetment is inadequate against the loading of cars
- a hard binder, a minimum number of 'gripping' points and a good quality surface finish to prevent vandalism.

6 The design should take into account traffic on the dike during both the construction phase and on completion. Construction access roads should be provided. In addition the size of the dike and its components, the method of construction and the mixes should be adequate. Specific dimensions can be derived by using conventional road construction methods (94).

7 Since asphalt is visco-elastic permanent features, such as pipelines etc. can cause deformations which eventually may lead to damage. Special provisions should be made.

8 The breaking of stones out of the revetment can be limited by using large stones and good grouting.

9 Since lean sand-asphalt filter layers and cores are ultimately covered, they are only prone to damage during the construction phase. Damage to filter layers during construction, may not be allowed.

10 Individual stones can be jerked relatively easily out of the revetment.

20 Design methods

In the following sections design methods are given for the most common applications of asphalt revetments, including:

- A dense asphalt revetment against hydraulic uplift pressures resulting from quasi-static conditions.
- A plate-type asphalt revetment against wave impacts.
- A dense underwater bed protection against uplift pressures caused by currents and waves.
- A surface- or pattern-grouted crushed stone layer against wave action.
- An asphalt revetment against currents.
- An asphalt revetment adapting to irregular settlement and scouring.

Determination of the maximum dike face slope is also discussed.
Design methods are not given for all types of loading situations. The methods described can probably also be applied to loadings other than those discussed.

20.1 Dense asphalt revetment designed to resist hydraulic uplift pressures

Hydraulic uplift pressures develop under a sealed, absolutely or relatively water-unpermeable dike revetment as a result of differences in water-level inside and outside the dike body. This pressures can force the revetment off the dike face. To prevent this and to ensure that the revetment continues to function it is necessary to take the phenomenon into account in the design.
Air pressure can develop under an airtight dike crest revetment as a result of rising groundwater levels. This pressure can cause the revetment to crack. The effect can be avoided by ensuring that there is good ventilation.
Air pressures act over the whole width of the crest whereas water pressures tends to act locally.

20.1.1 *Hydraulic uplift pressures*

Hydraulic uplift pressures can be caused by:

- Quasi-static conditions:
  - The groundwater level in the dike lags behind the ebb and flood of the tide. As soon as the groundwater level is higher than the water-level outside the dike body there is an hydraulic uplift pressure under the revetment.
  - The largest uplift pressures can be expected after a storm surge when the water-level outside the dike body falls rapidly and the groundwater in the dike falls more slowly.

- Uplift pressures can develop during construction and for some considerable time afterwards if in the vicinity of the revetment, sand is moved by means of hydraulic transport.

– Dynamic conditions:
  - Uplift pressure develops in the dike body when the water-level outside is lowered locally, over a very short time period, by a passing ship.
  - Uplift pressures develop when wind waves produce changes in water-level on the dike face.

Dynamic conditions which can cause uplift water pressures are not discussed further here. Often the time interval in which they act is so small that insufficient water can flow to develop pressures of any consequence. Further information is given in Section 20.3.

A large number of factors influence the development of hydraulic uplift pressures:

- the height, duration, and form of time-dependent boundary conditions such as storm surges and tides in front of the dike, and the potential on the rearside of the dike: the polder level, the drainage ditch level, etc.;
- the permeability and the differences in permeability of the soil in and under the dike body;
- the dike geometry: dimensions; dike face slope; berms; toe level;
- the water storage capacity of the subsoil;
- the level of the foreshore in front of the dike;
- the level of any impermeable layers, for example, clay, in the subsoil;
- the length of any sheetpiling in the toe;
- the presence and type of any drainage systems in the toe or elsewhere in the dike body.
  A drainage system is sometimes installed in the dike to control groundwater movements. By using a toe drainage it is possible to completely or partly prevent the build-up of uplift pressure. It is important that such drainage systems continue to function throughout the lifetime of the structure.

Because many of the above factors are variable it is not possible to give general rules for determining the amount of uplift pressure which can develop. It is always advisable, certainly for large projects, for sites where the subsoil is not homogeneous, when a permeable layer is present under the revetment, or if the revetment design is very different from that schematised in Figure 20.1 to determine the uplift pressures using electrical analogues (52, 53, 85) or a finite element method of calculation (79).
The Van der Veer method can be used to obtain a very preliminary estimation of uplift pressures. This method has, however, drawbacks, the most important being:

- The subsoil must be homogeneous, a situation which rarely occurs in practice.

- The groundwater level has to be estimated.
- The revetment must be schematized as shown in Figure 20.1.

The Van der Veer method often gives values which appear to be too low in comparison with electrical analogue results.

THE ELECTRICAL ANALOGUE

A good method for determining the hydraulic uplift pressures under an asphalt revetment is the electrical analogue. This method, which has been developed extensively, simulates the groundwater flow by electric current. Often the designer does not have an analogue available and advice must then be sought of the appropriate bodies. The best known analogues in the Netherlands are the ELNAG model and the electrical conductivity paper models, (Teledeltos) (52, 53, 85) of the Delta Department of the Rijkswaterstaat. With these analogues time-dependent boundary conditions such as tides and storm surges can be used as inputs, see Appendix III.

THE VAN DER VEER FORMULA

This formula is suitable for a preliminary estimate of uplift pressures. It is based on two-dimensional groundwater flow in a homogeneous subsoil and the presence of the maximim uplift pressure at the location of the outside water-level. The latter occurrence is valid if more than 20% of the revetment lies under water, measured from the bottom edge of the revetment to the level of the phreatic line. The height of the phreatic line must be estimated.

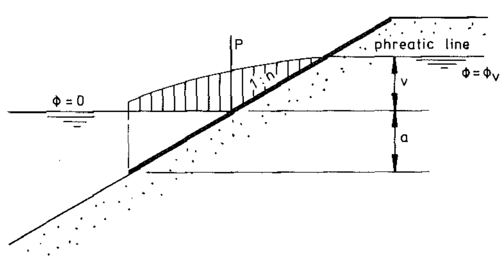


Figure 20.1 Schematization of the revetment for the Van der Veer formula.

The maximum uplift pressure is determined for stationary flow, – constant horizontal supply of groundwater – and non-stationary flow (51). The maximum uplift pressure, P, develops at the waterline:

$$\text{when} \quad \frac{v}{a+v} < 0.8 \text{ to } 0.85$$

and is given by $p = c \cdot \phi_V$ where ϕ_V is the difference between the water-level in front of the dike and the groundwater level in the dike body in the stationary case.

The coefficient c is given by:

stationary flow:

$$c=\sqrt{1-\left(\frac{v}{a+v}\right)^{\pi/\theta}}$$

non-stationary flow

with

$$c=\frac{1}{\pi}\arccos\left[2\left(\frac{v}{a+v}\right)^{\pi/\theta}-1\right] \quad \text{with} \quad \theta=\operatorname{arctg}(n)+\frac{\pi}{2}$$

Where the dike face slope is 1 : n.

In the stationary case, in which the phreatic line remains at a constant level, the uplift pressure is notably bigger than the value obtained with the formula for non-stationary flow.

The affects of a sheet pile wall or a toe protection are shown in Figure 20.2.

The coefficient c then becomes:

stationary flow:

$$c=\sqrt{1-\left(\frac{v}{a+q+v}\right)^{\pi/\theta}}$$

non-stationary flow:

$$c=\frac{1}{\pi}\arccos\left[2\left(\frac{v}{a+q+v}\right)^{\pi/\theta}-1\right]$$

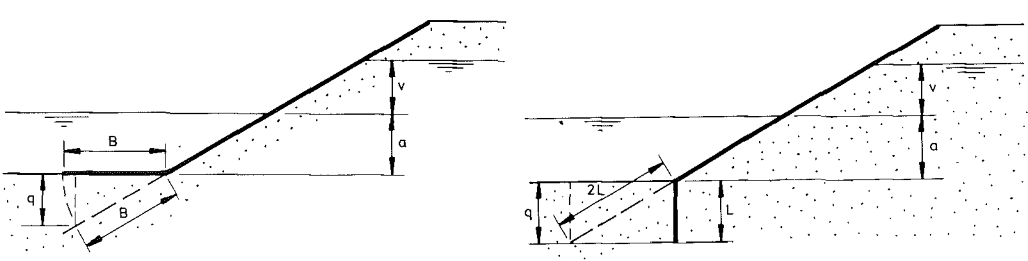


Figure 20.2 Schematization of a toe protection and a sheet pile wall

When making a preliminary estimate for tides and storm surges the parameter v, can be taken as 50% of the difference between the highest and the average outside water-levels. For long-term water-level differences, such as in reservoirs and in high water conditions in rivers v is taken as 100% of the difference.

20.1.2 *Design*

DESIGN CRITERIA

What is the effect of hydraulic uplift pressures on a dense asphalt revetment?

a. If the weight component of the revetment down the dike face is greater than the frictional resistance then the revetment will tend to slide locally. The revetment will hang on higher parts and rest on lower parts where the frictional resistance is still sufficiently large. As a result deformation – stress and strain – will develop (53).
   Because the asphalt is viscous the deformation will be permanent. With a series of loading cycles the deformation can become so large that the material will yield. In addition signs of fatigue will appear.
   The area of greatest uplift pressure will move together with the outside water level down the dike slope. The section of revetment which originally supported the section above will, at a certain condition, also slide. With successive high waters the revetment will, thus, tend to creep like a caterpillar down the slope.
b. If the hydraulic uplift pressure is larger than the weight component normal to the dike face then upward forces will develop which can lift the revetment. In the resulting cavity underneath the revetment, sand movements can take place which prevent the revetment from returning to its original profile. Because these sand movements are downwards there is a tendency for bulges to develop down the slope and subsidences on the upper parts. In view of the characteristics of asphalt the bulging can on the long-term or with regular loading, be considerable and continuing.

The following design criteria can be set out:

1. Sliding criteria
   The revetment should be designed so that it does not slide under frequently occurring loading situations such as spring tides.
2. Uplift criteria
   In loading situations which occur rarely, such as storm surges, the component of the weight of the revetment, normal to the dike face should be greater than the uplift pressure caused by the water.
3. Equilibrium criteria
   The revetment must be in equilibrium as a whole.

Remarks

In order to limit the uplift pressures an open revetment can be used in the area on the dike face where the greatest pressures occur, mostly in the tidal zone.

The uplift water pressures can be schematized as shown in Figure 20.3. The uplift pressure must be determined by the designer. Determination of the maximum uplift pressure (σ_{wo}) is treated in Section 20.1.1. The variation of the pressure can also be determined using an electric analogue or a finite element calculation. The term σ_{wo}, used in the following formulae represents the maximum hydraulic uplift pressure. The Van der Veer method and the electrical analogue give the maximum potential difference, p, in metres of water, at the surface of the revetment. The uplift water pressure, σ_{wo}, is then obtained by adding $h \cos \alpha$ to the value of p and multiplying the whole by $\rho_w g$ (80), see also Appendix III.

$$\sigma_{w0} = \varrho_w g(p + h \cos \alpha)$$

Which indicates that when $p = 0$, the uplift pressure $\sigma_{wo} = \rho_w gh \cos \alpha$

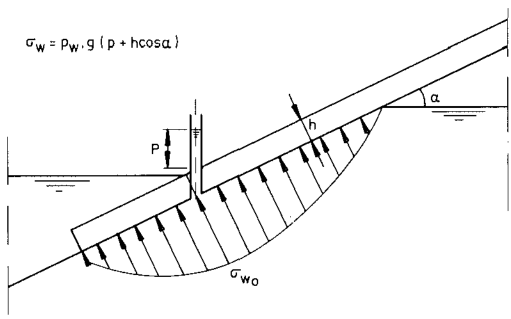


Figure 20.3 Schematization of the water pressure under a sealed revetment

The dimensions of the revetment can be obtained using the following formulas, see also Appendix III.

1. Sliding criterion

$$h \geqslant \frac{f \cdot \sigma_{wo}}{\varrho_a \cdot g \cdot (f \cos \alpha - \sin \alpha)}$$

2. Uplift criterion

$$h \geqslant \frac{\sigma_{wo}}{\varrho_a \cdot g \cdot \cos \alpha}$$

If the revetment is not supported below, for example, by a toe construction or by another revetment then a check must be made to ensure that the tensile strength in the asphalt is not exceeded, see Appendix III. If there is a possibility that it would be exceeded then the layer thickness must be increased and/or the dike face slope reduced. In this case the maximum value of the hydraulic uplift pressure which would be reached in the most extreme conditions can be used.

Symbols used:

h = revetment thickness (m)
σ_{wo} = maximum uplift pressure (N/m^2)
For the sliding criterion σ_{wo} is determined for frequently occurring conditions; for the uplift criterion, conditions which occur very rarely are used
α = slope of dike face (dgr)
ρ_a = asphalt bulk density (kg/m^3)
ρ_w = density of water (kg/m^3)
g = acceleration due to gravity
f = coefficient of friction: $f = \text{tg}\,\phi$ if $\phi > \theta$, else $f = \text{tg}\,\theta$
ϕ = angle of internal friction of the subsoil (dgr)
θ = angle of friction between the revetment and the subsoil (dgr).

3. Equilibrium criterion
See Section 20.7.

20.1.3 *Specific constructional features*

TOE PROTECTION

Mastic, in the form of a horizontal or almost horizontal slab, can be used in front of the toe of a dike, see figure 20.4.
If a mastic slab is sealed directly onto the toe of a dense revetment very high uplift water pressures can develop under the whole construction and a very thick revetment would be required. To avoid this an open 'berm' can be used between the slab and the toe or good drainage must be provided.

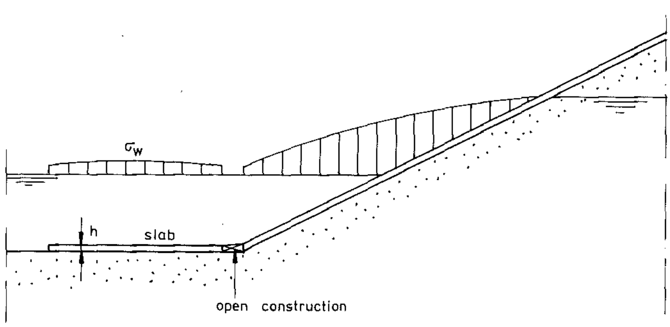


Figure 20.4 Hydraulic uplift pressures under a slab at the front of a dike.

The uplift criterion applies to the slab.
In the first approximation:

$$h \geqslant \frac{\sigma_w}{\varrho_a \cdot g \cdot \cos \alpha}$$

In which:

σ_w = maximum uplift water pressure under the slab (N/m^2)
h = slab thickness (m)
ϱ_a = bulk density of mastic (kg/m^3)
g = acceleration due to gravity (m/s^2)
α = angle of inclination of the slab (dgr)

If it is likely that scouring occurs and the slab has to adjust to this (Figure 20.5), the following checks must be made (4):

a. The uplift criterion in the new situation should not be exceeded.
   N.B.: the uplift pressures will be different in the new situation.
b. The slab should not slide.

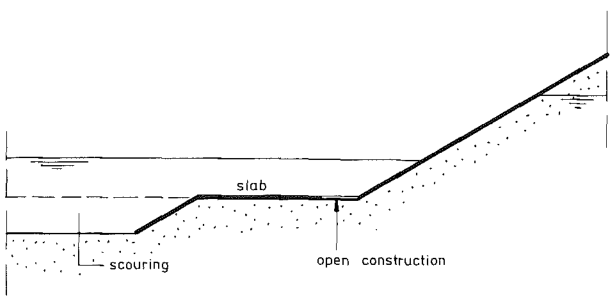


Figure 20.5 Mastic slab modifications after scouring.

GROUTED RIP-RAP (crushed stone)
In the past it was often recommended that, in that part of the dike face where the largest hydraulic uplift pressures would occur and where the sliding criterion would be exceeded (in the Netherlands, in the tidal zone), not to use revetments like asphaltic concrete. Material with a more definite skeleton structure such as (grouted) rip-rap was recommended. With such a material the normal stresses are transferred better to the toe. An asphalt revetment with a definite skeleton structure will behave less viscous than a mix in which all the particles are coated with bitumen. (Another solution, if it is possible, is a water-permeable revetment.)
A fully grouted stone layer has relatively large internal stability. Therefore, in the past, the design was never based on the sliding criterion; under extreme conditions the de-

sign was based, simply on uplift. It is essential that the revetment is well supported by, for example, a toe construction.
A fully grouted stone layer is, in principle, impermeable, although due to lack of adhesion between the stone and the grouting mortar, there will be a certain amount of permeability. No account of this should be taken in the design.

LEAN SAND ASPHALT

The permeability of lean sand asphalt should be similar to or larger than that of the underlying sand bed in order to prevent the development of hydraulic uplift pressures.

ASPHALTIC MEMBRANES

Asphaltic membranes must be watertight and must remain so under the water pressures which develop. The value of the limiting pressure can be found from a permeability test; see, for example (35) or derived from the manufacturer's specifications. It should not be possible for uplift water pressures to lift a membrane, that is, the protection layer should be sufficiently heavy. The dimensions of the protection layer can be found using uplift criteria:

$$h \geqslant \frac{\sigma_w}{\varrho \cdot g \cdot \cos\alpha}$$

In which:

h = thickness of the protection layer (m)
σ_w = uplift water pressure (N/m^2)
ϱ = bulk density of the protection layer (kg/m^3)
g = acceleration due to gravity (m/s^2)
α = slope angle (dgr)

If the membrane is applied on a slope and covered with an asphalt mix, the sliding criterion must also be applied. In addition, tensile forces in the membrane should not be too excessive, see also Appendix VII.

20.2 Design of a plate-type asphalt revetment against wave impacts

The largest forces that waves can exert on a plate-type revetment are impacts caused by plunging breakers.
Plate-type revetments include asphaltic concrete plates, mastic slabs, fully grouted stone layers, open and dense stone asphalt and layers of lean sand asphalt.

20.2.1 *Wave impact loads*

A wave impact occurs when a mass of water from a plunging breaker strikes the slope at great speed. A wave impact is, in fact, regarded as a pressure which acts over a certain width. To obtain the appropriate dimensions the impact is schematized as a line load.

$$P = p \cdot b$$

where:

P = the size of the wave impact (N/m^1)
p = the maximum pressure (N/m^2)
b = the width over which the pressure must act in order to represent the complete wave load (m)

Wave forces are dependent on a large number of factors such as, for example, wave height and steepness, and slope angle. Preferably they should be determined for each particular situation by observations and investigations. If this is not possible use can be made of the data given below. These data have been developed from the results of an investigation carried out by the Delft Hydraulics Laboratory (93) in consultation with Working-Group 1 of the Dutch Technical Advisory Committee on Water Defences. In this investigation into pressure magnitude, pressure width and duration and the location of wave impacts on the slope, only slopes of 1:3 and 1:4 were studied. Using values given in, for example, the 'Voorlopig Rapport 1961' (13) the results can be extrapolated to other slopes. The new wave impact values can differ from those which were used extensively in the past from the 'Voorlopig Rapport'.

The various parameters are given by the following relationships:

– Maximum pressure

$$p = \varrho_w \cdot g \cdot q \cdot H$$

in which:

ϱ_w = density of water (kg/m^3)
g = acceleration due to gravity (m/s^2)
H = wave height (m)
q = a factor related to the slope (see table below)

slope	q
1 : 2	2.3
1 : 3	2.7
1 : 4	2.3
1 : 6	2

– The schematized width over which the maximum pressure is considered to act:

$$b = 0.4 \cdot H$$

- The duration of the pressure effect, t sec, is, depending on the particular model:
  slope $\leq 1:3$; $t = 0.06 \cdot H^{1/2}$
  slope $\geq 1:4$; $t = 0.18 \cdot H^{1/2}$
- The length of the wave impact is dependent on the angle of wave approach to the slope and the speed of propagation of the wave. The larger these factors are, the shorter is the impact length.
- A breaking wave hits the slope at a distance Δh below still water level (SWL), see Figure 20.6. The 'area' in which Δh lies is shown in Figure 20.7.

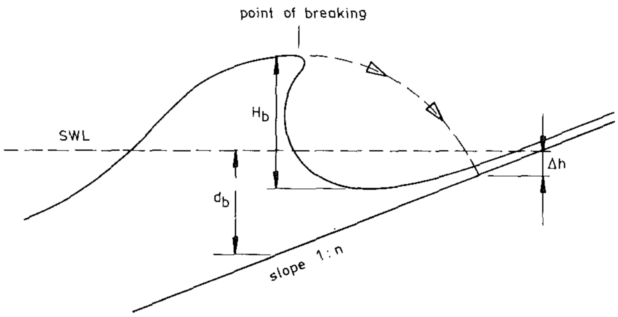


Figure 20.6 The breaking wave.

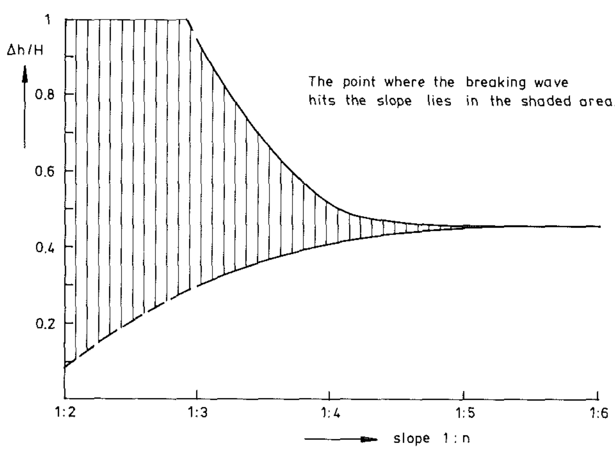


Figure 20.7 The point, Δh below SWL, where the wave impact hits the slope, related to the wave height H and plotted against the slope $1:n$.

In the above the parameter H is the height of a single wave. In practice the load on the revetment will be due to an irregular wave train containing a large number of waves of different height and frequencies of occurrence. The significant wave height, H_S, which characterises a particular wave field can be used for determining the wave impact. The number of times that this wave occurs is selected so that the same total 'fatigue load' is reached as that caused by the wave field as a whole, see Appendix I.2. The calculation of wave impact should take into account the fact that only a small number of waves in the field will cause actually an impact on the revetment.

20.2.2 *Construction schematization*

It is important that the schematization of the construction approximates as much as possible to reality. The most simple schematization is that of a plate lying on an elastic subsoil with a delayed response. This is developed in Appendix I.1.1.
The method treated in this Chapter is suitable for wave impacts and also for other types of loads which can act on the revetment. These include, for example, loads caused by colliding vessels, maintenance and recreations traffic, long-term loads and loads during the construction phase.
The loads can be:

a. Static; that is loads which are always present. The way in which a construction reacts to static loads depends, amongst other things, on the size of the load, the stiffness of the revetment and the subsoil, and the thickness of the revetment.
b. Dynamic; that is time-dependent loads. In addition to the factors mentioned above under static loads the speed, frequency and type of loading, the density of the revetment material and the damping and directly related mass of the subsoil are also important. When the revetment is frequently loaded the asphalt properties are altered: the strain at break reduces. This phenomenon is known as fatigue.

In this manual the static solution is applied to the selected schematization, see Appendix I.1.1. The number of loading cycles is taken into account in the calculation of the breaking strength of the material. The duration of loading is incorporated into the stiffness modulus of the asphalt mix.
The loading is schematized as a line load. Since, in practice, the wave impact is a distributed load, corrections are applied.
This schematization is not so suitable for complicated constructions (varying layer thickness, multi-layer systems, joints etc.). In these situations a more extensive calculation involving, for example, finite element methods can offer the best solution.

20.2.3 *Asphalt and subsoil properties*

When designing it is essential to know the asphalt and subsoil properties. Preferably data should be obtained by carrying out specific tests for each design. If such tests are not possible, use can be made of the general values given in Tables 20.1 and 20.3. The materials used in the construction should then be carefully checked against these values.

Table 20.1 Moduli of stiffness and related initial stresses and strains at break for different asphalt types. (With the exception of open stone asphalt, the values are determined using Figures 6.1., 6.3., and 6.4. with which values for the lower loading cycles have been extrapolated).

mix type	stiffness modulus N/m^2	Initial strain at failure					Initial stress at break				
		number of loading cycles					number of loading cycles				
		1	100	1000	10000	100000	1	100	1000	10000	100000
Asphaltic concrete	$7 \cdot 10^9$	$1.2 \cdot 10^{-3}$	$5.2 \cdot 10^{-4}$	$3.4 \cdot 10^{-4}$	$2.5 \cdot 10^{-4}$	$1.6 \cdot 10^{-4}$	$8.4 \cdot 10^6$	$3.6 \cdot 10^6$	$2.4 \cdot 10^6$	$1.8 \cdot 10^6$	$1.1 \cdot 10^6$
Asphaltic mastic	$1 \cdot 10^9$	$8.6 \cdot 10^{-3}$	$3.4 \cdot 10^{-3}$	$2.2 \cdot 10^{-3}$	$1.4 \cdot 10^{-3}$	$8.6 \cdot 10^{-4}$	$8.6 \cdot 10^6$	$3.4 \cdot 10^6$	$2.2 \cdot 10^6$	$1.4 \cdot 10^6$	$8.6 \cdot 10^5$
Dense stone asphalt	$4.5 \cdot 10^9$	$2.3 \cdot 10^{-3}$	$9.2 \cdot 10^{-4}$	$5.8 \cdot 10^{-4}$	$3.7 \cdot 10^{-4}$	$2.3 \cdot 10^{-4}$	$1 \cdot 10^7$	$4.1 \cdot 10^6$	$2.6 \cdot 10^6$	$1.6 \cdot 10^6$	$1 \cdot 10^6$
Open stone asphalt	$7 \cdot 10^8$	$3.4 \cdot 10^{-3}$	$1.3 \cdot 10^{-3}$	$7.9 \cdot 10^{-4}$	$4.8 \cdot 10^{-4}$	$3.0 \cdot 10^{-4}$	$2.4 \cdot 10^6$	$9.1 \cdot 10^5$	$5.5 \cdot 10^5$	$3.4 \cdot 10^5$	$2.1 \cdot 10^5$
Lean sandasphalt	$1 \cdot 10^9$	$1.1 \cdot 10^{-3}$	$4.2 \cdot 10^{-4}$	$2.6 \cdot 10^{-4}$	$1.7 \cdot 10^{-4}$	$1.1 \cdot 10^{-4}$	$1 \cdot 10^6$	$4.2 \cdot 10^5$	$2.6 \cdot 10^5$	$1.7 \cdot 10^5$	$1 \cdot 10^5$

Table 20.2 Mix compositions of the asphalt types given in Table 20.1.

Mix	Composition (by mass %)				Voids ratio (%v)
	crushed stone	sand	(very) weak filler	bitumen 80/100	
Asphaltic concrete	46.9	39.5	7.5	6.1	5
mastic	–	64	17	19	0
Dense stone asphalt	60	25.6	6.8	7.6	5
Open stone asphalt	82.9	9.9	4.4	3.1	31
Lean sand asphalt	–	96	–	4	30

The asphalt properties – modulus of stiffness and stress at break – are temperature and loading duration dependent. Since the heavy storms, against which the revetment is mostly designed, occur mostly in the winter season a temperature criterion of 5°C can be accepted (13). The loading duration is wave height-dependent and can be deduced from Section 20.2.1. Variations in this parameter, within practical limits, have little effect on asphalt properties.

Some properties of the most used asphalt types are given in Table 20.1.; the mix compositions of these types are given in Table 20.2. Bitumen 80/100 is used by which an unfavorable loss in penetration is taken into account. The temperature is fixed at 5°C and the loading period 3 seconds.

The stresses at break for loading cycles of less than 10,000 are found by linear extrapolation on a log-scale. Although this method is not completely correct it gives presumably safe values.

Table 20.3 General values for the modulus of subgrade reaction of different soil types.

soil type	modulus of subgrade reaction c (N/m^3)
sand – medium compacted (relative Proctor density 87-95) – well compacted (relative Proctor density 95-100)	1×10^7–1×10^8 1×10^8–3×10^8
sand + clay	3×10^7–8×10^7
sand + silt	2×10^7–5×10^7
clay – low compressibility – high compressibility	3×10^7–5×10^7 $< 4\times10^7$
Peat	$< 5\times10^7$
Gravel	$> 7\times10^7$
Lean sand asphalt	$> 5\times10^8$

For strongly deviating mix compositions, temperatures and loading conditions the mix compositions can be determined using the method given in Part A, Sections 6.3.1. and 6.3.2. The Poissons ratio of the mix follows from Section 6.3.4.
The parameter defining the subsoil is referred to as the modulus of subgrade reaction. General values of this parameter are given in Table 20.3. (10, 50).

20.2.4 *Design*

Design criteria are selected such that the stresses and strains developing in an asphalt plate of a certain thickness due to bending moments do not exceed the allowable values. Plate thickness can be determined using the calculation model developed in Appendix I.1. for a plate of constant thickness on a delayed reacting elastic subsoil.
The formula reads:

$$h = 0{,}75 \cdot \sqrt[5]{\frac{27}{16} \cdot \frac{1}{(1-\nu^2)} \cdot \left(\frac{P}{\sigma_b}\right)^4 \cdot \left(\frac{S}{c}\right)}$$

in which:

h = thickness of revetment (m)
σ_b = asphalt stress at failure (N/m^2)
P = wave impact (N/m^1)
S = stiffness modulus of the asphalt (N/m^2)
ν = Poisson ratio for asphalt
c = Modulus of subgrade reaction (N/m^3)
0.75 = reduction factor, see Appendix I.1.1.

Usually the revetment is designed for a design parameter such as the significant wave height, H_s, which characterises the wave climate in a severe storm or at an extremely high design water level.
If the revetment is also subjected to loads of a normal daily wave climate then this should be taken into account in the layer thickness calculated with the design H_s. A method for doing this which gives safe values for Dutch conditions is given in Appendix I.2.3. Distinction should be made between:

1. That part of the revetment on which only the design condition, H_s, acts. No correction is necessary here.
2. That part on which the design conditions act together with the normal wave climate.
3. That part on which only the normal wave climate acts (and not the design wave).

That part of the revetment which, for example, lies above the (spring) tide zone is classed in Category 1. The revetment in the (spring) tide zone is acted on by the normal wave climate but, in general, not by the design wave, H_s, which occurs at higher water levels.

This part is classed in Category 3. Dikes with a deep foreshore can be classed in Category 2.

20.2.5 *Practical application of the wave impact formula*

General values of layer thickness are given in Figures 20.8. to 20.12. for some standard mixes, for various significant design wave heights and subsoil parameters. These values are based on assumptions for application in Dutch conditions, see Appendix I.2.2.
The assumptions are:

- the design storm has a duration of 36 hours (3 tidal cycles)
- the number of wave impacts used for dimensioning is 10% of the total number of waves in the design storm
- the waves in the storm are generated in a wave field which has a Rayleigh distribution
- the relationship between the significant wave height H_s (m) which characterises the storm and the average wave period in the storm, $\bar{T}$(sec) is $\bar{T} = 3.5 \times H_s^{0.5}$
- the wave impact is determined from Section 20.2.1. in which $\rho_w = 1000$ kg/m^3 and $g = 9.81$ m/s^2
- the asphalt properties are given in Table 20.1. The number of loading cycles, n_s, related to the significant wave height, H_s, needed to determine the strength at break, is given in the table below.
  A value of 0.35 is taken for Poisson's ratio.

H_s (m)	n_s
2	9900
3	8000
4	6950
5	6200
6	5670
7	5250
8	4900
9	4630
10	4400

In order to obtain an optimum design or if the design conditions differ greatly from these assumptions, then the calculation of layer thickness can deviate from the method used for Figures 20.8. to 20.12.

Adaptations can be made by:
- changing the mix composition;
- treating/compacting the subsoil;
- changing the dike geometry.

It is then essential to carry out extra checks on site and in the laboratory. This will also indicate the programme for the execution requirements.

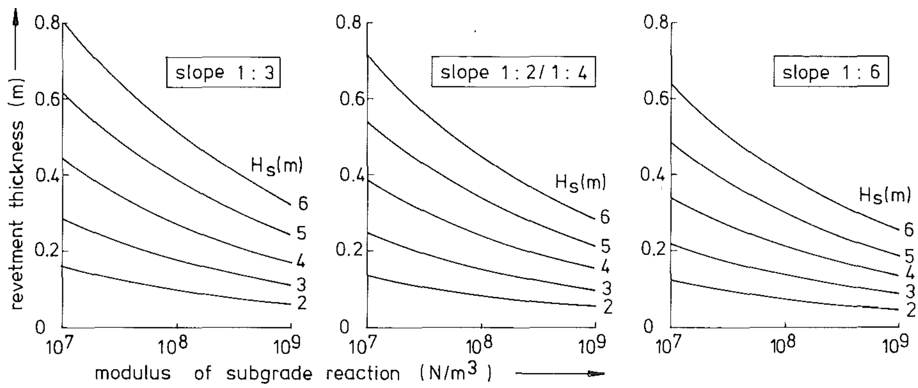


Figure 20.8 Necessary layer thickness for a revetment of asphaltic concrete, plotted against the modulus of subgrade reaction and for various significant wave height and slopes (see the text above for the basis of the graph).

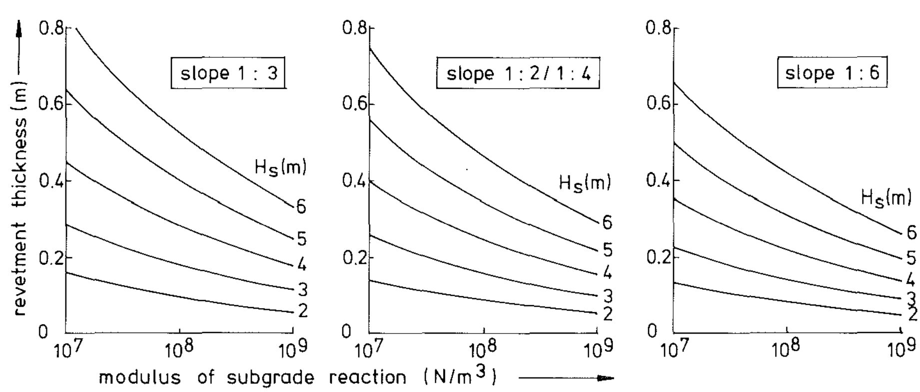


Figure 20.9 Necessary layer thickness for a revetment of mastic, plotted against the modulus of subgrade reaction and for various significant wave heights and slopes (see the text above for the basis of the graph).

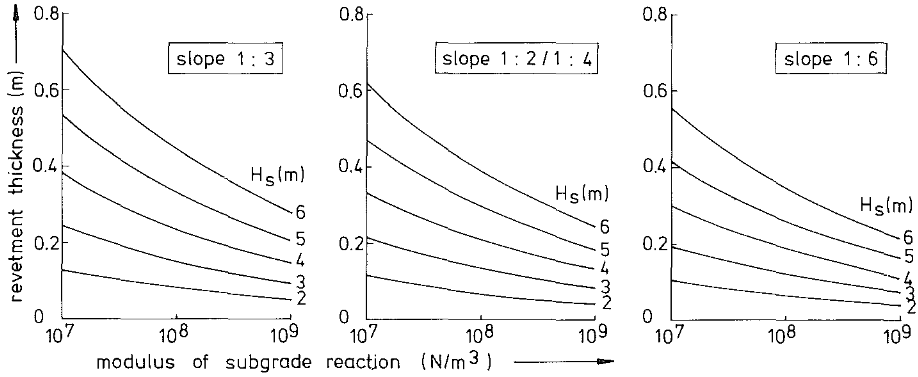


Figure 20.10 Necessary layer thickness for a revetment of dense stone asphalt plotted against the modulus of subgrade reaction and for various significant wave heights and slopes (see the text above for the basis of the graph).

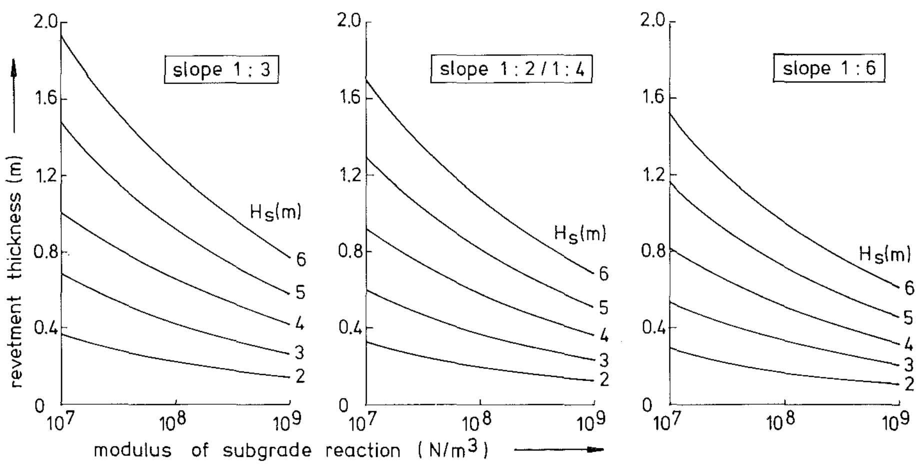


Figure 20.11 Necessary layer thickness for a revetment of open stone asphalt plotted against the modulus of subgrade reaction and for various significant wave heights and slopes (see the text above for the basis of the graph).

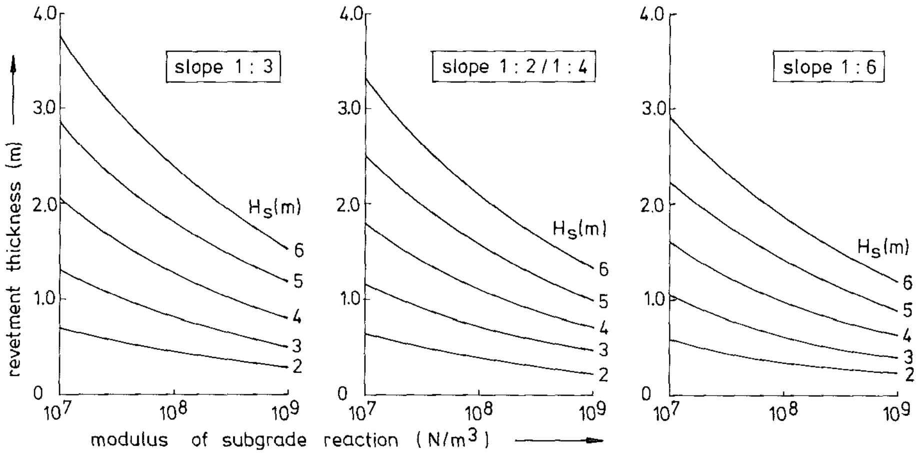


Figure 20.12 Necessary layer thickness for a revetment of lean sand asphalt plotted against the modulus of subgrade reaction and for various significant wave heights and slopes (see the text above for the basis of the graph).

The following remarks should be noted for certain specific materials:

- Fully grouted stone
  A fully grouted stone layer has, in general since it is composed of several layers of stone, considerable thickness. Because of this it is in most cases not necessary to design on the basis of wave impacts.
  The design of a fully grouted stone layer against wave impact in which the revetment is considered as a plate, is discussed in Appendix I.3.
  Design is based on the normal wave impact formula using the material properties of the grouting mortar (mastic). In order to obtain the design layer thickness the value calculated must be multiplied by a factor lying between 1.4 and 1.75. These factors are valid if the shear stresses which develop between the mastic and the crushed stone can be transferred. It is not immediately obvious to what extent this condition is satisfied.

- Open stone asphalt
  Because of the open character of open stone asphalt the wave pressure can quickly propagate through the revetment the result being that the load on the plate is less than that indicated in Section 20.2.1.

- Lean sand asphalt applied as a core material
  The performance under wave impact of lean sand asphalt applied as a core material can be calculated using the Boussinesq approximation, see Appendix V.

20.3 Design of underwater bed protection against hydraulic uplift pressures caused by currents and waves

20.3.1 *Uplift pressures caused by currents*

A mastic slab is often used to protect the bed against erosion caused by currents.
In extreme situations fluctuations in the flow of water and groundwater can create pressure differences across a bed protection. If the pressure above a revetment plus its weight is less than the pressure underneath, the revetment will tend to be lifted (81).
This should be prevented. If a slab lifts a cavity develops underneath into which water flows, the extent depending on the duration of the pressure difference. Because of the viscous properties of the asphalt mix it will deform. Depending on the duration and the quantity of inflowing water, the deformation can be so large that the slab breaks.
A simple solution to this problem cannot be given since the pressures which develop vary from situation to situation. For simple cases the water pressure can probably be estimated using simple formulas. For more complicated cases an extensive calculation programme or an electrical analogue can offer the solution.
More important than the lifting of the revetment by excess water pressure is the stability of the edges of a bed protection or mattress, see Section 20.3.2 and 20.5.

20.3.2 *Hydraulic uplift pressures due to wave action*

An impermeable bed protection slab, lying on sand, can be lifted by differences in pressure above and below the slab caused by wave action (41, 79). This must be prevented.
Wave action causes groundwater movements under the bed protection which change the groundwater pressure. These changes, however, are not necessarily the same as those which occur above the slab and an upward pressure can result.
Two situations can be identified:

a. The wavelength is longer than the bed protection.
The maximum pressure under the revetment, in this situation, can be estimated using Barends (79). Obviously the weight of the revetment must be greater than the uplift pressure. The following relationship is derived in Appendix II:

$$h \geqslant \frac{\varrho_w}{\varrho_a} \cdot \frac{H}{2} \qquad (l < L)$$

in which:

h = thickness of bed protection (m)
ϱ_w = density of water (kg/m^3)

ϱ_a = bulk density of bed protection material (kg/m^3)
H = wave height (m)
l = length of the bed protection in the wave direction (m)
L = wavelength (m)

b. The wavelength is much shorter than the length of the bed protection.
In order to prevent the slab from being raised its weight must be greater than the maximum uplift pressure underneath. An uplift pressure is caused by the pressure resulting from the wave action above the slab being, locally, less than the groundwater pressure below. The time-dependent pore water movements, which strongly determine the groundwater pressures, cause considerable damping to this effect (41, 79). An approximation for this system is given in Appendix II. With the formula derived, it is possible to estimate the thickness of revetment required. In the example, worked out in the appendix, it can be seen that the damping due to groundwater movements is very large.
Under normal conditions the phenomena of lifting by wave action is not of major importance. More important is the possibility of scouring at the edges of the bed protection as a result of erosion. If the sand at the edges is unstable it is recommended that the watertight bed protection is overlapped at the edges with an open sand-tight revetment.

20.4 Design of a surface- or pattern-grouted stone layer against wave attack

A much-used formula for calculating a construction built up out of discrete elements, for example, quarry stone, to resist wave attack is the empirical Hudson formula (44):

$$W = \frac{\varrho_a \cdot g \cdot H^3}{K_D \cdot \Delta^3 \cdot \operatorname{cotg} \alpha}$$

in which:

W = weight of an element (N)
ϱ_a = density of an element (kg/m^3)
g = acceleration due to gravity (m/s^2)
H = wave height (m)
Δ = relative density of an element

$$\Delta = \frac{\varrho_a - \varrho_w}{\varrho_w}$$

ϱ_w = density of water (kg/m^3)
K_D = damage coefficient, which takes into account the shape, degree of interlock, roughness and location of the revetment on the slope.
α = slope angle.

Remarks

1. The Hudson formula is valid for slopes with $\alpha < 33.7°$.
2. The formula has been developed for breakwaters, subjected to non -breaking waves. The effect of breaking waves can be introduced by lowering the value of K_D (86).
3. The formula is only valid for revetments on the front face of the breakwater.
4. The wave conditions are characterized by a single parameter, H. Investigations (87) indicate that the damage, that is the number of revetment elements removed, is fairly independent of the storm duration except when the design wave height is exceeded by over 30%.
5. The Hudson formula is designed for regular waves. Investigations (87) show that damage caused by a wave spectrum characterized by a significant wave height H_{sig}, is larger than that caused by regular waves of the same height. This difference becomes more pronounced as the spectrum width is increased.
6. The wave period is not taken into account.

The value of the damage coefficient K_D can be increased by grouting a crushed stone construction with asphalt mortar.

The following applications can be distinguished:

- Surface-grouted stone
  If the size or weight of crushed stone is barely inadequate to satisfy the wave conditions then the safety of the revetment can be increased by fixing the stone with an asphalt grout.
  If about 30% of the voids in the stone is covered, the K_D value can be multiplied by 1 to 1.5.
  Note: In principle the revetment does not consist of individual elements and this method should not, in fact, be applied. In practice, however, it works well and gives satisfactory results.
- Pattern-grouting
  If about 60% of the total surface is filled the K_D factor can be multiplied by 5 to 7. From model investigations it appears that a relatively smaller increase in stability is obtained by grouting more than 50% of the surface of the crushed stone. Grouting 50 to 70% of the total voids in the stone layer appears to give the best results (27).
  The revaluating of the K_D factor is very dependent on the execution and care must be taken to ensure that the grout does not remain in the surface of the stones or sags completely through the cover layer.
  Prototype tests with high waves, H_{sig} up to 8 m, demonstrate that, because of the enormous forces developed by breaking waves in the clefts between the stones, the grouted lumps themselves can be split and shell-shaped pieces can brake off. This type of construction, therefore, should not be used in areas of heavy wave attack; it has proved to be successful, however, in waves of 3 to 5 m (4).

For reasons of safety it is recommended that three layers of broken stone are used, only the top two being grouted. A few loose stone will probably be washed away by the waves but this will not be dangerous. If a lump of grouted stones is washed away the third layer will still protect the core since it is held fast by the overlying grouted lumps (27).
Ideally the K_D value should be found from model tests.
Grouting produces a smoother revetment surface and, as a result, there is more wave uprush. The crest, therefore, needs to be higher. An indication of this is given in Table 20.4.

Table 20.4 Wave uprush on certain slope surfaces compared with a smooth surface.

surface type	$\frac{\text{run-up on the surface}}{\text{run-up on a smooth surface}}$
smooth: impermeable	1.0
rip-rap	0.5-0.6
pattern grouted	0.6-0.7
fully grouted	0.6-0.8

20.5 Design of an asphalt revetment against currents

The possible forces which can be caused by flowing water are given in Section 19.1.3. The performance of various asphalt types under such forces is discussed in the present section.
Under normal conditions an asphalt revetment is very resistant to flowing water. Considerable damage however can develop, for example, if the water carries hard objects such as stones. In addition currents can lift the edges of a plate or mattress and cause to turn them over. This can be prevented by, for example, increasing the weight of the edges or by burying them.

ASPHALTIC CONCRETE

Asphaltic concrete is only attacked slightly by currents. If the water, however, carries hard particles, impacts can occur which damage the revetment material.
In Los Angeles it was found that, in the prevailing temperatures with a minimum of about 10°C, an asphaltic concrete revetment can resist the erosive effect of debris in the water if the binder is softer than penetration 50. For Dutch conditions, with a minimum temperature around the freezing point, bitumen 80/100 is more suitable (12).

MASTIC

Mastic is an overfilled mix with a relatively low stiffness. The stresses caused by impacts from debris etc. carried by the flow are generally small and can be withstood. It is important, however, that the edges of the plate are not made to flap by the current. This can be prevented by:

1. burying the edges of the plate so that the current can gain no purchase on it.
2. dumping crushed stone on the edges. These pieces will then penetrate into the mastic layer until they are in equilibrium. This process is dependent on the difference in density between the stone and the mastic, the shape of the stones and the viscosity of the mastic. (The penetration into a layer of stones takes longer than of loose stones.) Investigations indicate that a few centimeters of mastic slab originally 20 cm thick still remained under the stone, dumped under water, placed five years ago (88).

A mastic slab can be built up from a number of separate layers laid like rooftiles over each other. If there is not good adhesion between these layers, because of the presence of sand pollutions or inadequate heat transfer, flowing water can get hold of the unattached sections, lift them and break them off (81).

GROUTING MORTARS

The following formula is often used for designing loose crushed stone against stationary or quasi-stationary flow (25):

$$D_{50} \geqslant b \frac{\bar{U}^2}{2g\Delta} \{\cos\alpha \cdot \sqrt{1 - \mathrm{tg}^2\alpha/\mathrm{tg}^2\phi}\}^{-1}$$

in which:

D_{50} = median diameter of the revetment material (m)
$\bar{U}$ = current velocity parallel to the axis of the channel (m/s)
α = slope angle
Δ = $\frac{\varrho_s - \varrho_w}{\rho_w}$
ϱ_s = density of the revetment material (kg/m^3)
ϱ_w = density of the water (kg/m^3)
ϕ = angle of internal friction of the revetment
g = acceleration due to gravity
b = a stability parameter

The stability parameter is dependent on many factors. With a uniform, continuous flow, for conditions which occur in the Dutch waterways it is, on average, 1.4. For other situations reference should be made to literature (25).
With grouted crushed stone, the stability parameter b can be lowered. For preference, the value of b should be established for each particular situation by model tests.
The effect of currents on fully grouted stone is negligible.

DENSE STONE ASPHALT

Dense stone asphalt is an overfilled mix and, thus, very resistant to currents. The design should be such that the edges do not flap.

OPEN STONE ASPHALT

Information about the resistance of open stone asphalt to currents is, as yet, not complete. Investigations have produced the following results:

- With the stationary and quasi-stationary flow very limited erosion was observed after 34 hours with current velocities of 6 m/s, (6). The damage which did occur consisted mainly of the loosening of limestone particles.
- The material has also been tested with currents generated by ships' screws. These tests were made on a bank revetment of 12 cm lean sand asphalt under a cover of 18 cm open stone asphalt in the Rhein-Main-Donau Canal. The currents were produced by the screws of a cargo ship with a draught of 2 m and an 800 HP motor at full strength for 5 minutes. No damage ocurred.
- Investigations have been carried out into the resistance of open stone asphalt to wave attack, which showed that under normal tide conditions and also over a long period the material did not erode.

The edges of an open stone asphalt mattress can flap when the critical steady flow velocity (2.5 to 3 m/s for a 15 cm thick mattress) is exceeded. This can be prevented by making the edges heavier. This phenomena of flapping can occur with all types of slabs and mattresses.

LEAN SAND ASPHALT

Indicative tests have shown that loosely dumped lean sand asphalt is resistant to currents upto 3 m/s (32). This resistance to currents can be increased by raising the bitumen content and by a certain amount of compaction. When lean sand asphalt is used as a core material there is generally only need to take flow precoutions during the construction phase. The loss of a certain amount of core material during construction can be accepted provided that, on completion, the core is sufficiently large.

When lean sand asphalt is used as a filter layer it must only be exposed to direct currents during construction. This phase should be as short as possible.

If lean sand asphalt is used as a permanent revetment there must be no loss of material. Therefore the waves and currents to which it is exposed should not be to large. Tests indicate that the maximum current velocity is about 3 m/s. More investigations into this aspect are desirable.

Erosion can possibly be restricted by compacting or adapting the mix or providing it with a seal coat. These measures can affect the water permeability. Crushed stone in the weight range upto 60-300 kg can be laid directly on lean sand asphalt as a protection layer. By experience it appears that larger stones cause turbulence which can erode the lean sand asphalt. To prevent this an intermediate layer of, for example, gravel can be used.

20.6 Designing an asphalt revetment against irregular settlement and scouring

20.6.1 *Irregular settlement*

Depending on the speed of settling and the properties of the asphalt, an asphalt revetment will not immediately adapt to irregular settlement. After some time, however, provided that there is no break, the revetment will lie again on the subsoil. The bending of the asphalt plate due to irregular settlement and the time taken for it to adjust can be determined using simple formulas derived by mechanics (50).
If the surface of the settlement is assumed to be circular, the time taken for the revetment to adjust, t, can be determined using the following formula. By repeating the calculation a number of times with related values of the stiffness modulus, t can be approximated:

$$\frac{t^3}{S(t,T)} = \frac{16 \cdot v \cdot h^2}{3 \cdot \varrho_a \cdot g \cdot (1 - \upsilon^2) \cdot u^4} \qquad \text{(see Appendix IV.1)}$$

in which:

ϱ_a = density of the asphalt (kg/m^3)
g = acceleration due to gravity (m/s^2)
υ = Poisson's ratio for asphalt
h = thickness of the revetment (m)
u = horizontal speed of settling (m/s)
v = vertical speed of settling (m/s)
t = time (s)
time at which settling begins: $t = 0$
S = stiffness modulus of asphalt, time and temperature dependent. The related time, can for example, be fixed at 0.5 t

The speed of settling (v and u) is assumed to be constant, see also Figure 20.13.

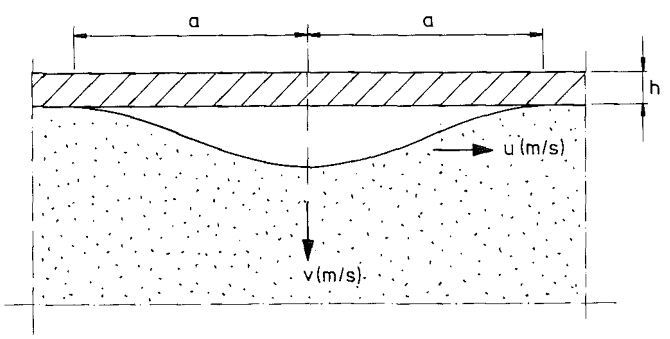


Figure 20.13 Settlement under an asphalt revetment (schematized).

The largest bending stress will develop just before the plate touches the bed, see Appendix IV.1. The bending stress is then:

$$\sigma_b = \frac{3 \cdot \varrho_a \cdot g \cdot u^2 \cdot t_1^2}{4 \cdot h}$$

t_1 = adjustment time

If the expansion of the settling has stopped before the revetment can adjust:

$$\sigma_b = \frac{3 \cdot \varrho_a \cdot g \cdot a^2}{4 \cdot h}$$

a = See Figure 20.13

If the allowable stress at break, $\sigma_{b\ max}$, is smaller than the stress which develops, σ_b, then the revetment will fail before the material can adapt to the subsoil surface.
The bend in the plate can also be so big that, after a long time, the deformation capacity is exceeded and the plate breaks.
The presence of a cavity under the revetment can in conjunction with other loads, for example, a wave impact, cause extra stress. This can lead to failure.
In addition large settlements can lead to a reduction in the layer thickness (viscous flow) which reduces the strength of the revetment.
If the asphalt revetment is to follow irregular settlement without cracking this will depend on the speed of settling and the asphalt properties.

- Asphalt mastic is a very suitable material for adjusting to irregular settlement because it is reasonably viscous. In the above mentioned formulas the stiffness modulus S can be replaced by $3\eta/t$ in which η is the value of the viscosity of the mastic, see Appendix IV.2.
- Grouted crushed stone must be able ot adjust to irregular settlement without losing its cohesion. It is more able to do this when the voids are completely filled.
- It is important that open stone mattresses remain in contact with the subsoil. If the mattress is anchored, stresses can develop in certain sections which in combination with wave induced forces can lead to failure.
- Because of its limited ability to adjust and in view of its function great care is needed when using lean sand asphalt as a filter layer or revetment. Failure will not develop, initially, because of large deformation but because of too great a deformation speed. Because of its large bulk when lean sand asphalt is used as a core material, its ability to adapt to settlement is of much less significance than that of a sand asphalt filter layer. The core should be designed to act monolithically under differential settlement, see also Appendix V. The tension level and the deformation speed should, however, be checked.

– An asphaltic membrane which adjusts to settlements will stretch, see Figure 20.14. If the extension is Δl and the original length l then the strain is $\Delta l / l$ (that is $\Delta l = \sqrt{\Delta h^2 + \Delta x^2} - \Delta x$).
This strain must not exceed the maximum allowable value.
Reinforcing in the membrane enables the strain which otherwise would concentrate in one place and could lead to excessive extension.

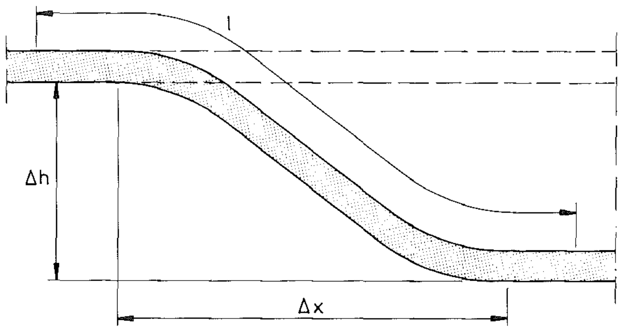


Figure 20.14 Extension of a membrane.

20.6.2 *Scouring*

Scouring usually takes place so quickly that the asphalt revetment cannot adjust. The dike should, therefore, be designed in such a way that the revetment cannot be undermined, for example, by providing a good toe and bed protection.
Because of its good viscosity, asphalt mastic can, to a large extent adjust to undermining, see Photo 8, and is, therefore, often applied as bed protection.
If scouring occurs at the end of a mastic apron it will bend, see Figure 20.15. The time for the end of the plate to reach the bottom of the hole can be determined from the following:

$$t = \sqrt[5]{\frac{2 \cdot h^2 \cdot z \cdot \eta}{\varrho_a \cdot g \cdot (1 - \upsilon^2) \cdot v^4}} \qquad \text{(see Appendix V.2.)}$$

in which:

h = thickness of the revetment (m)
η = viscosity of the mastic (Pa · s)
ϱ_a = density of the mastic (kg/m³)
g = acceleration due to gravity (m/s²)
υ = Poisson's ratio for asphalt
v = horizontal scouring speed (m/s)
z = depth of scour (m)
t = time when bending begins: $t = 0$ (s)

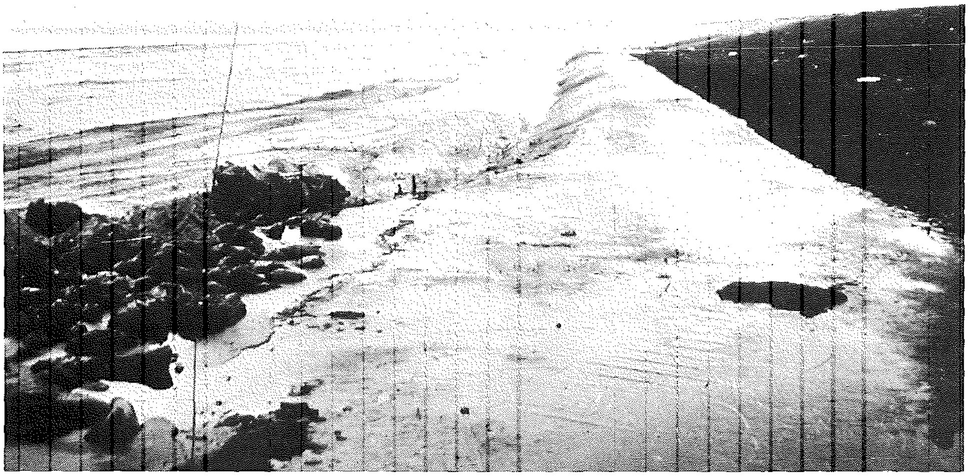


Photo 8 The effects of scouring on a mastic slab

The speed of scouring is assumed to be constant.

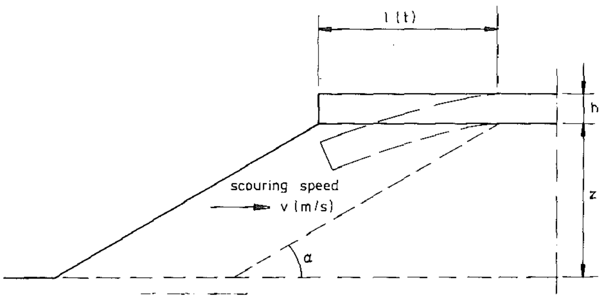


Figure 20.15 Scouring of a mastic slab (schematic).

The largest bending stresses will develop at a distance $l(t)$ from the end of the slab just before the end of the slab settles onto the bed of the scour hole. If these stresses exceed the limiting value the revetment will break.

$$\sigma_b = \frac{3 \cdot \varrho_a \cdot g \cdot v^2 \cdot t^2}{h} \qquad \text{(see Appendix V.2.)}$$

in which:

σ_b = bending stress (N/m²)
t = adjustment time (s)

As soon as the end of the slab touches the bed, in principle, the scouring process stops. The mastic slab, however, does not lie everywhere in contact with the bed and in between the slab and the bed there may be cavities.
The length over which the slab is not supported is increased since the bed underneath adjusts to its own natural flatter slope. The slab will then bend further to complete its adjustment. New stresses develop in the slab which in combination with the reduced slab thickness can lead to failure. In practice a crack can develop at A, see Figure 20.16.

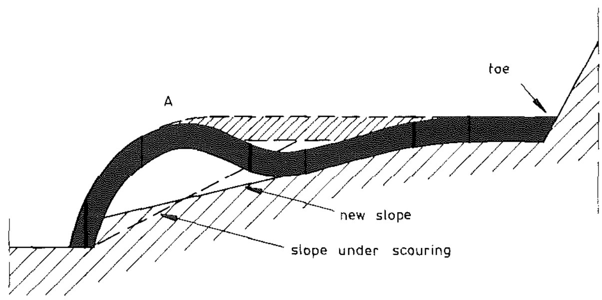


Figure 20.16 Bending of a mastic slab after the end has settled onto the scour hole.

Failure can also occur in a mastic slab lying on a sand bed which overlies a clay layer. In this situation wave pressures cannot propagate through the clay layer and will build up to cause large uplift pressures under the mastic slab.
Sand under the slab can be lost through cracks and as a result undermining can proceed. Because of the stiffness properties of the material cracks are more likely to develop in winter than in summer. In warmer periods it is possible for the cracks to flow together and re-seal.
The reduction of layer thickness due to viscous flow should not be overlooked.
The length of a mastic slab in front of a dike must be so large that the toe cannot be undermined and the slope stability is endangered.

20.7 Determination of the maximum slope

In order to prevent the revetment sliding off the dike body (Photo 9), the slope angle must be less than the angle of internal friction. For a relatively impermeable revetment, the slope angle in places where water is likely to occur behind the revetment, should be no larger than:

$$\text{tg}\,\alpha \leqslant \text{tg}\,\phi \cdot \left[1 - \frac{\varrho_w}{\varrho_n}\right] \qquad \text{(see Appendix III)}$$

in which:

α = slope angle (dgr)
Φ = angle of internal friction of the subsoil (dgr)
ϱ_w = density of water (kg/m^3)
ϱ_n = density of wet soil (kg/m^3)

For a slope of cohesionless material where groundwater can flow out freely, in order to prevent slope instability the angle should be (84):

— under water:

$$\text{tg } \phi > \frac{\text{tg } \alpha}{\left[1 - \left(\frac{\varrho_w}{\varrho_n - \varrho_w}\right) \cdot \frac{i}{\cos \alpha}\right]}$$

— above water:

$$\text{tg } \phi > \frac{\text{tg } \alpha}{\left[1 - \frac{\varrho_w}{\varrho_n} \cdot (1 + \text{tg}^2 \alpha)\right]}$$

in which:

i = potential gradient at the surface

The allowable slope is also determined by the internal stability of the mix itself, in both the construction and the completed phase. The method of construction can also be a determining factor in the choice of slope.

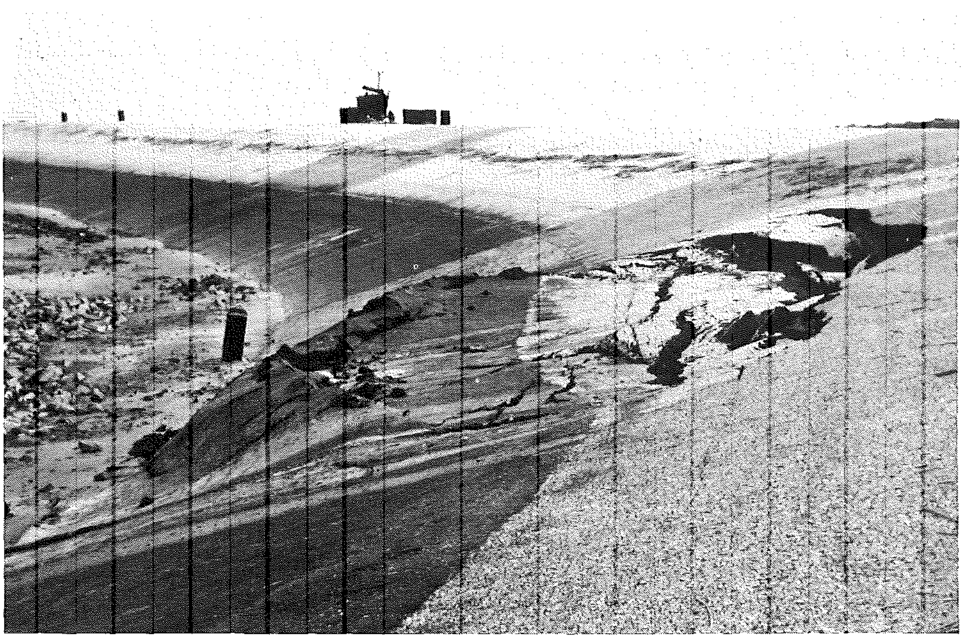


Photo 9 Failure of an asphaltic concrete revetment due to unstability of the sub-soil

ASPHALTIC CONCRETE

Depending on the mix composition asphaltic concrete is stable on a slope of about 1 : 1.7 even when hot (3, 22). After completion the mix is stable at the same angle. The construction method, particularly the applicability of the construction equipment, is generally the limiting factor.

MASTIC

Mastic is, preferably, used on a horizontal subsoil. For a lean mastic the maximum allowable slope is about 10° and for a fat mix 5°, depending on the duration and temperature regime.

GROUTING MORTARS, SURFACE-GROUTING

The method of applying the mastic must be such that in the whole surface the stone remains in place.

PATTERN-GROUTING

The grouted areas of stone must be stable under the prevailing conditions. The areas must satisfactorily lock up the interlying stones. It is, thus, important that the grouted areas remain as lumps. The mix composition and the method of execution must be such that the grout does not stay in the upper layers or sags completely through the layer.

FULLY GROUTING

Because of its internal skeleton this type of revetment has high stability, see also Chapter II.

DENSE STONE ASPHALT

Dense stone asphalt is preferably used on a flat subsoil. See also Chapter 12.

OPEN STONE ASPHALT

Open stone asphalt, when hot, is stable on slopes upto about 1 : 1.5, see also Section 13.3.1.
The stability of open stone asphalt mattresses should be such that they do not slide, as a whole, down the slope. The maximum allowable slope is about 1 : 2.5.
It is possible to anchor mattresses to the slope. In this case it is essential that they remain in contact with the subsoil.

LEAN SAND ASPHALT

Lean sand asphalt has because of its viscousity just as other asphalt mixes the tendency to creep down a slope

1. In constructions where deformations play an important role, such as foundations and abutments, creep has to be limited.

2. For constructions in which deformation is not important the only limitation is safety. The resistance to failure of granular materials is characterized by the angle of internal friction, ϕ, and the cohesion, c.
   The failure mechanism can be studied using, for example, a slip circle analysis or the Prandtl wegde method.

For present purposes the equilibrium method is used to calculate the stability of the material in which it is assumed that deformations remain sufficiently small if the relationship between the shear stresses which develop and the maximum allowable shear stress is equal or less than 0.75. For bituminous products the stability problem is more complicated because of the viscosity. A time-dependent component must be taken into account.
Investigations have been carried out on lean sand asphalt made from sand from the Eastern Scheldt and 3 to 4% bitumen 80/100. The object was (32):

1. To establish the minimum angle of internal friction, ϕ and thus the stress situation for which the deformation speeds are small. Under such shear stresses ϕ was in the range 20° to 24°.
2. To establish the maximum angle of internal friction, ϕ that is, the situation when deviator-stresses produce deformation speeds which do not become smaller but remain constant or increase. Under such shear stresses, ϕ was in the range 30° to 34°.
3. To establish relationships between the stress conditions and deformations and deformation speed between the maximum and minimum.

There are several ways in which the failure criteria can be established:

1. Bishop's slip circle analysis
   In this method the total shear strength which develops at the critical slip surface is determined. The Bishop method is the most traditional and readily applicable method, see Appendix VI.
2. A finite element method
   The applicability of what is referred to as the MARC calculation program for simulating creep in asphalt is studied at the Laboratorium voor Grondmechanica (Soil Mechanics Laboratory) at Delft.
3. The plate method
   This is a modification of the Bishop method, the underlying principle of which is that the sliding of adjacent plates of asphalt can be calculated with the existing method and that from this the relationship between deformation speed and shear stress can be found simply. In the method stiff plates can slide over each other by means of imaginary viscous interlayers. The summation of the separate deformations indicates the extent of the total deformation.
   Large deformations, developing in a short time indicate unallowable loads and thus failure (32).

ASPHALTIC MEMBRANES

Asphaltic membranes are generally covered. In situ prepared membranes are often covered with earth; prefabricated membranes with earth, rubble, concrete, asphalt, etc. The following failure mechanisms can develop:

1. The membrane causes the protection layer to slide off.
   To prevent this the friction component along the membrane should be larger than the weight component of the protection down the slope, that is:

   $f > \mathrm{tg}\, \alpha$ *(see Appendix VII)*

   in which:

   f = the friction component between the membrane and the subsoil or protection layer
   α = the slope angle of the membrane

2. The shear stress transfer over the membrane can cause such large deformations that it can result in changing geometry or damage of the protection.
   The deformations which develop as a result of shear forces acting on the membrane should not exceed certain limits, see Appendix VII.

PART D

EXECUTION

Summary

Part D deals with the execution techniques applied for asphalt revetments used in hydraulic engineering. The execution is divided into three phases, namely the preparation, the transport and application of the asphalt. In the first chapter some general aspects are considered and the most common types of asphalt plants in the Netherlands are simply described.
A more detailed description is then given of the three execution phases for the various and most frequently used asphalt types in hydraulic engineering. These are:

- asphaltic concrete;
- mastic;
- grouting mortars;
- dense stone asphalt;
- open stone asphalt;
- lean sand asphalt;
- membranes.

In order to make the various chapters easier to read and to avoid cross-references, the text has been repeated where necessary.

Joints form a very important part of the execution and often it appears that these are the weak points of an asphalt construction. Special attention has, therefore, been given to this part of the design. When considering joints, distinction can be made between joints in the same material in the form of daily joints and joints between other types of revetment materials, onto structures and onto toe constructions.

21 General aspects

To a large extent the quality of an asphalt revetment is determined by the execution. The specifications, given in V.U.C.M. (18), can be applied to the execution in so far as they are related to hydraulic structures.
The execution can be divided into three phases:

1. The production of the asphalt mix;
2. Transport from the production site to the construction site and onsite storage;
3. The placing of the mix.

21.1 Production

Preparation is generally in a normal asphalt mix-plant which produces a homogeneous mixture of bitumen and minerals.
The following types of mix-plants can be distinguished (5):

1. Batch plants;
2. Continuous mixing plants;
3. Semi-continuous mixing plants;
4. Special plants, for example, drummixers.

In the Netherlands, the first type is commonly used. A special type of batch plant is, what is referred to as, the tower mixing plant. This has a large number of hot bins in which a great variety of different mineral aggregates can be stored hot. This system gives very good flexibility for the production of different types of mix.
Continuous and semi-continuous mixing plants are especially suitable for producing large quantities of the same type of mix.
A batch plant is shown schematically in Figure 21.1. The following components can be distinguished:

- cold bins;
- dryer drum;
- dust collector;
- screening unit;
- weight hopper;
- pugmill;
- mineral base storage;
- bitumen storage;
- filler storage;
- hot mix storage.

The various cold aggregates are brought together in the cold bins in the required proportions, the quantity depending on the particular production. The cold elevator then transports the mineral to the dryer drum where it is dried and heated. It is then delivered by the hot elevator to the hot screens where it is fractionated into a number of specified sizes. These fractions are weighed out in a hopper just as the filler and the bitumen. The components are then mixed in the pugmill. After mixing the mixer is emptied into the lifting bin.
The mixing time depends on the type of installation and mix; the mixing temperature depends on the transport distance and the mix viscosity required. The temperature must not be above 190°C.
An important part of the mixing installation, partly due to environmental considerations, is the dust collector system. In this equipment the dust and fine sand which has been carried from the dryer drum by a stream of air is collected. There are two types of systems:

a. The dry dust collector. This collects the dry dust which is then stored in bins. If possible the dry dust is put back into the production process.
b. The wet dust collector. In this equipment the air containing the dust is passed through a water spray or bath and a mud is produced.

The cold aggregate must be stored separately at the mix-plant, preferably on a hard surface, so that contamination with other materials is avoided.
The stock of sand, gravel and crushed stone must be large enough to ensure uninterrupted production.

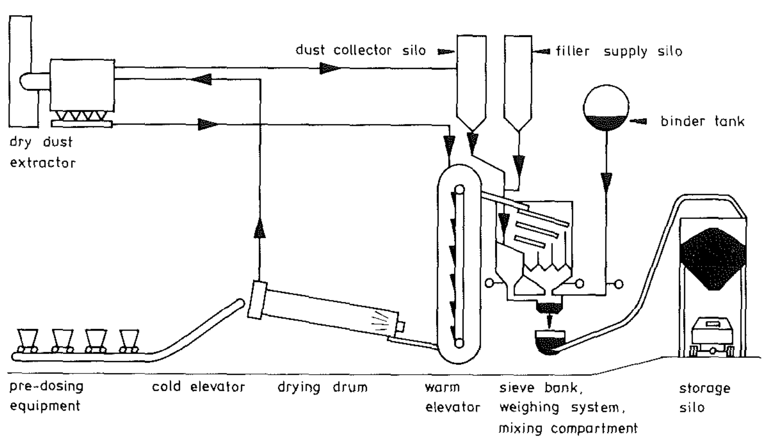


Figure 21.1 A batch mixer shown schematically.

The bitumen is stored in heated tanks at the mixing-plant site, the filler in separate silos. Asphalt for revetments is preferably applied on the day on which it is prepared. Long-term storage is only possible in well-heated, airtight silos which prevent excessive cooling and oxidation of the binder.

Specific installations, which are adapted to that particular work, are often developed for large-scale hydraulic engineering projects. An example of this is the modified drum mixer, see Figure 21.2, which is used for producing lean sand asphalt in bulk. In this installation the bitumen and mineral are not mixed in separate compartments but in the dryer drum itself. With this system it is easier to produce a continuous flow of material. Other examples of specific installations are those used on the asphalt ship, the 'Jan Heijmans', for producing large quantities of asphalt mastic and the dense stone asphalt installation at IJmuiden which was specially built for mixes using large stones.

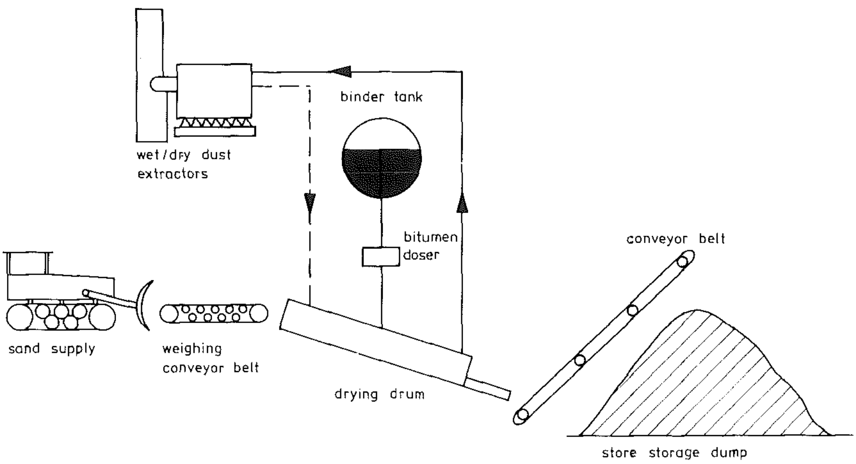


Figure 21.2 A modified drum mixer shown schematically.

21.2 Transport

Transport from the mixing plant to the construction site is, in general, by truck and/or, depending on the type of mix and the transport distance, in insulated containers, covered containers, open containers or stirring kettles. Cooling must be restricted as much as possible and the mix should be protected from the rain. The containers must not be contaminated and should not contain any hard pieces of asphalt.
In order to prevent the mix adhering to the inside of the container, only a solution of surface-active material in water should be used or a limited quantity of petroleum, uniformly sprayed on the surface of the container.

Transport forms a large part of the total placing costs and on large projects it may be profitable to locate the asphalt mix-plant close to the construction site. A balance must

be reached between, on the one hand, the advantage of a shorter transport distance for the asphalt mix and, on the other hand, the disadvantage of relocating the installation, possibly longer transport distances for the basic materials and obtaining the various permits required.
Reliable supplies to the site are essential for smooth operations. Temporary storage of the mix on site for a long duration and of large quantities, due, for example, to interruptions in operations, must be avoided. Lean sand asphalt which is to be used in bulk under water can be stored since the mix can only be used after it has cooled to a working temperature of less than 100°C.
In hydraulic engineering projects the transport distance on the site itself is often very large, for example, when constructing a dike. In such situations it is very tempting to transport the asphalt over the already completed section of revetment. Since, however, the material is most times unsuitably for this, it should be discouraged.
The method of transporting the mix to the site should be given very serious consideration during the design phase.

21.3 Application

Distinction is made between applying asphalt products in hydraulic works under water and above water, in the dry or in the wet. In general, greater control and accuracy can be achieved when laying above water. Some techniques can only be used above water whereas others are more suitable under water; extra care must be taken when placing in the tidal zone. In this respect asphalt should not be placed on a slope as long as the water-level in the dike body is causing a flow of water out of the slope. This would cause construction problems because of the deformation of the subsoil, damage to the asphalt and, possibly, uplift pressures.
Asphalt constructions are made using particular equipment depending on the mix type and the location. Certain mix types must be mechanically compacted. The mix design has to be taken into account in the choice of application method.
It is important that skilled site personnel are available. The planning of the work must be interrelated with the production, transport and application methods.

22 Asphaltic concrete

22.1 Production

22.1.1 *The mixing plant*

Production is carried out in a normal asphalt mix-plant. Batch plants are generally used in the Netherlands.

22.1.2 *Storage of building materials*

Separate storage facilities should be used for the crushed stone, gravel and sand, so that contamination with other materials cannot occur. Filler must be stored in silos, bitumen in heated tanks.

22.1.3 *Mixing time and temperature*

Mixing time depends on the type of plant.
The mixing temperature is determined by the mix viscosity required and the transport distance; it lies between 140 and 190°C.

22.1.4 *Hot mix storage*

Hot mix can be stored in insulated bunkers for a considerable time.

22.2 Transport

22.2.1 *Means of transport*

a. Trucks
If the transport distance and the air temperature allow it, the transport can be in open trucks covered with a tarpaulin sheet. Insulated trucks are recommended for longer transport distances.

b. Containers
For special situations requiring transport over water or in which excessive cooling is likely, insulated containers are used.

22.2.2 *Onsite storage*

In order to ensure that the placing temperature is maintained satisfactorily high separate insulated transport bunkers should be used, if long-term temporary storage is unavoidable.
Trucks, either open or with a tarpaulin sheet, should not be exposed to rain or wind for long.

22.3 Application

22.3.1 *Application techniques*

Asphaltic concrete must be placed in the dry. This means that, on sea dikes, it can only be used above mean high water. The sand bed must be smooth and satisfactorily compacted, see Photo 3.

PLACING
Asphaltic concrete can be placed using:

- Earth-moving equipment
  The sand bed must not be disturbed during placing of the asphalt. It is, therefore, recommended that earth-moving and other equipment are used on the slope as little as possible. Tracks should be smoothed out. Another method is to use portable screeds.
  If a crane is used then the boom length is important; one single placing-stroke is preferable to two.
  The mix should be shedded in not too large heaps, working up the slope; this will prevent uneven compaction, see Figure 22.1. The mix should then be spread in not too broad strips, 3 to 4 m wide, and accurately profiled.

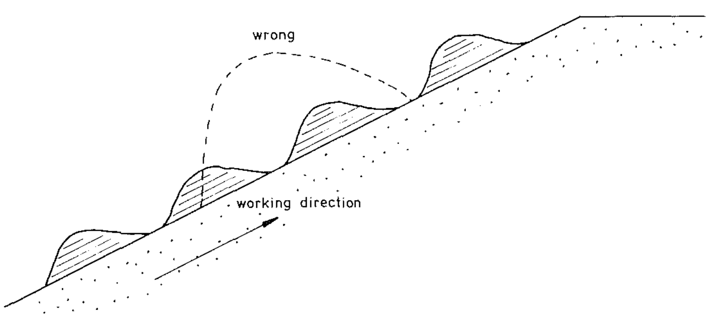


Figure 22.1 Asphaltic concrete brought onto the slope in small heaps, working upwards.

It is important, during execution, that the upper and lower edges of the asphalt layer are well-sealed with boards. The boards should not be removed too quickly or the edges may collapse. After removal, earth should be placed as quickly as possible against the edge of the asphalt.

The following working methods can be used:

a. A crane shedds, spreads and accurately profiles the mix on the working surface. Where necessary placing can be finished by hand.

Photo 10 Construction of an asphaltic concrete revetment: The crane dumps the asphalt on the slope; the backhoe spreads the asphalt

b. A dragline dumps the mix on the working surface where it can be spread by the dragline or, as is more usual, by a backhoe, a payloader or by hand. With the latter three methods the surface can also be accurately profiled, see Photo 10.
c. A pay loader shedds, spreads and accurately profiles the mix on the working surface. Care should be taken, especially with this method, to ensure that the sand bed is not churned up.

If a mix is difficult to work with, this can indicate that:

1. The bitumen content is too high, in which case the mix must be re-designed.
2. The mineral in the mix was not dry enough; this can occur when the mixing plant production is higher than the application capacity and the necessary balance between production and placing is lacking.

FINISHERS

Finishers can spread the layer to the required thickness in one or more passes, at the same time compacting it.
The following types of finishers are normally available:

a. Normal road construction finishers, used on gentle slopes upto 1 : 6 (Photo 11).

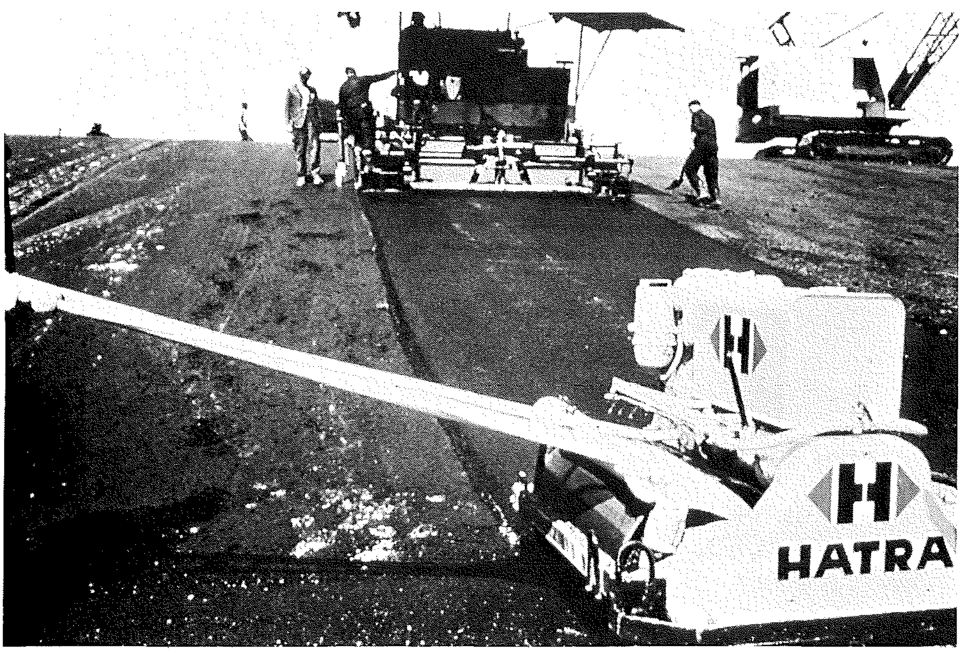


Photo 11 Construction of an asphaltic concrete revetment using a normal road finisher

b. Finishers adapted for use on slopes. The equilibrium down the slope is supplied by the finisher attached by cables to winches at the top of the slope. The supply of mix to the finisher is by special small trucks which are also attached by cables or by a bucket suspended from a crane, see Photo 12.

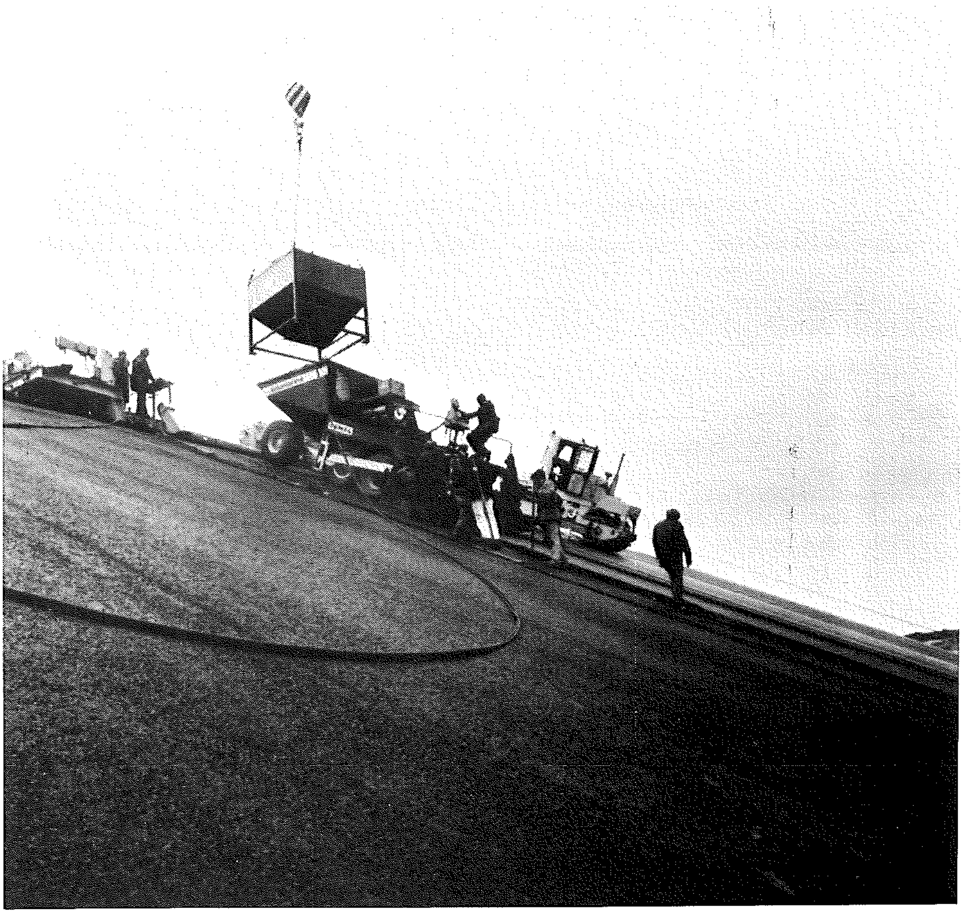


Photo 12 Construction of an asphaltic concrete revetment using a finisher adapted for use on a slope

c. Finishers specially designed for use on slopes (Photo 13). These machines include a framework which spans the whole slope. It is so arranged that the finisher can move along the slope. The asphalt is fed continuously on to the slope while the machine moves slowly in the working direction.

Type a and b finishers generally move from the top to the bottom of the slope, but for special applications they can also be moved along the slope. Longitudinal joints must be avoided.

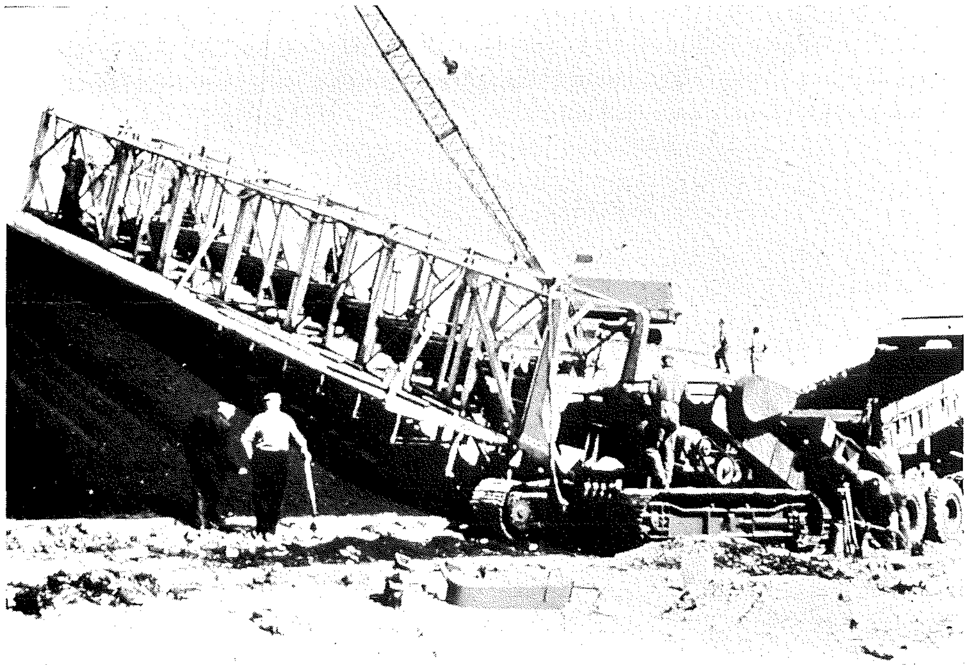


Photo 13 Placing of an asphaltic concrete revetment using a finisher specially constructed for use on a slope

COMPACTION

After placing, the asphaltic concrete must be compacted to obtain the required voids ratio. In order to achieve satisfactory compaction, rolling should begin as soon as the asphalt is sufficiently stable, thus at the highest possible mix temperature.
Recent investigations indicate that the best compaction results are obtained by direct rolling at the highest possible temperature using a small self-propelled tandem vibratory roller. This procedure is possible using the present day mix composition (6.5% bitumen).
If, in view of the time needed for compaction it is not possible with the high temperature to use this type of roller directly on the slope, then the material can be pre-compacted with a single or double non-vibratory roller. When using a finisher there will automatically be a certain amount of pre-compaction.
On slopes steeper than 1 : 4 provisions must be made to neutralize the weight of the roller down the slope. This can be achieved by attaching the roller by cables to winches at the top, see Photo 14.
Compaction should, preferably, be carried out in the direction normal to the axis of the dike, (that is up and down the slope). Important factors which influence the results of compaction are the type of roller, the weight of the roller, roller speed, the temperature variation in the asphalt, the layer thickness, the asphalt composition, the quality of

Photo 14 An asphaltic concrete revetment being compacted with a tandemvibratory roller

the sand bed, the weather and the method of laying. These factors determine the number and weight of the rollers together with the number of roller passes. The precise number of passes with a tandem vibratory roller and possibly a non-vibratory roller should be fixed at the start of the compaction operation and also when the mix composition has been changed. An indication of the number of passes (up and down) required is: 5 to 6 with a tandem vibratory roller or; 2 non vibratory roller passes plus 4 to 5 with a tandem vibratory roller. The heavier the roller, the greater the amount of compaction energy applied.

In practice slopes are often compacted further to obtain a smooth surface finish long after the required voids ratio has been reached. The asphalt is then too cool and cracks develop in the revetment. To prevent this happening, compaction should be stopped as soon as waves develop in front of the roller.

If, after a few passes, the asphalt begins to flow, particular at the bottom of the slope, this can indicate poor workability. Rolling should then be interrupted until the mix has cooled sufficiently. At the same time, the mix composition should be reconsidered and, if necessary, adjusted.

TACK COATS

When a two or more layer revetment is to be used there must be good cohesion between the layers.

To ensure this:
- the surface must be clean and free of sand, clay, etc.
- a tack coat of emulsion (type unstable) or a salt water-resistant asphalt tack substance should be applied before a hot layer is laid on a layer which has already cooled.
  The tack coat should be uniformly sprayed on the whole surface, preferably using spray equipment. It is important that the correct quantity (about 0,1 to 0.2 kg/m^2 of residual binder) is used; too much will cause slipping; too little will have no effect.

The tack coat must not be applied when it is raining. Applying the following layer has to be delayed until the emulsion has broken or that the solvent has evaporated because otherwise the asphalt can slide during compaction and possibly crack. In addition moisture can be sealed in, which can form steam and disrupt the adhesion.

22.3.2 *Application temperature*

Asphaltic concrete must be applied at a temperature not less than 130°C. The temperature at which rolling can begin, should be determined by testing on site. Indications are: compaction or pre-compaction with a roll or a tandem vibratory roller can begin at temperatures between 120 and 150°C; subsequent rolling can be carried out at temperatures between 90 and 100°C depending on the roller weight and the subsoil.
The application can be endangered in certain weather conditions. The most important adverse factors are rain and wind; air temperature is also of importance.
Asphalt concrete can still be placed under slight rainfall. Placing should be stopped if the site temperature falls below 0°C or if the temperature, in °C, is less than the Beaufort wind force (°C<Bf), the temperature and wind force being measured on site 1 m above ground level (16). The work should also be stopped, if the air temperature (°C) is a little higher than the wind force (Bf) and it is raining.

22.3.3 *Seal coat*

The revetment can be provided with a seal coat, see Section 9.1.6.

23 Mastic

23.1 Production

23.1.1 *The mixing plant*

Asphalt mastic can be produced in a normal asphalt batch plant. Mixing plants which are designed for specific applications, such as the asphalt ship, the 'Jan Heijmans', can have sizes which differ from the normal. Especially the requirements for storage capacity and dosing capacity of filler and bitumen can vary considerably and a adapted mixer should be considered.

23.1.2 *Storage of building materials*

Sand should be stored on a clean base to prevent contamination; filler is stored in bins and bitumen in pre-heated tanks.

23.1.3 *Mixing time and temperature*

The mixing time depends on the type of installation and the method of production. The mixing temperature is determined by the transport distance and the required application viscosity on site and lies between 130 and 190°C.

23.1.4 *Mix storage*

Since mastic should have a homogeneous composition and temperature it should be stored in a stirring-kettle. A container can be used if the storage period is less than one hour.

23.2 Transport

23.2.1 *Means of transport*

In principle asphalt mastic should be transported in a stirring-kettle. For short distances, for example, when the installation is on or very near to the site, an asphalt container can be used, depending on the quantity of mastic to be carried.

23.2.2 *Onsite storage*

Mastic should be stored on site in a stirring-kettle.

23.3 Application

23.3.1 *Application techniques*

Mastic can be applied using:

- a chute or pipe (constant discharge, see Appendix VIII);
- a crane bucket (application above water-level);
- a bottom-opening bucket suspended from a crane, or a dosing bucket running on tracks.

The maximum slope on which mastic can be applied is largely dependent on the mix composition, especially the proportion of bitumen. Maximum slopes for normal viscosity mixes are:

- under water, 1 : 7;
- above water, 1 : 10.

UNDER WATER

Sufficiently large batches should be used under water in view of the limited working accuracy which can be achieved and the adhesion which must be produced by heat transfer. The minimum layer thickness should be 8 cm, equivalent to about 160 kg of mastic per m^2. In order to lay a uniform layer thickness the subsoil should be smooth, especially at joints. It is important that tacking surfaces are sand-free. The mix should be hot enough to flow under water and form a slab of adequate thickness which will adhere to the slab layer already present to form a single revetment layer.
When applying with a pipe the mix should be able to flow out smoothly (Photo 15). The transfer-velocity of the pipe must be directly related to the discharge of mastic from the outlet nozzle. The latter in turn, dependent on the level of mastic in the pipe, the mix viscosity, and the size of the outlet nozzle (78), see also Appendix VIII. A possible method of placing with a pipe is given in Figure 23.1. (71). If the revetment thickness is a cm, then each 'roof-tile' layer is 1/3 a cm (minimum 8 cm) thick.

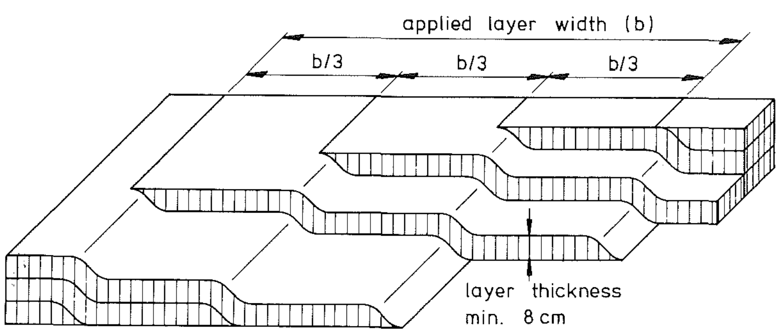


Figure 23.1 Application of a mastic slab under water using a pipe.

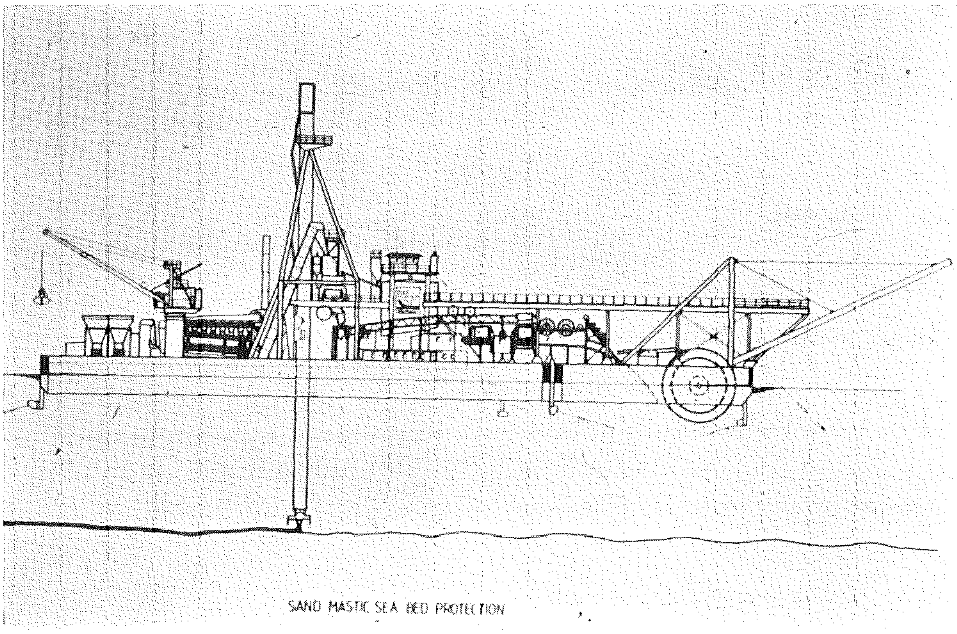


Photo 15 Applying asphaltic mastic under water on a large scale using the asphaltship 'Jan Heymans'

Mastic can also be laid using a special insulated bucket in the bottom of which are valves. The bucket is filled with mastic above water and then held about 1 m above the bed at the predetermined spot. The material flows out over the bed. The various locations, also referred to as 'plots', are previously fixed so that the mastic slabs overlap each other. The layer thickness required, the bucket content and the mastic viscosity have all to be taken into account when planning and executing this method. The flow distance and layer thickness produced by a given quantity of asphalt mastic of a certain composition, temperature and viscosity, under given conditions, must be known (78). These data can be determined by laboratory and in-situ tests.

ABOVE WATER

The application aspects discussed above for under water laying also apply to above water. The mix composition should, however, be adapted in view of the different temperature regime and application methods.

23.3.2 *Application temperature*

Above water the mix temperature should lie between 100 and 170°C, depending on the required viscosity. Below water the temperature should be lower then 150°C.
If the mix viscosity is incorrect it should never be adjusted by changing the temperature. Adjustment should be made by adapting the mix composition.

24 Grouting mortars

24.1 Production

24.1.1 *The mixing plant*

Grouting mortars are mixed in normal plants.

24.1.2 *Storage of building materials*

Sand, gravel and crushed stone are stored in such a way that they cannot be contaminated by other materials. Filler is stored in silos, bitumen in heated tanks.

24.1.3 *Mixing time and temperatures*

The mixing time is dependent on the type of installation and the production method. The temperature of the mix is determined by the transport distance and the required application viscosity and lies between 130 and 190°C.

24.1.4 *Mix storage*

Since the mix is prone to segregation it should be stored in a stirring-kettle. Mixes containing larger stones should, preferably, not be stored.

24.2 Transport

24.2.1 *Means of transport*

Mixes should, if possible, be transported in a stirring-kettle. In general this applied to mixes with upto 50% fine crushed stone. Mixe with a higher content or with coarser stones need to be carried in an asphalt container. The mix should be carefully remixed before application, for example, using the bucket of a crane.

24.2.2 *Onsite storage*

The material should, preferably, be applied directly. Stirring-kettles should be used for long-term storage. Coarse stone mixes should be stored in containers and carefully remixed immediately prior to application.

24.3 Application

24.3.1 *Application techniques*

Hot grouting mortars can be applied using chutes, the bucket of a crane or directly from a stirring-kettle or asphalt container, see Photo 16 and 17. The mortar is then spread further by hand with screeds or scrapers. Also the mortar container can be lifted clear above the slope with a crane, so that the mix can flow directly into the crushed stone, see Photo 18.
If a pure mastic is applied as a grouting mortar then the application can be as described in Section 23.2.1.
On relatively steep slopes which have to be grouted from the top a careful check is needed on the outflow rate and the quantity of grout used per unit area. If the flow is too high, small stones can be carried down the slope and grout can be lost if it flows over the toe of the revetment. Because of this it is better to work from the bottom of the slope upwards. On the other hand, if the grout flows too slowly and cools too quickly there is a risk that the toe is not reached and that voids remain in the revetment. Sometimes the grouting needs to be carried out in several layers. Experience and skill are necessary here. The method described above can also be applied if the toe lies just under water. Since the area to be grouted cannot be seen extra care is needed.

Photo 16 Grouting a crushed stone layer directly from an asphalt container truck

Photo 17 Grouting a crushed stone layer with the use of a crane

Other techniques are available for grouting under water, for example, dosing buckets, or, if a steady flow of grout is required, an insulated pipe with a special nozzle. The nozzle should be kept at a height of 0.5 to 1 m above the bed.

The quantity of grouting mortar needed depends on the thickness of the stone layer and the dimensions, form and grading of the crushed stone and the extent of grouting needed. For example: about 400 kg/m^2 of mastic are needed for fully-grouting a 50 cm thick crushed stone layer, and about 300 kg/m^2 for a 30 cm thick crushed stone layer, based on a stone voids ratio of about 40%.
A surface-grouting is obtained by filling at least one third of the crushed stone voids (37), only the top part of the layer being filled. In pattern grouting 50 to 80% of the voids is filled.
The quality and shape – no flat pieces – of the stones are important to achieve a good surface.
The stone to be grouted must be clean. In order to prevent fouling with sand and debris, ideally the stones should be grouted directly, in the same tide. Sometimes it is necessary to hose the revetment clean before grouting. Any hollows which occur during placing the stones should be filled in by hand.
The maximum slope which can be grouted with the more common grouting methods is

Photo 18 Grouting a crushed stone layer using a bucket

1 : 1.7 above water and 1 : 1.3 below water (71). If steeper slopes are required the application method has to be changed. Under water a slope of upto 1 : 2 can be produced.

24.3.2 *Application temperature*

The application temperature above water lies between 100 and 190°C, depending on the required viscosity and the method of application; below water the temperature must be smaller then 150°C.

If the mix temperature is too high it can damage an underlying fabric which then weakens, melts, wrinkles or ages rapidly. The maximum temperature, depending on the type of fabric, for example, a polypropylene cloth, is about 140°C.

25 Dense stone asphalt

25.1 Production

25.1.1 *The mixing plant*

Dense stone asphalt is usually mixed in a normal batch plant unless the stone size is too large. If heavy stones are to be used a special mixing plant is needed. For the construction of the IJmuiden harbour breakwaters, for example, a special plant was designed which included:

1. A normal mixing plant for the mastic;
2. An oven to dry and heat the larger stones (15 to 50 cm);
3. A rotating drum to dry and heat the smaller stones (5 to 25 cm);
4. A large rotating drum which produced a continuous mix from the above components.

25.1.2 *Storage of building materials*

Stone, gravel and sand should be stored separately to prevent contamination with other materials. Filler is stored in silos and bitumen in heated tanks.

25.1.3 *Mixing time and temperature*

The net mixing time, depending on the type of installation, lies between 25 and 30 sec. The mixing temperature, depending on the required viscosity and transport distance, is between 130 and 190°C.

25.2 Transport

25.2.1 *Means of transport*

Dense stone asphalt is carried in an asphalt container which is sometimes covered.

25.2.2 *Onsite storage*

Preferably the mix should be used directly. If temporary storage is necessary because of interruptions in placing operations, the mix should be kept in asphalt containers. Remixing, for example using the bucket of a crane, is then essential and should be carried out carefully.

25.3 Application

25.3.1 *Application techniques*

Dense stone asphalt can be placed:

- with buckets;
- with a crane
- directly from the means of transport; this method is used especially when laying a slab for toe protection.

The mix, depending on the composition and temperature, is workable on slopes up to 1 : 4. This should, however, be tested on site. For steeper slopes there are special measures such as adaptation of the mix and the use of mix packed into open-work bags, to be taken. See also Photo 19.

25.3.2 *Application temperature*

The application temperature, above water, is in the range 100 to 190°C; below water the maximum temperature is 130°C.

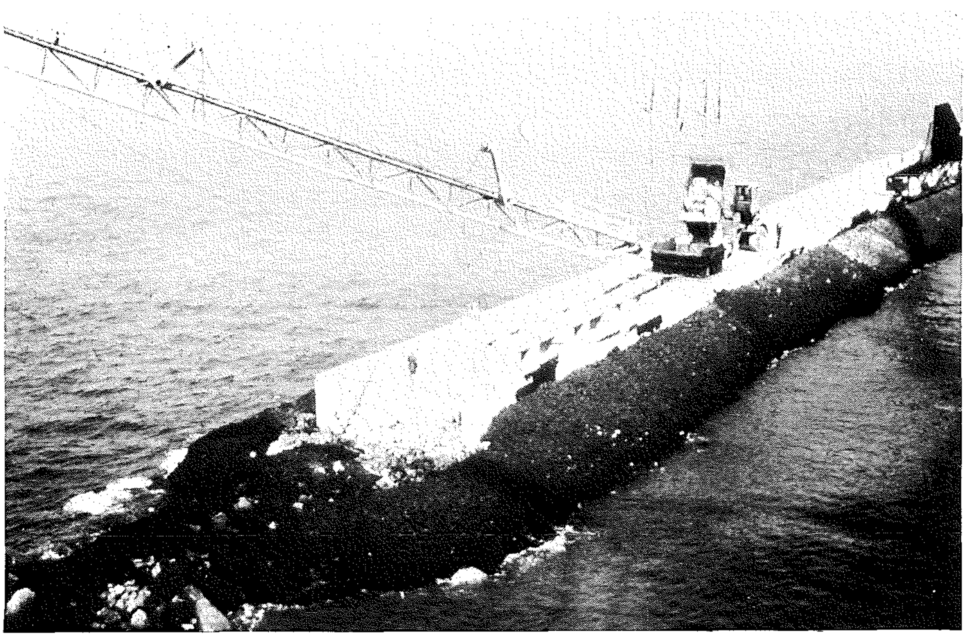


Photo 19 Construction of the IJmuiden breakwaters using dense stone asphalt

If a temperature higher than 130°C must be used under water in order to achieve the required flow, then boiling phenomena and steam develop which result in a mix with a porous surface.
When using a fabric, the maximum temperature of the mix will depend on the fabric properties. For a polypropylene cloth, for example, the maximum mix temperature is 130 to 140°C.

26 Open stone asphalt

26.1 Production

26.1.1 *The mixing plant*

Production is carried out in a normal batch mixer, in two phases:

Phase 1: Mastic mixing;
Phase 2: Mixing the mastic with the preheated, dried crushed stone.

26.1.2 *Storage of building materials*

Crushed stone and sand should be stored separately to prevent them being contaminated with other materials. Filler is stored in silos and bitumen in heated tanks.

26.1.3 *Mixing time and temperatures*

The mixing time for Phase 1 must be established by tests. An indication of the net mixing time, after dosing with bitumen, is 25 to 30 sec. The net mixing time for Phase 2 is about 30 sec.
The mixing temperature is between 140 and 150°C.

Open stone asphalt can be manufactured in two ways:

1. The mastic is prepared in advance and stored in a stirring-kettle, from which it is drawn off as required.
2. The mastic, required for a single batch, is prepared in the first phase and then mixed with the stone in the second phase. This process is repeated.

Generally the first method produces the most homogeneous mastic mix because the full capacity of the mixer is used. In practice, however, the second method is easier to apply. With optimal mixing techniques and a good state of maintenance of the mix-plant there is no noticable difference between the two methods.

26.1.4 *Mix storage*

Preferably the mix should not be stored at the mix-plant. Direct discharge into the means of transport is recommended because at higher temperatures segregation can occur.

If storage is unavoidable the mix should be placed in silos, and the amount should be limited to one truck-load.

26.2 Transport

26.2.1 *Means of transport*

- Open stone asphalt to be laid in-situ:
  Depending on the mixing and placing temperatures the material should be transported in sealed trucks or open trucks covered with a tarpaulin. In general long transport distances should be avoided since vibrations can cause segregation.
- Open stone asphalt mattresses:
  Mattresses can be carried flat up on a truck or a ship or rolled up on a cylinder which can float.
  The allowable radius of 12 cm thick mattress, provided that it is rolled up and unrolled slowly (5 m/min), is 4 m (38).

  If the mattress is not rolled up a special lifting frame has to be used.

26.2.2 *Onsite storage*

Open stone asphalt, to be applied in situ, should preferably be laid directly. If storage is unavoidable, due to an interruption in the work, the mix can be stored for a short time in

Photo 20 On site storage of open stone asphalt in a transport container

transport containers or sealed containers (Photo 20). The mix should then be re-mixed before use, for example, with the bucket of a crane. Careful remixing is essential to obtain the best results.

26.3 Application

26.3.1 *Application techniques*

Open stone asphalt should never be laid in situ under water.
Above water it can be laid using:

1. A crane. A crane with a slope profiling bucket can be used for re-mixing the material, which is then carried by the crane onto the slope. On the slope the material is profiled to the required thickness with the scoop. The minimum layer thickness depends on the stone dimensions, for example, with limestone 20/40 mm this should be 12 cm, see Photo 21.
2. A finisher. This method, with the exception of one project, has not yet been used.

The mix must be placed, in a single pass, at the required thickness. There should be no moulding carried out on the slope. Light pressure can be applied with the flat side of the bucket or possibly with a light roller. The material should not be finished with the sharp

Photo 21 Placing an open stone asphalt layer on a filter cloth using a backhoe

edge of the bucket. A filter must be placed under the open stone asphalt and the subsoil must be smooth. The maximum allowable slope of the revetment is 1 : 1.5.

OPEN STONE ASPHALT MATTRESSES

Open stone asphalt mattresses are made in formwork on a smooth foundation using a crane or a finisher. First of all cables or tapes are laid out, then the fabric is laid smoothly and flat and cut to the required length. The stone asphalt is then laid on top of the fabric. Reinforcing is built-in to take the tension forces which develop during transport, rolling/unrolling and sinking operations. The reinforcing has no further function once the mattress is in place, see Photo 22. After pre-stressing the fabric (about 6 kN/m^1) mattresses shorter than about 7 m can be picked up without additional reinforcing.

An important laying condition is that the mattress is laid smoothly on the subsoil and remains so. Efficient execution is essential for this and unless special care is taken this cannot be guaranteed. If the mattress is not well-laid in the tidal zone, it will begin to 'clap' causing too much movement which the material cannot follow.

The subsoil must be clean and smooth. The sinking procedures must be such that a normal unrolling of the mattress is achieved. Lifting and lowering must be done carefully and, thus, slowly. During hoisting and sinking the mattress should not be subjected to a too small a radii of curvature; for a 12 cm thick mattress the smallest radius allowable is 4 m.

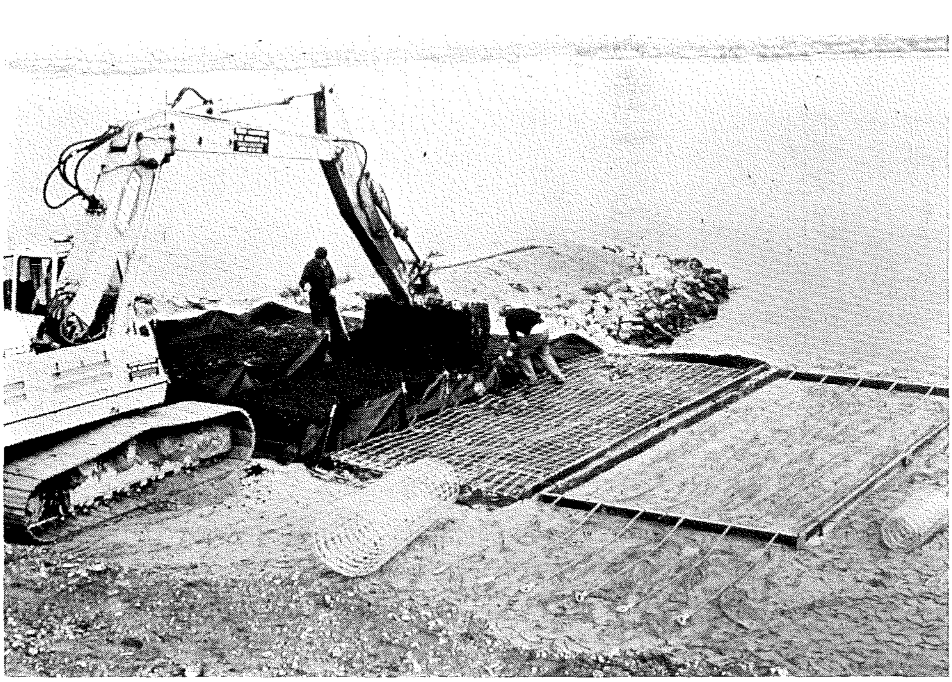


Photo 22 Fabrication of an open stone asphalt mattress. The various construction phases can be clearly seen

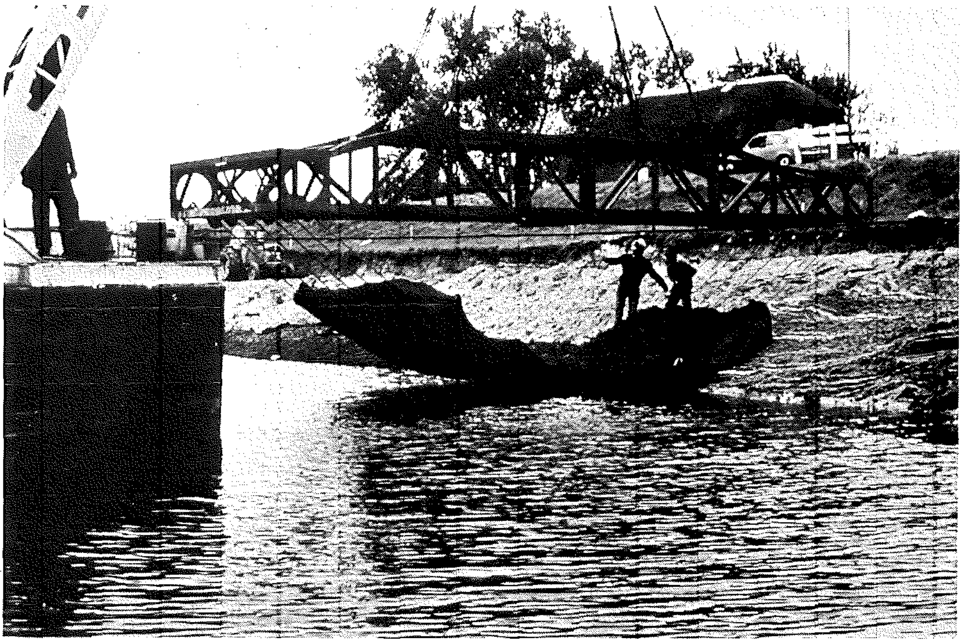


Photo 23 Placing an open stone asphalt mattress

The cables, in the mattress, are used for transport and placing. Large mattresses are hung directly, by the cables, from a front edge beam. With small mattresses, the cables, which have no reinforcing function, are placed under the mattress, which is then suspended, by a special frame hanging from a crane or derrick. Smooth and light cables should be used to facilitate their removal from under the mattress once it is in place, see Photo 23.
The maximum allowable slope is dependent on the subsoil and the method of anchoring. The mattresses can be used successfully on slopes of 1 : 1. In such cases great care is essential to ensure that the mattress stays permanently in contact with the subsoil.

26.3.2 *Application temperature*

In situ open stone asphalt should be laid at a mix temperature of between 110 and 160°C, depending on the sagging of the material.
When the material is laid on a fabric the maximum temperature depends on the type of fabric; for a polypropylene cloth the maximum allowable temperature is 140°C.

27 Lean sand asphalt

27.1 Production

27.1.1 *The mixing-plant*

Lean sand asphalt can be produced in a normal batch plant or in a drum mixer.

27.1.2 *Storage of building materials*

Sand should be stored on a clean foundation to prevent contamination. Bitumen is stored in heated tanks; filler, if required, in silos.

27.1.3 *Mixing time and temperatures*

The mixing temperature, with a batch mixer, is in the range 125 to 190°C; with a drum mixer 125 to 140°C.
The mixing time depends on the type of installation and the production method.

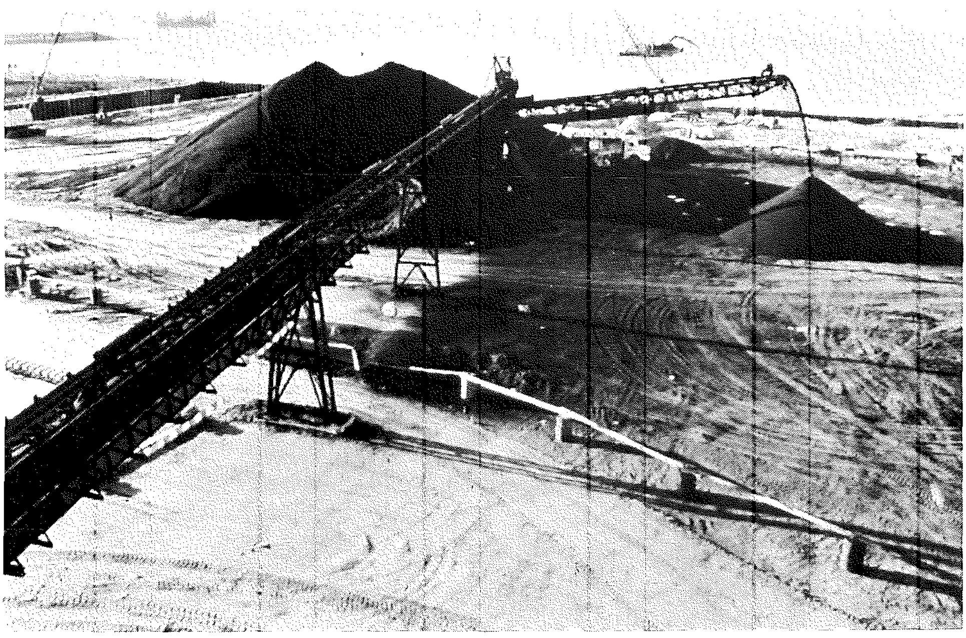


Photo 24 Long time storage of lean sand asphalt

27.1.4 *Mix storage*

When lean sand asphalt is to be used as a core material it can be stored in bulk (Photo 24). At large depots the cooled skin of the material can be put back in depot.
Lean sand asphalt for filter layers should be stored in storage bins, which can be isolated.

27.2 Transport

27.2.1 *Means of transport*

Lean sand asphalt for filter layers should be transported in sealed trucks or open trucks fitted with tarpaulins. Lean sand asphalt for core material can be carried in open trucks but when long distances are involved a tarpaulin cover should be used.

27.2.2 *Onsite storage*

Filter layer material should not be stored like bulk material but, preferably, placed directly out of the transporter.
Core material can be stored in bulk; because of the small heat losses the material at the centre of the pile remains workable for periods of 1 to 2 weeks.
Because of this the choice of working temperature and period can be chosen with flexibility in relation to weather conditions and water-level.
A greater loss of bitumen penetration than with other asphalt mix types can be expected (32).

27.3 Application

27.3.1 *Application techniques*

a. Lean sand asphalt used for filter layers or temporary slope revetments.
   The material can be placed with a finisher or a crane. The material must be laid to the correct thickness in one operation. Under water the material should be dumped in sufficiently large charges of about 1000 kg and with a thickness greater than that required theoretically, to allow for certain losses due to erosion.
   Moulding on the slope which can endanger the cohesion of the material as it cools down should not be carried out. Depending on the particular application, it is possible to compact the lean sand asphalt. The subsoil must be well compacted and correctly profiled.
   The minimum layer thickness, above water, is 15 cm and below wate 70 cm. With good subsoil the layer thickness, above water, can be 10 cm. Provided that the execution is carried out very carefully it is possible to lay a thickness of 50 cm under water.

b. Lean sand asphalt used as a core material.
   Lean sand asphalt, stored in bulk, can be placed in various ways:
   1. Direct shedding from the transporter followed by grading with a bulldozer or crane. See Photos 25 and 26. The finishing-off of thick layers can also be done with a bulldozer or crane.
      In view of temperature effects lean sand asphalt cannot be moulded. If a certain amount of compacting is required this can be done using the underside of the crane bucket, see Photos 25 and 26.
      In practice it has been found that the maximum depth in which lean sand asphalt can be dumped is 3 m. When dumped in greater depths the spread is excessive which accelerates the cooling process.
   2. Pouring through a pipe.
      Lean sand asphalt can be placed at greater depths using a tapered pipe. During the dumping operation the asphalt level in the pipe must always be above the water-level (Photo 27).
      In practice, the material has been placed in this way, in depths upto 23 m.
      The end of the pipe should be kept at a fixed distance above the bed and then moved horizontally as the lean sand asphalt flows out. Sometimes the flow of material is improved by shaking the pipe.
      The advantages of this method are:
      – the material is protected until it reaches the bed;
      – the pipe nozzle can be positioned accurately.
   3. Shedding from a bucket. Lean sand asphalt can be placed at greater depths by using large loads (2000 kg).
   4. Shedding from split-barges. This method is convenient for large individual charges of 200000 kg or more. In this method the sand asphalt should be packed in wire nettings.
   5. Shedding from a cableway. This is a good method for dumping large quantities. In this method, also, the material should be packed in wire-nettings.
   6. Shedding from a conveyor belt. This method has been used in water depths upto 10 m. Care should be taken to ensure that the application method does, in fact, produce a slope which is stable while still hot, under waves and currents.

Generally, reclamation bunds, made from lean sand asphalt, have only a temporary function. In view of the possible loss of material due to erosion extra material should be used. Immediately before applying the final revetment the bunds should be accurately profiled. Lean sand asphalt, minimum thickness 15 cm above water and 70 cm below water, can be used for this.
Sharp corners on the slopes should be avoided. These provide attacking points for waves and currents at which pieces of material can be broken off. In the tidal zone, for the same reason, construction joints should be avoided wherever possible.

Photos 25, 26 Construction of a lean sand asphalt reclamation bund

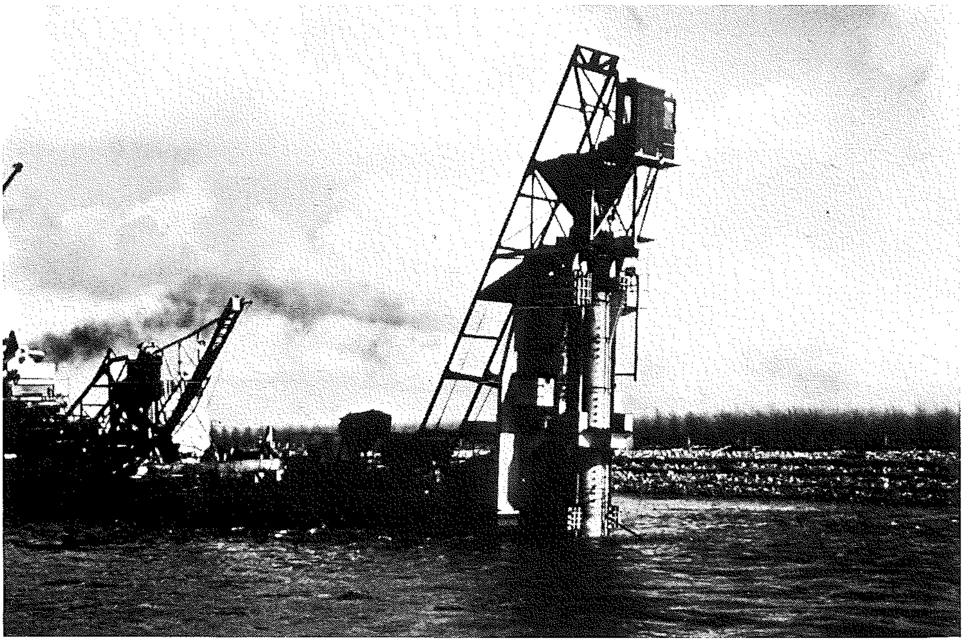


Photo 27 Applying lean sand asphalt under water using a pipe

27.3.2 *Application temperature*

Filter layers: 90 to 190°C
Core material: under water 80 to 110°C
above water 80 to 190°C.

Under water lower temperatures are necessary because of the danger that the bitumen skin is stripped from the sand grains. The material should also be laid at lower temperatures if it is to be driven over by trucks and bulldozers.

28 Membranes

28.1 Production

For prefabricated membranes reference should be made to the manufacturers specifications.

28.2 Transport

For prefabricated membranes: see the manufacturers specifications.
Bitumen, to be used for making membranes in situ, should be transported in a special tank.

Photo 28 Placing a prefabricated membrane (Hypofors)

28.3 Placing

a. Membranes carried out in-situ.

These membranes are carried out using a spraying-truck fitted with an extension piece which sprays bitumen in two or three layers on to the subsoil, until the required thickness of 8 to 12 mm is produced. The quantity needed is 6 to 8 kg/m^2 depending on the subsoil texture. The application temperature is a maximum of 200°C. The maximum slope is 1 : 2.5.

The subsoil must be compact and smooth and free of stones or sharp objects. It should be treated with a planticide or, alternatively, a plantproof sheet should be provided. The use of certain planticides can be detrimental to the environment. The bitumen can also be sprayed onto a pre-laid reinforcing fabric.

The membrane should be covered with at least 30 cm of earth or gravel, or 50 cm if plant-growth is expected (35).

b. Prefabricated membranes.

These membranes are, usually, slowly unrolled from a special frame attached to the arm of a backhoe (36), see Photo 28.

Unless the bitumen is specially adapted, these membranes should not be laid in low air temperatures (below 8°C). In such weather conditions the membranes should be stored in heated sheds.

The subsoil must be smooth and free of sharp objects. If necessary the membrane can be protected with a layer of earth, gravel, asphalt or concrete. When applying asphalt as a protection an intermediate layer of, for example, a fabric can be used to minimise the heat and mechanical effects on the membrane itself.

29 Joints in bituminous revetments

29.1 Introduction

An important aspect of dike revetment construction, which requires special care, are the joints; joints onto the same material, onto other revetment materials and onto structures in or on the dike body. A lot of thought should go into the design and execution of joints, especially since experience shows that damage often occurs at these locations. A principal requirement is that joints should be permanently sandtight. In a watertight revetment the joints should, obviously, also be watertight. Proper execution is essential in order to obtain satisfactory joints. Generally the length of joints should be kept as short as possible.

29.2 Joints onto the same material

When the work on one day is stopped and continued on the following day a transverse joint is required which is referred to as a daily-joint. The length of the joint should be kept as short as possible by executing it in a straight line and, preferably with no angles, see Figure 29.1.

To prevent cracks forming in the joints the adhesion between the two different surfaces must be as good as possible. To obtain this the joint must be clean and in the case of underfilled or precisely filled mixes, coated with an emulsion or pure bitumen, or pre-heated with infra-red heaters. It is recommended that the joints are not located at places where there is a change in the alignment or profile.

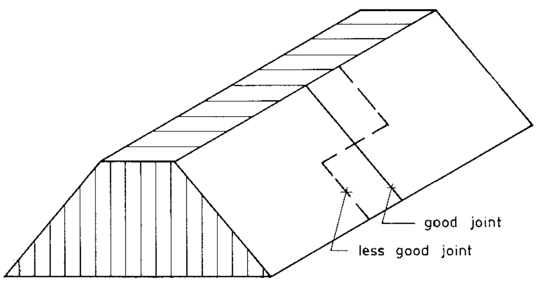


Figure 29.1 A transverse joint in the length direction.

ASPHALTIC CONCRETE

A well-designed joint between two surfaces is shown in Figure 29.2. Before the following layer can be laid the joint must be cleaned and coated with a Type Unstable asphalt

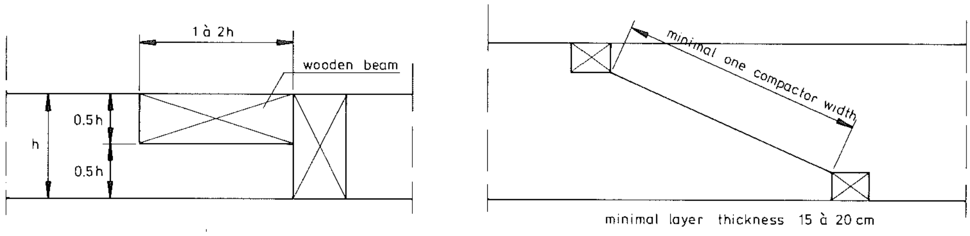


Figure 29.2 Transverse joints in asphalt concrete.

bitumen emulsion or it should be heated with infra-red rays to 100 to 160°C.
Heat should be applied very carefully since if a spot is heated too long the asphalt will burn. The joint should preferably have a rough surface. When executing the revetment in more than one layer the seams must not lie directly above one another.

MASTIC

Joints, both above and below water, must be clean and sandfree. A smooth subsoil is desirable. Adhesion between the various surfaces is obtained by melting them together.

GROUTING MORTARS

Joints must be clean and the transition area must be fully grouted.

DENSE STONE ASPHALT

Transverse joints, above and below water, must be clean. The joint need not be trimmed vertically, see Figure 29.3.

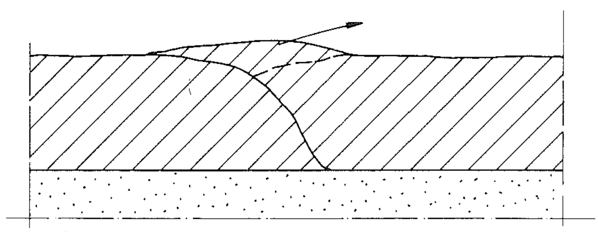


Figure 29.3 Daily-joint in dense stone asphalt.

OPEN STONE ASPHALT

a. In situ: Daily-joints must be clean and heated, possibly by the material to be used, or treated with bitumen or an emulsion. Daily-joints should be trimmed vertically with wooden beams or something similar, see Figure 29.4.
   Filter cloths should be spread smoothly and linked with an overlap onto adjacent filters. The cloth should not be pinned into place with, for example, reinforcing bars in such a way that it is damaged.

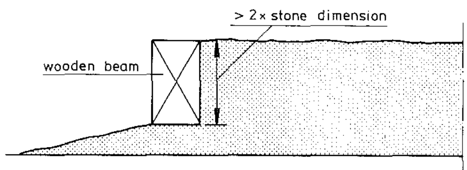


Figure 29.4 A transverse joint in open stone asphalt.

b. Mattresses: With open stone asphalt mattresses the underlying fabric should always overlap the fabric of adjacent mattresses. For 12 cm thick mattresses the overlap should be a minimum of 50 cm. Any cracks which develop under water can be repaired by dumping stone or another material onto them.

LEAN SAND ASPHALT

a. Filter layers/revetment material: joints should be clean and if necessary, heated.
b. Core material: joints must be clean. (In Zeebrugge, because of the heavy wave forces, a number of joints have also been coated with an emulsion).

MEMBRANES

Prefabricated membranes must be attached to each other with a liberal amount of blown bitumen. The method used for Hypofors membranes is shown in Figure 29.5. With membranes prepared in situ the bitumen should be sprayed with considerably overlap onto the clean, underlying membrane.

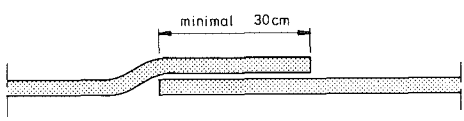


Figure 29.5 A joint used for Hypofors prefabricated membranes.

29.3 Joints between different types of revetment materials

When jointing to a different type of material the difference in stiffness of the materials has to be taken into account to avoid the development of cracks and fissures. Generally a filter is not required in the transition between two dense revetments. A filter should, however, be used under the transition from a solid to an open revetment.
Special provisions must be made when jointing onto another type of material, such as basalt, where the edge is irregular and not straight, or at places where compaction is difficult. A mastic gutter is a good solution here.

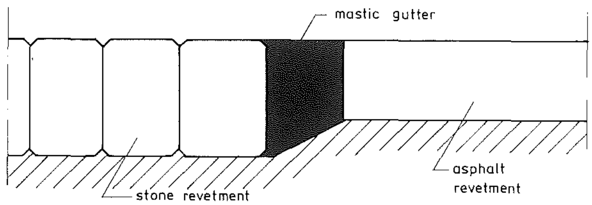


Figure 29.6 A joint onto another type of revetment material, using a mastic gutter.

The joint location is often made heavier for the following reasons:
- to smooth out the junction between two different thicknesses;
- to make an overlap sandtight;
- to limit the damage if the underlying layer yields;
- to take up any differential settlement and deformation between the two surfaces.

Disadvantages are:
- execution can be complicated;
- compaction may be difficult;
- groundwater may be trapped locally leading to the development of water pressures.

Joints between different types of asphalt must be clean and heated or coated with bitumen or an emulsion.
Asphaltic concrete can be jointed onto a clay revetment as shown in Figure 29.7. If no special provisions are made then the clay layer should be no thinner than 60 cm, to prevent it drying out. When mastic is linked to another material the bed should be smooth. For grouted stone a fully grouted strip should always be provided along the joint. A mastic gutter should be constructed along joints between open stone asphalt and other materials. Bitumen can be used for small gutters. When joining onto open stone asphalt mattresses, the mattress should first be placed and then the other revetment material laid against it. A membrane should extend under the adjacent material.
If sand asphalt (i.e. as core in a bund) is used to make a connection-joint with a quarry run or a rubble mound core, differences in grading of the several materials should be

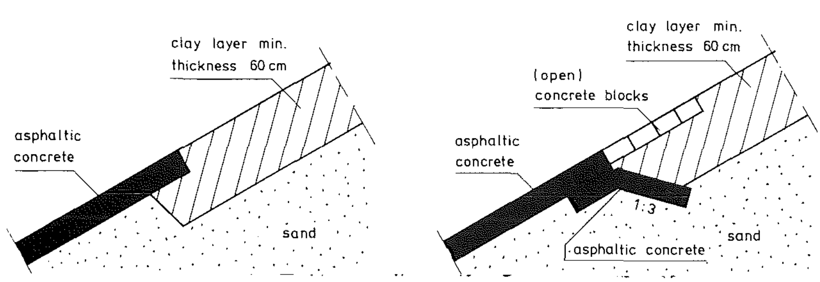


Figure 29.7 Examples of joints between asphaltic concrete and day.

kept in mind. To reduce penetration and erosian causing flow velocities, coarse gravel (30 mm and up) should be added to the sand asphalt (up to 40%) at the transition zone.

29.4 Revetment joints onto structures

The shape of the structure should be such that joints can be made easily. Joints should not be vulnerable to attack from waves and currents; also they should be flexible.
Usually joints are provided with flexible elements such as metal strips, fabric or membranes so that they can adjust to differences in settlement. These elements should have good adhesion to or be firmly attached to the structure and the revetment for example by a strip with an adhesive or by preset bolts. The revetment material is then placed against the structure and compacted, see Figure 29.8. Other solutions are possible which involve recesses or bitumen strips. Mastic slabs have to be jointed on a flat surface.

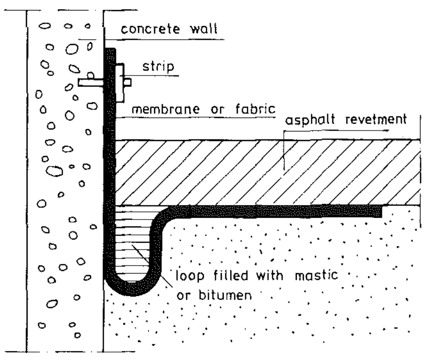
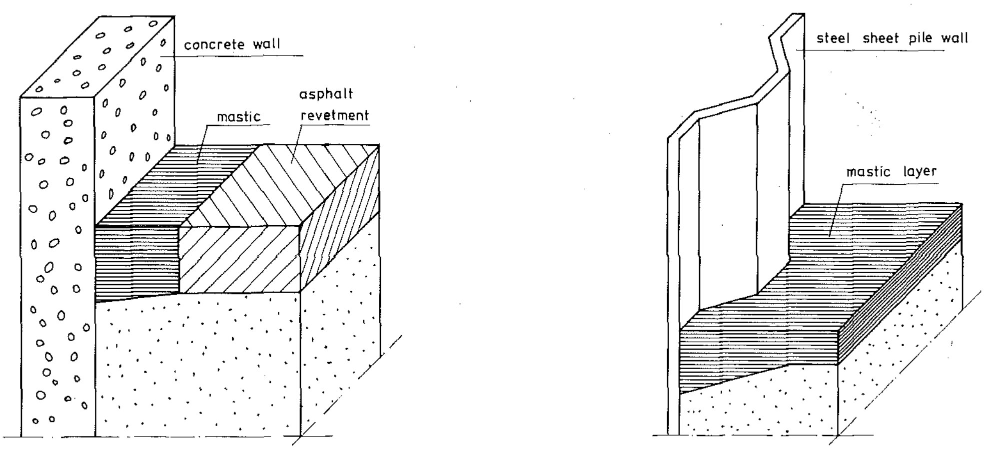


Figure 29.8 An example of a joint.



a) Jointing an asphalt revetment on to a structure using a mastic gutter.

b) Linking a mastic layer on to a steel sheet pile wall.

Figure 29.9 Examples of joints.

Other possible jointing solutions are given in Figure 29.9 in which the flexible joint is formed with mastic. The connection between a pre-fabricated membrane and a stiff structure should also be flexible, see Figure 29.10 (36).

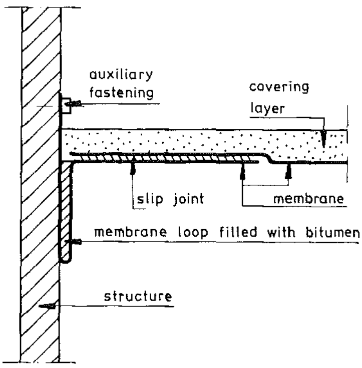


Figure 29.10 Sealing a membrane on to a structure.

29.5 Joints onto toe structures

The toe of a dike forms the transition between the dike revetment and the foreshore which is often provided with a bottom protection. In the Netherlands the toe of a sea dike is generally located between mean high and mean low water, unless the foreshore lies above mean high water (90). For lake dikes, the toe is generally a little above the normal water level. If the foreshore lies under water the toe can be constructed with a berm.
The functions of a toe construction are to:

- protect the revetment from scouring;
- support the revetment.

Over the years many different types of toe constructions have been applied. It is not necessary to consider them all here (90). Every situation requires a different solution; the choice lies with the designer.
Generally two types can be distinguished, see Figure 29.11:

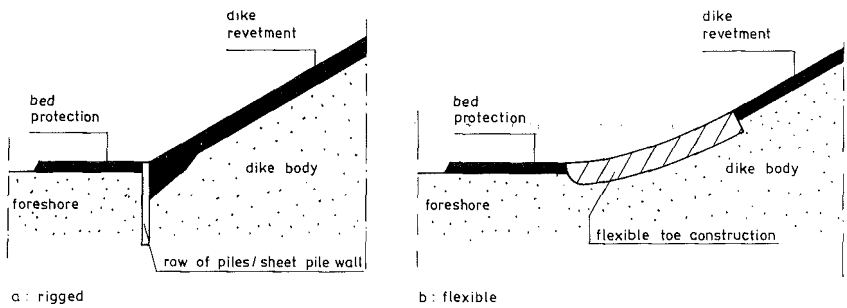


Figure 29.11 The two basic toe constructions shown schematically.

– a rigid toe construction which includes a row of poles or a sheet pile wall;
– a flexible toe construction in which the revetment runs into the foreshore protection.

The advantages of a well-executed rigid toe construction are:

– it gives good support to the revetment;
– it prevents material being washed out from under the revetment, provided the construction extends to sufficient depth;
– during execution its location is clearly marked and it provides a good basis for laying the revetment.

The intention is that the dike revetment and the foreshore protection function as a single unit. A rigid toe construction, however, forms a discontinuity in an otherwise flexible asphalt construction. Differential settlements can, therefore, develop.
With an impermeable revetment a sheet pile toe can hinder the flow of groundwater out of the dike body and result in extra uplift pressures. A drainage system can help to prevent this but then it should function permanently (17, 90).

– Since joints between the parts of and on to a flexible toe construction involve transitions between different types of materials reference should be made to Section 29.3 which deals with this aspect;
– Because of the discontinuity, permanent joints are difficult to obtain with a rigid toe construction.
  A sandtight joint can be made if the revetment material is firmly fixed against the toe beams and if a filter is used which extends sufficiently far under the revetment and along the toe, see Figure 29.12a. Another method, which can be used if the revetment material is not easily laid against the toe, or if the structure must be watertight, is to use an asphalt mastic gutter, see Figure 29.12b.

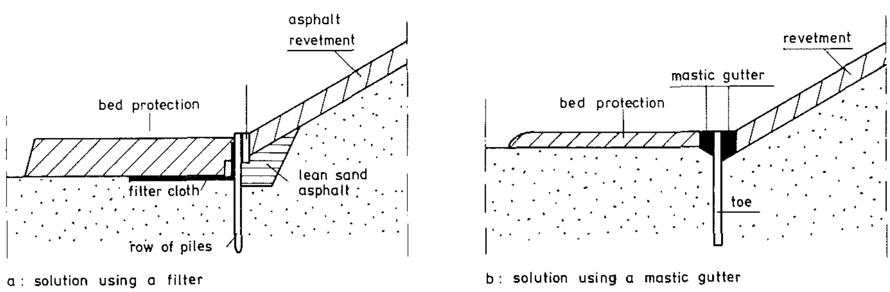


Figure 29.12 Possible joints on to a rigid toe construction.

PART E

MANAGEMENT AND MAINTENANCE

Summary

Part E deals with the management and maintenance of asphalt constructions used in hydraulic engineering. Management and maintenance are essential if these constructions are to function properly.
One aspect of maintenance is to repair damage. Damage can be caused by the forces acting on the revetment and, also, can result from poor design and execution. A review of the types of failure which can occur is given.
The way in which damage can be noted is treated generally. Possible ways of preventing damage are also discussed.
The types of damage and methods for detection are considered for the most frequently used types of asphalt mixtures in hydraulic engineering.
Repair methods for the various types of damage are then discussed, also per asphalt type. In the discussion repair work above water level is distinguished from that below.

30 Introduction

By management is meant the care needed to ensure that a revetment continues to function satisfactorily. Good maintenance is needed to achieve this.
A review of the possible forces and the damage which can be caused is given below:

- Wave impacts
  Wave impacts occur a little below still water level. During a storm this level is often much higher than normal and so the largest impacts tend to be predominantly on the highest parts of the slope.
  The smaller waves tend to occur lower down the slope. Although these cause relatively smaller impacts they can, because of their higher frequency of occurrence, lead to material fatigue and, therefore, to damage. Wave forces on the underwater part of a revetment are generally not so large.
- Lifting due to wave action
  An underwater revetment, for example, a bed protection can be lifted by standing or translatory waves. The chance of this happening, however, is very small.
- Water currents
  Currents, together with transported material, can erode an asphalt revetment. Objects which are carried along can exert impact forces on the revetment. If the current gets purchase on the edge of a thin asphalt slab or mattress, it can cause it to flap.
- Hydraulic uplift
  When the water level at the dike body is lower than that of the groundwater in it, pressures can develop under an impermeable revetment which tend to lift the revetment. If the design criteria, see Section 20.1, are frequently exceeded, the result can be that the revetment lifts locally and may eventually fail.
- Settlement
  The revetment material will try to adjust to subsoil settlement. If such settlements occur quickly or if the revetment is loaded, for example, by a wave impact, before it has adjusted completely, this can lead to damage, see Photos 5, 6, 7.
- Scouring
  Generally an asphalt revetment cannot adjust to scouring, although overfilled mixes such as mastic can have considerably deformation without cracking, see Photo 8.
- Instability
  Loss of bearing capacity and/or stability of the slope, the subsoil, or the toe can cause a part of the revetment to slide.
  An overfilled mix such as mastic will try to flow down a slope, naturally. This can lead to problems on kinks and joints.

- Biological attack
  Plants can exert such forces that they can grow through the revetment, see Photo 2.
  An open revetment can become sealed by plant growth and/or mud and as a result local hydraulic uplift pressures can develop. Animals can burrow under the slope, so that the revetment is no longer supported adequately. Animal droppings, for example, cow dung, can while drying out, attack the surface. Sea organisms attack the surface and give it a rough finish, see Photo 1.
  A revetment which is permanently under water will be attacked less than one which is above the highwater line or gets periodically dry.
- Traffic
  Traffic on the revetment can produce fatigue.
  Parked cars can deform the revetment and any form of long-term loading, for example, pipelines laid over a dike can cause permanent deformation. A manager should take these effects into account and institute regulations.
  Oil and petrol attack asphalt.
- Vandalism and accidental damage resulting from recreational activities.
  Damage to the revetment can be caused, for example, by small fires, fishing rods, and the like.
- Weather, including ice, wind and rain can attack the surface and lead to changes in asphalt properties (ageing, 'stripping').
- Collisions
  An impact from a moving vessel can cause substantial damage to a revetment, not only to the surface but also to the filter. The impact can also soften the subsoil and produce a loss in bearing capacity.
- Anchors
  Anchors can cause damage by impact, dragging and snagging on the revetment.
- Damage caused by other factors including moving ice, floating debris and chemicals.

Inadequate design and/or execution can, in the long term, lead to damage. The following aspects are important in this respect:

- Initial cracking of asphaltic concrete due to inefficient compaction.
- Ineffective transverse joints and poor jointing between placed layers, mattresses and transition constructions through which the subsoil can be washed away, see Photo 29.
- Rough and open surfaces as a result of which the construction is less resistant to wave and current attack.
- Inadequate layer thickness. With grouted slopes and revetment of stone asphalt, the loss of a single stone can result in a considerable loss in layer thickness.
- Inadequately grouted stone.
- The sealing of an open stone asphalt or partly grouted stone revetment leading to the build-up of hydraulic uplift pressures.
- Poor adhesion between the sand and bitumen in a lean sand asphalt mix.
- Holes which develop in membranes when the subsoil is poorly prepared and the

protection layer carelessly laid. With in-situ membranes the adhesion between the various layers, and with pre-fabricated membranes the adhesion between adjacent panels, may not be sufficient.

Photo 29 Because of a poorly constructed daily joint the subsoil material has been washed out causing the revetment material to deform

In order to discover damaged areas and to prevent further damage it is necessary to carry out proper inspection checks. These must be effective and should, therefore, be carried out regularly and systematically so that the chance of damage being detected is adequately high.
Depending on the type of damage the inspection can be made visually or with special apparatus.
After damage has been detected it should be repaired immediately if the dike or the revetment is directly endangered. If repairs are not so urgent they may be postponed until a favourable time of the year.
Also if the damage is not so serious, it can be left until it develops further, at which stage the unit costs of repairs may be relatively cheaper. If the circumstances are such that an essential repair cannot be made completely right away then the damage should be repaired temporarily.

Different detection and repair methods are employed for revetment damage above and below water.

Damage above water is generally easier to detect and repair. Underwater damage is, often, only noticed because of associated phenomena such as:

- Inexplicable lowering of the water-level in a canal or reservoir.
- The seepage of water on the rear side of a dike.
- Failure of a dike or revetment due to the loss of core material.
- A potential damage causing-situation such as the grounding of a ship on the slope.
- The emergence of material which has deviously originated from within the construction.
- Settlement of the revetment above low water.

The underwater part of a construction should be probed periodically. It is important to carry out detailed inspections, occasionally, with a diver.

31 Damage prevention

An optimum design and good execution are the best methods to prevent damage. Possible ways by which the manager can, substantially, prevent serious damage or can limit the chance of it occurring are:

1. Early inspection of any deformation of the revetment (resulting, for example, from settlement or uplift water pressure) to find out if it will lead to cracking. Visual inspection can be made above water-level. It may be possible, in the future to carry out these inspections with special equipment. Underwater inspections should be made using probes or by divers.
2. The foreshore and the toe of a dike should be inspected, for example by soundings, so scouring can be tracked and the necessary measures taken in time.
3. The timely detection of cracks and small areas of damage (to membranes and lean sand asphalt filter layers also in the protection layers) and, if necessary, their repair. The presence of pieces of underwater revetment on the slope can indicate damage. This can be detected by regularly dragging the slope.
4. Rubbish, such as pieces of wood and also oil, should be removed promptly.
5. Animals such as musk rats which cause damage should be trapped.
6. The minimum possible amount of traffic should be allowed on the revetment.
7. Provisions should be made for recreation, including car parks. Regulations should be enforced to prevent vandalism.
8. Precautions should be taken to limit ship collision as much as possible.
9. Anchoring should not be allowed in vulnerable areas.
10. The water level in reservoirs and canals should be recorded regularly and any unaccountable changes should be investigated promptly.
11. Excessive plant growth should be removed.
12. Surface wear should be detected and repaired promptly; asphaltic concrete should be regularly given a seal coat.
13. The execution of repairs should be carefully supervised. Obviously all construction work should also be carefully supervised. A dike manager should walk along the dike on the bottom end of the slope at low water, certainly after every storm, but preferably at regular intervals, for example, every one or two weeks.
14. The design boundary conditions should be observed. If any changes are proposed the consequences must be considered, for example, the decision to increase the number of push-tow units on the Rhine from 4 to 6 barges.

It is recommended that the manager keeps a maintenance inventory and periodically tests the revetment materials, in situ or with core borings, and compares the mix properties with the design assumptions.
Visual inspection of the revetment can be based, perhaps, on the system used in road engineering.

32 Types of damage

32.1 Asphaltic concrete

The most likely damage which can occur to asphaltic concrete and the causes are given in Table 32.1.
One type of damage which occurs frequently is cracking. Cracks in the revetment should be detected promptly since they can lead to extensive damage. Damage above water can often be detected visually. Cracks and open joints can only be located when they are fairly large and on the surface.

The extent of a crack, whether or not it goes through the revetment layer, can be checked by core borings; sometimes water seepage can indicate that the crack extends through the layer. It may be possible, in the future, to track small cracks internally using special measuring equipment.

Table 32.1 Types of damage to asphaltic concrete revetments.

Type of damage	wave impacts	lifting by wave action or currents	currents	hydraulic uplift pressure	settlement	scouring	instability	plants	animals	sea organisms	traffic	vandalism/recreation	collisions	anchors
Cracks	x	x		x	x	x	x	x			x			
Pieces of revetment broken out holes			x					x	x			x		x
Surface damage			x					x	x	x	x		x	
Deformation				x							x			x
Grooves in and through the revetment													x	x
Pieces broken from the edge of the slope						x								
Sliding off of part of the revetment							x				x		x	
Opening up of daily joints and joints						x						x		

Surface wear is shown by the loss of the surface dressing and/or the exposure of the minerals in the asphalt; the fine mortar is then removed from the surface.

Poor execution is revealed by:
- cracking;
- the opening of connections and joints.

Damage under water can be detected by, for example:
- surveying and/or probing extensive damage;
- divers;
- underwater photography;

If the situation allows, damage can be detected in for example reservoirs using:
- coloured or radio-active water pumped through cracks and holes;
- measurements of electrical conductivity.

Preferably, if it is possible, the water-level in a canal or reservoir should be lowered periodically so that any damage can be traced in dry conditions.

32.2 Mastic

Possible types of damage and its causes are given in Table 32.2.

Generally a mastic slab lies under water which makes it difficult to trace any damage. Sometimes the slab, if not covered with sand, is exposed at low water: visual inspection is then easier.
Poor execution is revealed, eventually, by poor jointing between adjacent panels or charges. In addition, if the viscosity of the mastic was too low when it was laid, the layer thickness can be reduced, particularly at joints and changes in the profile.

Damage can be detected under water by, for example:

- surveying and probing larger damage;
- divers;
- underwater photography.

- Reduced layer thickness can be detected by taking core borings or by using a special thickness probe which is a needle which is pressed through the layer at constant speed; the force required gives an indication of the layer thickness.

If the situation allows:
- coloured or radio-active water can be pumped through cracks and holes;
- the electrical conductivity can be measured.

If possible the water-level should, ideally, be lowered periodically so that damage can be traced in dry conditions. Damage, above water, can be traced in the way described for asphaltic concrete, see Section 32.1.

Table 32.2 Types of damage ot mastic revetments.

cracking	Lifting by water or current action	current	hydraulic uplift pressure	settlement	scouring	instability	collisions	anchors
Pieces of revetment broken out	x		x	x	x			
Flapping and/or breaking at the edges							x	x
Sliding off of part of the revetment		x			x			x
Grooves in and through the revetment						x	x	
Deformations			x		x	x		
Holes in or through the revetment								x

32.3 Grouting mortars

Possible types of damage and its couses are given in Table 32.3 for pattern grouting and surface grouting, and in Table 32.4 for fully grouting.

Larger damages, above water, can often be detected visually.
Incomplete grouting can be due to poor execution.

Damage under water is more difficult to detect and often only seen in side effects.
Such damages can be traced by:

- surveying or probing
- underwater photography or diving

The loss of loose stones or pieces of revetment can develop into more extensive damage. It is, therefore, important to notice this.

Table 32.3 Types of damage to incompletely grouted stone.

Type of damage	wave action	current action	hydraulic uplift pressures due to plants, mud sealing, and poor execution	settlement	scouring	instability	plants and seaorganism	collisions	anchors	vandalism/recreation
Loss of loose stones	x	x								
Loss of cohesion between stones in a 'stone lump' and stones being broken out	x	x		x	x		x			x
Sliding of part of the revetment	x	x								
Braking out of a whole 'stone lump'	x		x		x	x				
Sliding of part of the revetment			x			x		x		
Grooves in and through the revetment								x	x	

Table 32.4 Types of damage to fully grouted stone.

Types of damage	wave action	hydraulic uplift pressure	settlement	scouring	instability	vandalism/recreation	collision	anchors
Cracks in the grouting mortar between the stones	x	x	x	x	x		x	
Loss of stones or pieces of revetment (stone + asphalt)	x	x				x		
Deformation			x	x				
Sliding of a piece of revetment		x			x		x	
Grooves in and through the revetment							x	x

32.4 Dense stone asphalt

Table 32.5 gives possible types of damage and causes.

Damage above water can often be detected visually, cracks and open joints, however, can only be seen when they are fairly large and on the surface.

Core borings are needed to determine whether or not a crack goes completely through the layer; sometimes the seepage of water out of the revetment can be an indication of this. Surface wear can be detected by the exposure of the mineral in the asphalt.

Poor execution is revealed by:

- poor transverse joints and corrections;
- irregular layer thickness.

Table 32.5 Possible types of damage to dense stone asphalt.

Type of damage	wave impact	lifting by wave action	currents	hydraulic uplift pressure	settlement	scouring	instability	biological attack	effects of weather	collisions	anchors
Cracking	x	x		x	x		x	x			
Breaking out of pieces of revetment and hole formation			x							x	x
Breaking of parts of the edges of the revetment						x					
Sliding of part of the revetment				x			x			x	
Surface wear								x	x		
Grooves in and through the revetment										x	x

Damage under water can be detected by, for example:

- surveying or probing larger damages;
- under water photography and diving inspections.

32.5 Open stone asphalt

Table 32.6 gives possible types of damage and causes. Open stone asphalt is permeable and a filter should be used under the revetment. If the filter becomes exposed or is damaged, the whole dike can be endangered.

Table 32.6 Possible types of damage to open stone asphalt.

Type of damage	wave action	current action	settlement	scouring	instability	biological attack	vandalims/recreation	effects of weather	collisions	anchors
Cracking	x		x	x	x	x				
Breaking out pieces of revetment	x	x				x	x			x
Surface attack	x	x					x	x		
Flapping of mattres edges		x								x
Breaking of pieces of the edges of the revetment				x						
Sliding of part of the revetment					x					
Grooves in or through the revetment									x	x
Deformation			x							
Stripping								x		

Open stone asphalt revetments in situ have to be constructed above water. Damage can, therefore, be detected in the dry. Open stone asphalt mattresses can, however, be used under water and then tracing damage is more difficult.
Damage and external cracks above water can be detected visually. Larger damage under water can be traced by probing, diving inspection or by using under water photography. Erosion of the surface, is caused by the movement of coarse particles on the slope and can be detected visually.

Poor execution is revealed by:

- premature 'stripping' due to execution in a wet environment;
- the sealing of the revetment
- poor connections and transverse joints;
- variation in layer thickness.

32.6 Lean sand asphalt

Table 32.7 gives possible types of damage and its causes.

Table 32.7 Possible types of damage by lean sand asphalt.

Type of damage	wave attack	current action	settlement	scouring	instability	animals	plantgrowth	vandalism/recreation	effects of weather	collisions
Cracking	x		x	x	x		x			x
Breaking of pieces out of the surface	x	x					x	x		x
Sliding of part of the revetment					x	x				
Surface erosion/ attack	x	x						x	x	
Breaking of asphalt at the bottom on the slope				x						
Grooves in and through the revetment										x
Deformation			x							

Lean sand asphalt when used as a filter layer, sublayer and core material, is generally covered with a protection layer. The influence of external forces is, therefore, more limited and almost negligible compared with its use as a revetment material.

Surface damage or cracking, above water, is often visible; crack depth has to be determined by core borings.

Under water damage can be traced by:

- probing larger damages;
- diving investigations and underwater photography.

Erosion of asphalt is indicated by a smooth scouring of the surface.

Poor execution is revealed by:

- cracking caused by material adhering to the scoop of a crane during laying;
- premature stripping due to laying too little quantities in a wet environment;
- variation in layer thickness.

32.7 Membranes

Membranes are used to make, for example, reservoirs and canals watertight. They are generally covered with a protection material. The function of the membrane is lost if it is punctured. Damage can occur both above and below water-level. Detection under water is possible by:

- electrical conductivity measurements;
- pumping coloured or radio-active water through the damaged area;
- using underwater photography and divers in areas of more extensive damage;
- a drainage system.

If possible, the water-level should be lowered periodically, so that damage, if any, can be traced in the dry.

Damage above water-level can only be located if the membrane is not covered or if the protection layer itself indicates that the membrane below is damaged.

33 Methods of repair

When damage appears on a revetment the possible causes should first be investigated. If it appears that there is a design fault, then obviously, repairs to the original state will probably only lead to further failure. It may be better to modify the design. For example the instability of the subsoil, which causes the revetment to slide with it, can be due to faulty design. When repairing the dike and the revetment the cause of the damage, for example, too steep slopes or hydraulic uplift water pressure, should be removed.
Repair methods are discussed in the sections below, for each type of revetment material. A distinction is made between repairs above and below water-level. In general repairs above water-level are easier and can be executed more precisely; under water repairs are more difficult and not all types of mix can be used. In view of the lower level of accuracy which can be achieved and the possible problems related to heat conservation, larger quantities of material are used for underwater repairs. If possible underwater damage should be repaired in the dry, for example, by lowering the water-level or using a caisson. It is recommended that hot mix types are used for repairs rather than cold mix types. The edges of a damaged area should be clean to ensure that there is good adhesion with the repair material. In the case of underfilled or just filled mixes a tack coat should be applied.
It is also important to check and repair any damage to the filter layers. Underwater inspection can be carried out by divers.

33.1 Asphaltic concrete

ABOVE WATER

– Cracks, open connections and joints.
  The edges of cracks, less than 2 cm wide, should be cleaned with a high pressure air jet, preferably with hot air. If hot air is not used the edges must be dried with a flame; care being taken not to overheat the material. The crack is then filled with a rubber-based joint sealer.
  Very small cracks, less than 1 cm wide, should, if possible, be sealed with emulsion or a viscous tar bitumen. The repair is then strewn with sand or chippings, for example, chippings 2/6 mm. With this repair method, the binder should be carefully poured into the clean cracks; run out on to the surface should be avoided. Since, in general, the repairs are carried out on slopes, it is recommended that a binder with a relatively high viscosity is used.
  For very large cracks, 2 to 10 cm wide, the edges should be cleaned with a high pressure air jet and, if hot air is not available, should be dried with a flame, care being

taken not to overheat the material. The edges should then be tack-coated and sealed with mastic. The temperature should not be less than 130°C.

- Very broad cracks, wider than 10 cm and pieces of revetment broken out.
  Depending on the extent of the damage and the local availability of materials, the revetment material should be removed over the full thickness. The edges should be cut back to form a lap joint, see Figure 33.1, cleaned with a wire broom and given a tack-coat. The hole should then be filled with asphaltic concrete at a temperature greater than 130°C and compacted with a tandem vibratory roller.
  If the damage has occurred at a construction transition it is recommended that the connection is sealed with mastic (to which gravel might have been added).

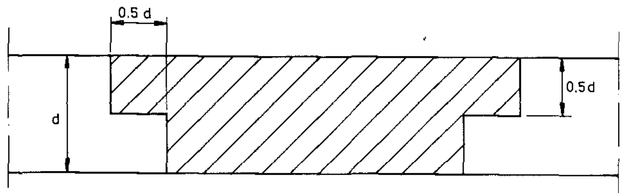


Figure 33.1 Method of repairing a hole in an asphaltic concrete revetment.

- Surface deterioration
  - Surface raveling and/or open surface texture.
    The damaged area must be thoroughly cleaned with a wire broom and then have a surface treatment.
  - Serious loss of surface material
    The damaged area must be mechanically fraized, see Figure 33.2., and the loose material removed. The area should then be tack-coated, filled with asphaltic concrete and compacted with a tandem vibratory roller, see Photos 30, 31 and 32. Small fraizing machines, similar to those used in road works, are suitable for this operation.
- Revetment which has slipped
  It is essential to determine the cause of slipping so that the design can be adapted, if possible. The old revetment should then be reconstructed.

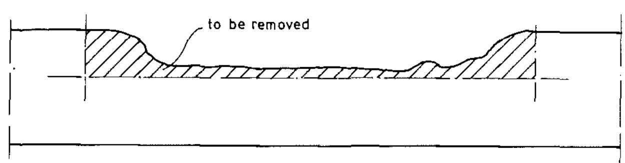


Figure 33.2 A severely deterorated surface of an asphaltic concrete revetment to be fraized.

UNDER WATER

All forms of repair work.

Sand must be cleaned from the damage area with water jets. Care should be taken not to enlarge the hole and to ensure that subsoil is not washed out. The area should then be filled or covered with:

- mastic (not on slopes);
- grouted stone (if extra weight is needed);
- dense stone asphalt.

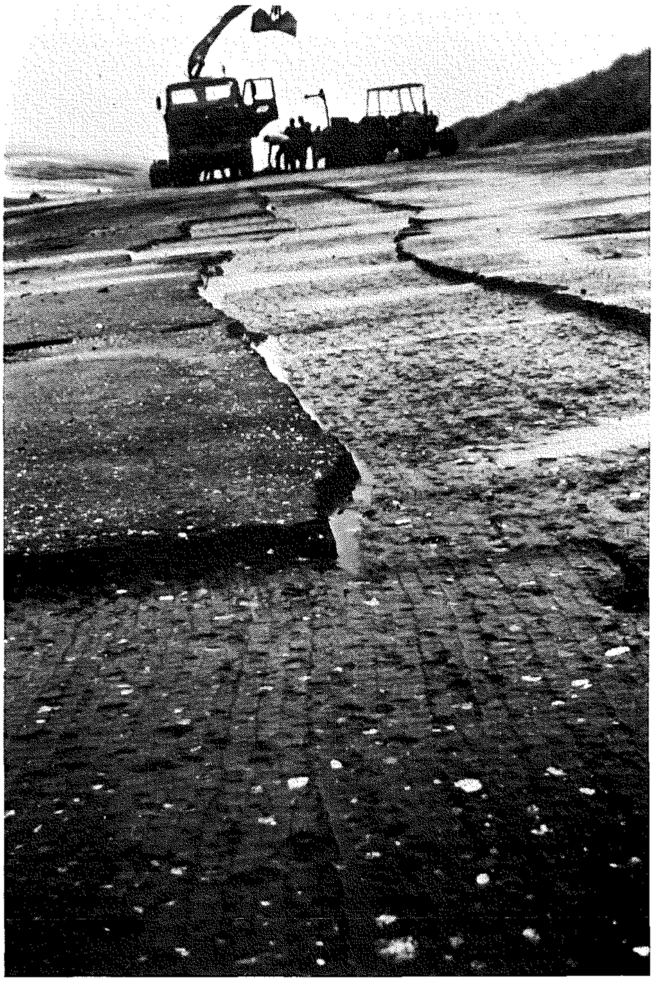


Photo 30 Repairing the seriously damaged surface of an asphaltic concrete revetment: The damaged area has been fraised out

Photo 31 Repairing the seriously damaged surface of an asphaltic concrete revetment: The fraised area is refilled with asphaltic concrete

Photo 32 Repairing the seriously damaged surface of an asphaltic concrete revetment: The refilled area is compacted with a tandem vibratory roller

During and after the repair, the work should be checked by divers. The method and extent of repairs are more or less dependent on local conditions such as currents and also the degree of accuracy which can be obtained.

33.2 Mastic

ABOVE WATER

- Cracks only develop when deformations are so large that the material cannot adjust. If the cause of the deformation cannot be removed quickly the damage to the mastic layer can spead. An emergency measure is to place a relatively large amount of mastic over the damaged area, after first cleaning it with a wire broom and/or high pressure airjets. The quantity of mastic used should be sufficient to obtain a good cohesion by heat transfer.
  When the deformation speed decreases the revetment material has sufficient time to flow. When no sand is trapped, cracks which do not break through the layer can close themselves.

- Reduction in layer thickness
  After a repair or when local conditions are changed, for example, when a slab is left unsupported after it has been undermined, a check should be made to ensure that there has been no reduction in layer thickness. Deformation can also lead to a reduction of layer thickness at joints and changes in the profile. If there has been a reduction the surface should be cleaned with wire brooms or high pressure air jets and an extra layer of mastic, minimum thickness 10 cm, provided. The edges of the repair area should be temporary seated in, in an order to reduce excessive heat losses during application.

- Poor sealing between adjacent changes and strips
  The edges should be cleaned with wire brooms or high pressure air jets and then the damaged areas filled with mastic. If the quantity of mastic is too small the edges of the opening should be heated to promote good adhesion.

- Loss of broken pieces of revetment, flapped or broken edges
  The damaged area should be cleaned with wire brooms or high pressure air jets and then filled with mastic. Broken edges should be re-made with an overlap, see Figure 33.3. If repairs have to be carried out on a slope, for example, after scouring, then it is better to use dense stone asphalt and possibly to lengthen the slab, see Figure 33.4.

- Revetment which has slipped
  The cause of slipping should be investigated and, if necessary the design adapted. A new revetment should then be laid.

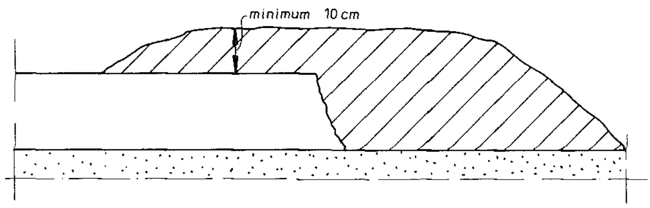


Figure 33.3 Repairs to a broken edge of a mastic slab with an overlap.

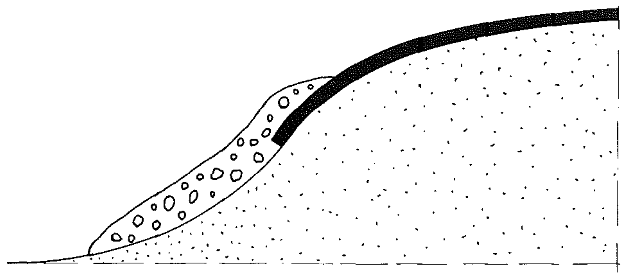


Figure 33.4 A broken slab repaired with dense stone asphalt.

UNDER WATER

All forms of repairs

Sand should be removed from the damaged area using water jets possibly with air. Care should be taken to ensure that the hole is not enlarged. The damaged area is then covered and overlapped with a new layer of satisfactory thickness, minimum 10 cm. The amount of repair material used should be sufficient to promote good sealing by heat transfer and the required overall stability, see Figure 35.5.

Dense stone asphalt or grouted stone should be used for repairs to slopes.

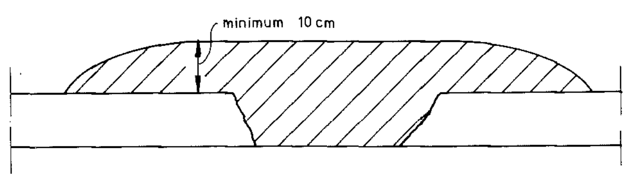


Figure 33.5 Repairs to a mastic slab under water.

33.3 Grouting mortars

33.3.1 *Pattern grouted stone*

If the damage points to insufficient grouting the degree of filling in the stone skeleton should be increased.

ABOVE WATER

- Loss of stones and stone lumps, weakened cohesion between stones, and grooves.
  The damaged area should be cleaned of reeds, waste and sand and if necessary re-filled with stone and then re-grouted.
- Inadequate grouting
  The revetment should be re-grouted with extra mortar.
- Areas of revetment which have slipped
  The cause of the damage should be found and the design, perhaps, modified. New revetment should then be laid.

UNDER WATER

All forms of repair.

- The damaged area should be cleaned and all traces of sand removed.
- The condition of the filter should be checked by divers.
- Stones should be placed and grouted or dense stone asphalt applied.

33.3.2 *Fully grouted stone*

ABOVE WATER

- Cracks
  Cracks should be cleaned with high pressure air jets and then sealed with mastic or grouting mortar, depending on the width of the crack.
- Loss of pieces of the revetment, holes and grooves
  The damaged area should be cleaned and the stones replaced and grouted.
- Areas of revetment which have slipped
  The cause of damage should be traced and the design, possibly, modified. A new revetment should then be laid.
- Inadequate grouting
  The stones must be regrouted, possibly with grout of a different composition. If the original grout did not penetrate fully, but hangs in the stones, then this must, first of all, be dug out.

UNDER WATER

All forms of repairs

- The damaged area must be cleaned and all traces of sand removed.
- The filter should be checked by divers.

– Stones should be replaced and grouted with an overlap around the edges, see Figure 33.6. An alternative is to use dense stone asphalt.

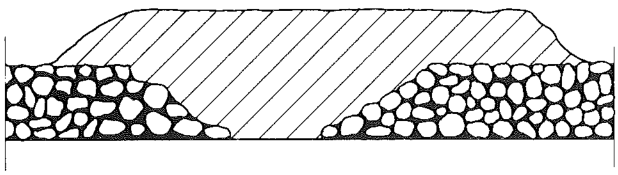


Figure 33.6 Repairs to a fully grouted crushed stone layer under water.

33.4 Dense stone asphalt

ABOVE WATER

– Cracks and open connections and joints
  Depending on the relative proportions of stone to mastic the dense stone asphalt can be regarded as a 'fluid' or a 'stable' mix. The breaking point is a ratio between mastic/stone of about 30/70. For a 'fluid' dense stone asphalt the method of repair is as described under mastic, Section 33.2; for 'stable' dense stone asphalt the repair method is as given under asphaltic concrete, Section 33.1.
– Broken pieces of revetment, holes, grooves and broken edges
  The full layer thickness of the damaged area should be removed and the edges dug out and cleaned, see Figure 33.1. A sufficient quantity of stone asphalt should then be laid on the damaged area to ensure an optimal heat transfer. If the edges are broken, the construction can be improved by making them there heavier.
– Areas of revetment which have slipped
  The caue of the damage should be investigated and, if necessary, the design adapted. A new revetment should then be laid.

UNDER WATER

All forms of repair.

– All traces of sand should be removed from the damaged area.
– The condition of the filter should be checked by divers.
– The damaged area should be covered by a sufficiently large load and overlapped in order to ensure a good adhesion by heat transfer and, also, in view of the limited accuracy achieved when working under water.

33.5 Open stone asphalt

It is very important with permeable materials to check and repair any damage to the filter.

Care should be taken, when repairing with a dense material, that the water permeability function is not lost; it is essential that a sufficient part of the surface stays open.
If the filter is damaged and cannot be repaired then sand can be lost through the revetment from the subsoil (Figure 33.7).

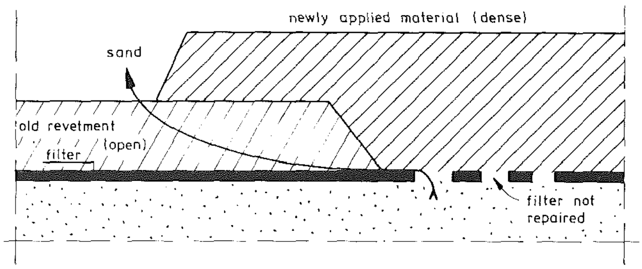


Figure 33.7 Loss of sand from under a poorly repaired stone asphalt revetment.

ABOVE WATER

- Cracks, badly sealed connections and joints, grooves
  Methods of repair are as discussed under asphaltic concrete, Section 33.1. When repairing wide cracks etc. open or dense stone asphalt can be used instead of mastic.
- Broken pieces, holes, flapped and/or broken edges
  The damaged area should be cut out straight over the full layer thickness. The edges should be cleaned, tack-coated and heated up. The area should then be filled with open stone asphalt. Flapped or broken edges can be made heavier by adding larger quantities of dense stone asphalt.
- Surface deterioration
  Small-scale local deterioration can be haulted by treating with mastic asphalt; previously the area has been cleaned. More serious local damage should be cut out completely over the full depth of the layer and a new layer, minimum thickness 12 cm, provided.
- Areas of revetment which have slipped
  The cause of damage should be investigated and, if necessary, the design should be adapted. A new revetment should then be laid.
- Accidental sealing of the revetment
  The revetment should be replaced with open stone asphalt of good composition. One reason for the loss of permeability is that the mastic sags from the limestone and forms an impermeable layer at the bottom.

UNDER WATER

All forms of repair

- All traces of sand should be removed from the damaged area.
- The condition of the filler should be checked by divers.

– The damage should be repaired using:
  - mastic (not on slopes);
  - grouted stone (if extra weight is needed);
  - dense stone asphalt.

  The repairs should overlap at the edges.

33.6 Lean sand asphalt

33.6.1 *Lean sand asphalt filter layers*

A filter is an essential component of a dike, and, when repaired, it must always be sandtight. Since this layer is generally covered, damage cannot be seen unless some of the protection is removed.

ABOVE WATER

– Cracks, holes, broken pieces, grooves, connections and joints
  The damaged area should be cut out over the full depth of the revetment. The edges should be cleaned, tack-coated and heated up.
  Damage can be repaired using:
  - mastic (small damage);
  - lean sand asphalt (large damage);
  - small areas of damage can also be repaired with a piece of filter cloth.

– Areas of revetment which have slipped
  The cause of damage should be traced and, if necessary, the design adapted. A new revetment should then be laid.

UNDER WATER

All forms of repair.
All traces of sand should be removed from the damaged area and then lean sand asphalt should be laid in sufficiently large loads of about 1000 kg with overlaps at the edges. The minimum layer thickness of the repair material should be 0.7 m. In view of the danger of stripping the mix temperature should not be higher than 100 °C.
It is essential that divers check the repairs.

33.6.2 *Lean sand asphalt reclamation bunds*

Since reclamation bunds generally have only a temporaty function, some damage during the construction phase is acceptable.

ABOVE WATER

If the definitive revetment is damaged it may be necessary to repair the bund. This can be done in the following ways:

– Small areas of damage can be repaired with mastic, after the surface has been cleaned and treated with a tack coat.
– More extensive damage should be repaired with lean sand asphalt. The surface should first be cleaned, tack-coated and heated up. The minimum layer thickness should be 15 cm.

UNDER WATER

Repairs which are essential, because the stability of the structure is endangered, should be carried out with large loads of at least 1000 kg so that the material does not cool off too quickly and because of the material behaviour under water. The minimum layer thickness should be 0.7 m and the maximum mix temperature 100°C.
All traces of sand should be removed from the damaged area before the repair work commences.

33.7 Membranes

Repairs to all sorts of damage:
– The membrane is first cleaned after removing the protection layer, if any.
– In the case of in situ membranes a new membrane is sprayed on; for pre-fabricated membranes a plaster should be attached over the damaged area. This is a piece of membrane the size of which depends on the extent of the damage.

Badly made joints in pre-fabricated membranes should also be covered with a plaster. Open joints should be covered with mastic or, on steep slopes, with dense stone asphalt, see Figure 33.8.

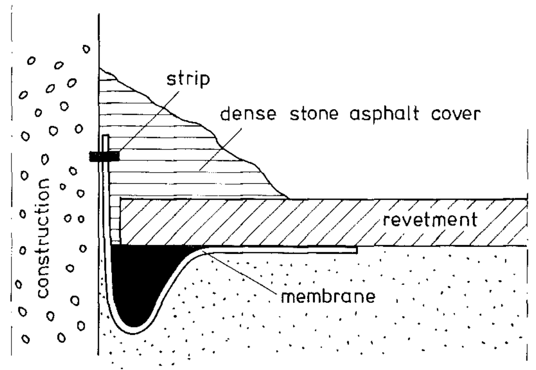


Figure 33.8 Method of covering open membrane joints.

UNDER WATER

Damaged areas should be cleaned and all traces of sand removed. Repairs can be made with:

– mastic (no bitumen);
– dense stone asphalt.

Sand should be removed from open joints which should then be covered with mastic or dense stone asphalt, see Figure 33.9. The amount of overlap depends on the accuracy with which the repair can be executed; it is usually between 0.5 and 2 m.
Open joints can be repaired as shown in Figure 33.9.

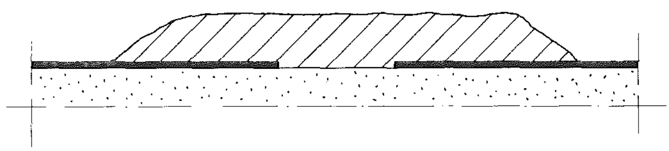


Figure 33.9 Repairing an open membrane joint under water.

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APPENDICES

Appendix I

Designs a plate-type asphalt revetment against wave impacts

I.1 Design of a plate-type revetment on a line load

I.1.1 *A calculation model for a plate-type revetment on a single line load*

A breaking wave will exert a short-term impact load on a plate-type asphalt revetment. Since the length of the impact is very large compared with its width, the wave impact has been schematized, as a line load, which, with time has a block form (Fig. 1). In the scheme, the visco-elastic subgrade reaction is characterised by a dashpot-spring (Fig. 2). The mass of the subsoil is effectively added by the mass of the plate.

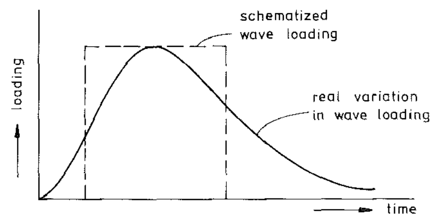

Figure 1 Variation of wave impact forces with time.

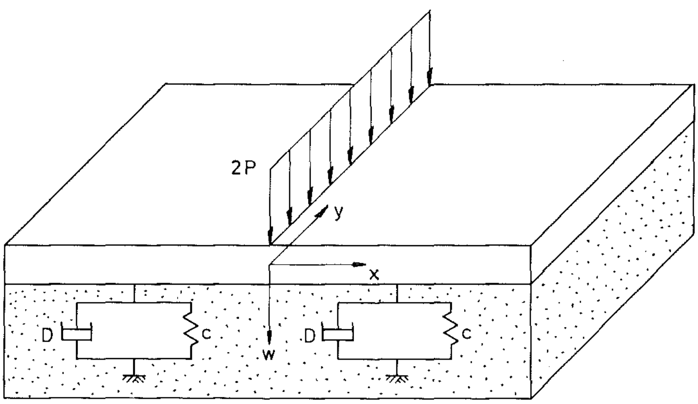

Figure 2 Schematisation of the sytem.

The deflection of the plate, due to a line load P (N/m^1), can be described by the following equilibrium equation:

$$K \frac{\partial^4 w}{\partial x^4} + M \frac{\partial^2 w}{\partial t^2} + D \frac{\partial w}{\partial t} + cw = 0$$

in which:

$K = \dfrac{E \cdot h^3}{12(1-\upsilon^2)}$ is the bending stiffness of the plate

E = the modulus elasticity (N/m^2)
h = plate thickness (m)
υ = Poissons ratio for asphalt
M = mass of the plate + added mass of the subsoil (kg/m^2)
D = subsoil damping coefficient (Ns/m^3)
c = modulus of subgrade reaction (N/m^3)
w = deflection of the plate (m)
t = time (s)
x = horizontal axis (m)

In the calculation it is assumed that the plate is infinitely long in the y-direction; the derivatives are therefore zero.

Using the Laplace transformation (40).

$$\bar{w} = \int_0^\infty w e^{-st} \, dt$$

it follows that:

$$K \frac{\partial^4 \bar{w}}{\partial x^4} + Ms^2 \bar{w} + Ds\bar{w} + c\bar{w} = 0$$

The solution of this is:

$$\bar{w} = \frac{P}{8K\lambda^3 s} e^{-\lambda x} (\cos \lambda x + \sin \lambda x)$$

with: $\lambda = \beta \sqrt[4]{\left(\dfrac{s}{\gamma}\right)^2 + \dfrac{s}{\delta} + 1}$

$$\beta = \sqrt[4]{\frac{c}{4K}}$$

$$\gamma = \sqrt{\frac{c}{M}}$$

$$\delta = \frac{c}{D}$$

The equation of the deflection of the plate, w, can be determined by solving the Laplace-inverse. The terms $\partial w/\partial t$ and $\partial^2 w/\partial t^2$, in the general differential equation, after a time, become zero and the system damps out. In other words, the amount of bending tends to a limiting value of $\varkappa\infty$. The way in which the limiting value is reached depends on the amount of damping. The damping is normally expressed in, what is referred to as, the critical damping (56):

$$D = aD_{\mathrm{kr}}$$

The critical damping:

$$D_{\mathrm{kr}} = 2\sqrt{Mc}$$

Three basic cases can be distinguished:

$D > D_{\mathrm{kr}}$ supercritical damping
$D = D_{\mathrm{kr}}$ critical damping
$D < D_{\mathrm{kr}}$ subcritical damping

Figure 3 shows schematically how the system reacts, in these three cases, under a step line load.

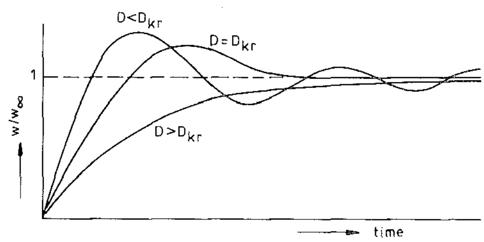

Figure 3 Reaction of a visco-elastic system under a step line load.

The inverse of the Laplace transformation can be determined for the case of no damping ($D=0$).
When $D=0$, $\delta=0$ and

$$\lambda = \beta \sqrt[4]{\left(\frac{s}{\gamma}\right)^2 + 1}$$

If $t \to \infty$:

$$w_{x,\infty} = \lim_{s\to 0} s\bar{w} = \frac{P}{8K} \lim_{s\to 0} \frac{e^{-\lambda x}(\cos\lambda x + \sin\lambda x)}{\lambda^3} = \frac{P}{8K\beta^3}\, e^{-\beta x}(\cos\beta x + \sin\beta x)$$

when $x=0$:

$$w_{0,\infty} = \frac{P}{8K\beta^3}$$

Therefore, when $\chi \times 0$:

$$\bar{w} = w_\infty \frac{1}{s\left[\left(\frac{s}{\gamma}\right)^2 + 1\right]^{3/4}}$$

$$w = w_\infty L^{-1}\left[\frac{1}{\left[\left(\frac{s}{\gamma}\right)^2 + 1\right]^{3/4}}\right]$$

After some modification it follows that (with $\tau = \gamma \cdot tl$):

$$w = w_\infty \frac{\sqrt{\pi}}{2^{1/4}\Gamma(\frac{3}{4})} \frac{1}{2^{1/4}\Gamma(\frac{5}{4})}\left[\frac{\tau^{3/2}}{3/2} - \frac{\tau^{7/2}}{7/2\cdot 1\cdot 5} + \frac{\tau^{11/2}}{11/2\cdot 1\cdot 2\cdot 9\cdot 5} - \frac{\tau^{15/2}}{15/2\cdot 1\cdot 2\cdot 3\cdot 13\cdot 9\cdot 5} + \dots\right]$$

$w/w\infty$ is plotted against $\gamma \cdot t$) in Figure 4.

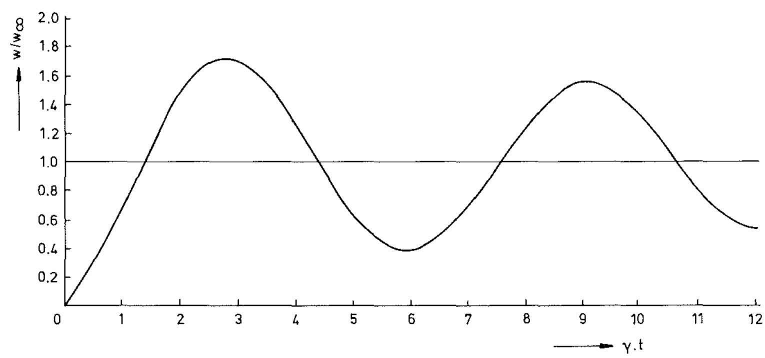

Figure 4 Relative deflection ($\gamma \cdot tl$) plotted against.

In order to calculate the required revetment thickness it is necessary to know the bending stresses involved. As a dimensioning criterion it is assumed that the asphalt stress at break may not be exceeded.
N.B.: A second criterion which can be used is a particular value of the deformation of the sandbed, in relation to stability, which must not be exceeded.

In order to determine the bending stress it is necessary to know the moments, m, which develop.

$$m = -K\frac{\partial^2 w}{\partial x^2}$$

$$\bar{w}=\frac{P}{8K\lambda^3 s}\, e^{-\lambda x}(\sin \lambda x-\cos \lambda x)$$

$$\frac{\partial^2\bar{w}}{\partial x^2}=-\frac{P}{4K\lambda s}\, e^{-\lambda x}(\sin \lambda x-\cos \lambda x)$$

$$\bar{m}=\frac{P}{4s\lambda}\, e^{-\lambda x}(\sin \lambda x-\cos \lambda x)$$

Again, the case of no damping ($D=0$) is considered.
If k$t \to \infty$:

$$m_{x,\infty}=\lim_{s\to 0} s\bar{m}=\frac{P}{4}\lim_{s\to 0}\frac{e^{-\lambda x}(\sin \lambda x-\cos \lambda x)}{\lambda}=\frac{P}{4}\,\frac{e^{-\beta x}(\sin \beta x-\cos \beta x)}{\beta}$$

the momentum is a maximum at $x = 0$:

$$m_{0,\infty}=\frac{P}{4\beta}$$

at $\chi = 0$

$$\bar{m}=m_\infty\cdot\frac{1}{s\left[\left(\frac{s}{\gamma}\right)^2+1\right]^{1/4}}$$

$$m=L^{-1}\left[\;m_\infty\,\frac{1}{s\left[\left(\frac{s}{\gamma}\right)^2+1\right]^{1/4}}\right]$$

which, with further treatment, becomes:

$$m=m_\infty\,\frac{2^{1/4}\sqrt{\pi}\,2^{1/4}}{\Gamma\left(\frac{1}{4}\right)\Gamma\left(\frac{3}{4}\right)}\left[2\tau^{1/2}-\frac{\tau^{5/2}}{5/2\cdot 1\cdot 3}+\frac{\tau^{9/2}}{9/2\cdot 1\cdot 2\cdot 7\cdot 3}-\frac{\tau^{13/2}}{13/2\cdot 1\cdot 2\cdot 3\cdot 11\cdot 7\cdot 3}+\dots\right]$$

with $\tau = \gamma t$.

m/m_∞ is plotted against $\gamma \cdot t$ in Figure 5.
As can be seen in the figure the maximum moment is about 1.22 m_∞.

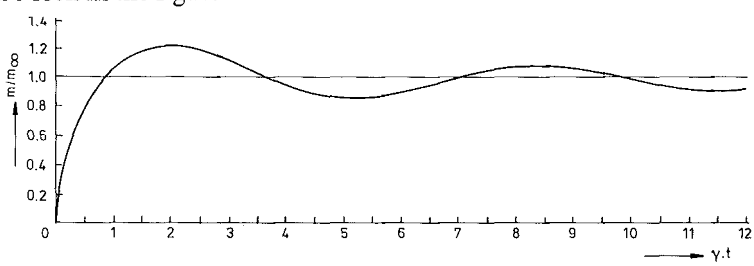

Figure 5 The relative moment m/m_∞ plotted against $\gamma \cdot t$.

In the analysis below a moment of m_∞, is used. The factor, 1.22, has been given a value of 1.00, for the following reasons:

- Subsoil damping, which would reduce this factor, is, in fact, not included in the analysis.
- Further analysis indicates that the factor 1.22 has hardly any influence on the layer thickness calculation.
- It has been found, in similar road construction situations, that the static linear elastic solution, i.e. $m = m\infty$, is sufficient.

Thus:

$$m = \frac{P}{4\beta} = \frac{P}{4\sqrt[4]{\frac{c}{4K}}}$$

with:

$$\sigma = m/W$$

in which:

σ = developed bending stress (N/m^2)
W = resisting moment = $1/6h^2$
h = layer thickness (m)

from this it follows that the layer thickness can be found from:

$$h^2 = \frac{6m}{\sigma_b}$$

in which:

σ_b = asphalt stress at break (N/m^2)

that is:

$$\boxed{h = \sqrt[5]{\frac{P^4}{\sigma_b^4}A} \quad \text{met} \quad A = \frac{27S}{16(1-\upsilon^2)c}}$$

In this equation E, is replaced by S, which represent the modulus of stiffness of the asphalt.

With this derivation a formula is obtained by which the thickness of an asphalt revetment can be determined in relation to a particular line loading, for example, a wave impact of finite size P. The number of times that the wave impact can occur is taken into account in σ_b which is dependent on the loading frequency.

In practice a wave impact is not linear but distributed, and a reduction in the calculated layer thickness, with a factor of 0.75, is tentatively accepted for Dutch conditions.

Thus, for a wave impact:

$$\boxed{h = 0{,}75 \sqrt[5]{\frac{P^4}{\sigma_b^4} A}}$$

I.1.2 *Calculation of a layer thickness, subjected to more than one wave impact*

The formula developed in Appendix I.1.1. is only suitable for calculating the layer thickness under one particular load. In practice a dike revetment will be attacked by a large number of different wave forces each of which can occur a number of times.

If a plate has a certain thickness, h, then the maximum stress which will develop in the plate, as the result of a wave impact of size P, is given by:

$$\sigma = P\left(\frac{A}{h^5}\right)^{1/4}$$

(Assuming that the reduction factor, 0.75, is neglected, for the time being).

It is assumed that all the wave impacts have the same duration and that the temperature remains constant.
For a particular type of asphalt mix, the factor:

$$\left(\frac{A}{h^5}\right)^{1/4} = \text{constant}$$

that is, $\sigma = P \cdot c$ *where* c is a constant.

Asphalt is sensitive to fatigue: a stress, σ, can only be applied a number of times, N, to the revetment before fatigue failure occurs.
The general fatigue relation is often expressed as:

$$N = k\sigma^{-a}$$

in which:

k and a are constants for a particular type of mix with a certain modulus of stiffness.
N is the number of impacts, of size σ, that the revetment can receive.

In the following it is assumed that Miners rule is valid. This states that, for each load cycle of a certain value, on a material which, with this load, has a lifetime of N cycles, after n loading cycles the damage will be n/N.
Break or failure will occur when the sum of the individual damages reaches a value of k1. Thus: for a combination of n_i loading cycles of amplitude σ_i the breaking limit is reached when the following conditions is satisfied:

$$\sum_{i=1}^{j} \frac{n_i}{N_i} = 1 \quad \text{ofwel:} \quad \frac{n_1}{N_1} + \frac{n_2}{N_2} + \frac{n_3}{N_3} + \dots + \frac{n_j}{N_j} = 1$$

By substituting $N = k \cdot \sigma^{-a}$ in this formula:

$$\frac{n_1}{k\sigma_1^{-a}} + \frac{n_2}{k\sigma_2^{-a}} + \dots + \frac{n_j}{k\sigma_j^{-a}} = 1$$

that is:

$$n_1\sigma_1^a + n_2\sigma_2^a + \dots + n_j\sigma_j^a = k$$

with $\sigma_i = c \cdot P_i$, it follows that:

$$n_1(P_1c)^a + n_2(P_2c)^a + \dots + n_j(P_jc)^a = k$$

$$c = \left(\frac{A}{h^5}\right)^{1/4}$$

thus:

$$n_1P_1^a + n_2P_2^a + \dots + n_jP_j^a = \frac{kh^{\frac{5a}{4}}}{A^{a/4}}$$

that is:

$$h = \sqrt[5]{\frac{[n_1P_1^a + n_2P_2^a + \dots n_jP_j^a]^{4/a}}{k^{4/a}} A}$$

$$h = \sqrt[5]{\left[\frac{\sum_{i=1}^{j} n_iP_i^a}{k}\right]^{4/a} A}$$

This formula can be used for calculating the thickness of an asphalt revetment under a variable number of wave impacts of different sizes and number of occurrences.
The factor a, is about 3 for mixes with a low PI and 7 for mixes with a high PI. For normal, mixes, a is, generally 5.

I.2 Application of the wave impact formula for situations with varying waves and waterlevels

I.2.1 *General*

In the formula developed in Appendix I.1.2. P_i represents a wave impact which occurs n_i times in the lifetime of the revetment. In order to determine these two parameters it is necessary to know the wave conditions in front of the dike. It is also significant to know where the wave impacts hit the slope, so that not all the waves in the wave train needs to be introduced into the calculation. The calculation must also take into account the fact that not all waves will cause an impact.
The wave conditions in front of the dike can be subdivided into a number of wave height and water-level related classes. The number of wave occurring in a particular class is given. By dividing the slope into a number of horizontal strips a division can be made based on the position of the wave impact. The number of impacts in a particular strip is used for dimensioning. This categorizing process is shown schematically in Figure 6.

		water level				
		d_1	d_2	d_3	....	d_j
wave height	H_1	n_{11}	n_{12}	n_{13}	....	n_{1j}
	H_2	n_{21}	n_{22}	n_{23}	....	n_{2j}
	H_3	n_{31}	n_{32}	n_{33}	....	n_{3j}
	H_4	n_{41}	n_{42}	n_{43}	....	n_{4j}
	....	....	....	....	....	....
	H_i	n_{i1}	n_{i2}	n_{i3}	....	n_{ij}

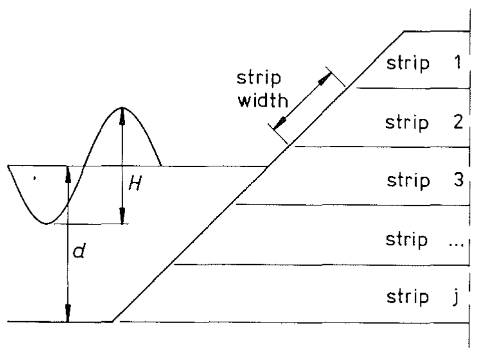

Figure 6 Subdivision of a wave field in front of a dike into a number of classes in which the number of waves are given in particular wave heights and water-level.

The wave height (H) at a particular water-level (d) at the dike and the frequency of occurrence of the wave height (n) can be obtained from such a scheme.
The size of the wave impact (P), the place of impact and the particular strip are then determined from the formulas given in Section 20.2.1. This is shown schematically in Table 1.

Table 1 Division of wave impacts into size and number in the various dike face strips.

strip 1		strip 2		strip 3		strip n		not on the cover layer	
P	n	P	n	P	n	P	n	P	n
ΣnP^a		ΣnP^a		ΣnP^a		ΣnP^a			

With this information the layer thickness can be determined for a particular strip, by working out the particular value of ΣnP^a in the wave impact formula.

I.2.2 *Practical method for designing a revetment for particular design conditions*

It is usual practice to dimension a dike to specific design conditions. The crest level, for example, is designed in relation to, what is referred to as, the design water-level. This is a water-level which has only a small chance of being exceeded. It will be natural to dimension the revetment also in relation to a particular design condition. A procedure for this is discussed below. For this purpose various schematisations and assumptions were made. The design is a super storm which the revetment should be able to withstand. It should be borne in mind, that asphalt suffers from fatigue and all wave impacts, during the lifetime of the revetment, tend to 'weaken' the material. The exceptional super storm must be resisted upto the end of the planned lifetime of the revetment even though the material will have been progressively weakened by wave action.
Applying Miner's law this can be stated in the form:

$$n_1(P_1c)^a + n_2(P_2c)^a + n_3(P_3c)^a + \dots n_j(P_jc)^a + n_s(P_sc)^a = k$$

P_1 to P_i, inclusive, designate the wave impacts with n_1 to n_j number of occurrences during the lifetime of the revetment at a particular water-level. P_s is the wave impact representing the superstorm at a particular water-level and n_s the number of times this impact will occur during the storm.
That is:

$$(P_sc)^a\left[n_1\left(\frac{P_1}{P_s}\right)^a + n_2\left(\frac{P_2}{P_s}\right)^a + \dots n_j\left(\frac{P_j}{P_s}\right)^a + n_s\right] = k$$

with $c = \left(\frac{A}{h^5}\right)^{1/4}$ it follows that:

$$h = \sqrt[5]{\frac{P_s n_s^{4/a}}{k^{4/a}} A\left[\Sigma\frac{n_i}{n_s}\left(\frac{P_i}{P_s}\right)^a + 1\right]^{4/a}}$$

This can also be written as:

$$h = f\sqrt[5]{\frac{P_s^4 n_s^{4/a}}{k^{4/a}}}A$$

This formula can be considered in two parts:

a. a factor f which characterises the asphalt fatigue due to loads which occurred before the superstorm. f is referred to as the 'fatigue factor';
b. the term $[P_s^4 n_s^{4/a}/k^{4/a}]^{0,2}$ which is used to dimension the layer thickness under design conditions. This referred to as the 'dimensioning part' of the wave impact formula.

The procedure is as follows: the revetment thickness is calculated with the design wave impact, P_s, with a definite number of loading cycles, n_s. In the case of fatigue due to previous loading the thickness is corrected with the factor f.

Evaluation of the designing part of the wave impact formula
The dimensioning part of the wave impact formula is given by:

$$h = \sqrt[5]{\frac{P_s^4 n_s^{4/a}}{k^{4/a}}}A$$

Design is carried out for design conditions, in this case, a superstorm. The superstorm is characterised by a wave spectrum with a significant wave height, H_s; the wave impact, P_s, being obtained directly from H_s. It is assumed that the waves in the spectrum have a Rayleigh probability distribution.
The chance that a wave height, H, is exceeded in the spectrum is (73) then:

$$P(H) = \exp\left\{-2\left(\frac{H}{H_s}\right)^2\right\}$$

The probability density function of this is:

$$p(H) = \frac{4H}{H_s^2}\exp\left\{-2\left(\frac{H}{H_s}\right)^2\right\}$$

The chance that a wave height, H, occurs in an interval range of ΔH is:

$$P(H - \Delta H/2) - P(H + \Delta H/2) = p(H)\Delta H$$

The wave spectrum can be subdivided into a number of groups of waves with height H_i all of which occur a number of times, n_i. If the spectrum is characterised by a significant wave, H_s, then this value of H_s must occur a number of times, n_s, to give the same fatigue loading as all the waves in the spectrum. This is the case when:

$$\sum_{i=1}^{j} n_i P_i^a = n_s P_s^a \quad \text{that is} \quad n_s = \sum_{i=1}^{j} n_i \left(\frac{P_i}{P_s}\right)^a$$

The wave impact, P is given by:

$$P = \varrho_w g t H b$$

in which:

ϱ_w = density of water (kg/m^3)
g = acceleration due to gravity (m/s^2)
t = a particular constant depending on slope angle
H = wave height (m)
b = width on which the wave impacts acts ($b = H/q$) (m)
q = a certain factor

thus:

$$P = \varrho_w g \frac{t}{q} H^2$$

then:

$$n_s = \sum_{i=1}^{j} n_i \left(\frac{H_i^2}{H_s^2}\right)^a$$

The assumption is made that, if m is the total number of waves in the spectrum during the storm, then the number of times that a wave height, H_i, occurs is given by $p(H_i) \cdot \Delta H \cdot m$. Now:

$$n_s = \sum_{i=1}^{j} p(H_i)\, \Delta H m \left(\frac{H_i^2}{H_s^2}\right)^a$$

that is:

$$n_s = \sum_{i=1}^{j} \frac{4H_i}{H_s} \exp\{-2(H/H_s)^2\} \frac{\Delta H}{H_s} m \left(\frac{H_i^2}{H_s^2}\right)^a$$

For $a = 5$, a typical value:

$$n_s = 3.75m$$

The total number of waves in a wave spectrum, or in this case, a storm, can be determined using:

$$m = \frac{T}{\bar{T}}$$

In which:

T = duration of the wave spectrum or, in this case, the storm

$\bar{T}$ = average wave period in the spectrum. This can be determined using empirical relationships in the form $H_s = A\bar{T}^B$, in which A and B are constants.

The water-level varies in a storm, for example, as shown in Figure 7. To take this into account the dike face could be divided into horizontal strips and the number of wave impacts, per strip, determined. This could lead to rather an involved calculation and, since insufficient is known about the variation in wave height and water-level during a storm, this approach is not attempted. More information about this subject is given in (74).

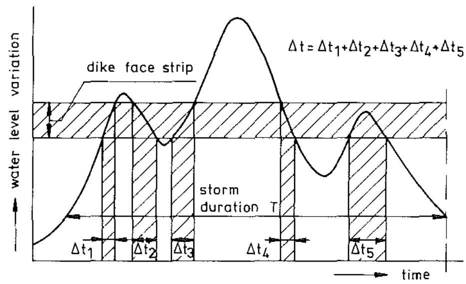

Figure 7 Example of water-level variation during a superstorm.

A practical form for the dimensioning part of the wave impact formula is developed below.

The previously developed relationship $n_s = 3{,}75\ T/\bar{T}$ has to be corrected by a factor of 0.1 for the following reasons:

- The water-level during the superstorm, to which a duration of 36 hours is ascribed (3 tidal cycles), is taken as constant.
- Only a small percentage of the waves cause an impact, and not all impacts are at the same place.

To determine n_s use can be made of the empirical relationship between the significant wave height, H_s, and the wave periode, T:

The North Sea: $\bar{T} = 3{,}94 H_s^{0{,}376}$

The entrance to the Eastern Scheldt: $\bar{T} = 3{,}55 H_s^{0{,}45}$

$\bar{T} = 3{,}36 H_s^{0{,}48}$

The following relationship has been assumed for present purposes:

$$\bar{T} = 3{,}5 H_s^{0{,}5}$$

then:

$$n_s = 0{,}1 \cdot 3{,}75 \cdot \frac{36 \cdot 3600}{3{,}5 \cdot H_s^{0{,}5}} = \frac{13885{,}7}{H_s^{0{,}5}}$$

Table 2 shows a number of values of H_s and the related values of n_s.

Table 2. Values of n_s related to different values of H_s.

H_s (m)	n_s	H_s (m)	n_s
2	9819	8	4909
3	8017	9	4628
4	6943	10	4391
5	6210	12	4008
6	5669	14	3711
7	5248	16	3471

The 'dimensioning part' of the wave impact formula reads:

$$h = \sqrt[5]{\frac{P_s^4 n_s^{4/a}}{k^{4/a}} \frac{27S}{16(1-\upsilon^2)c}}$$

with $n = k\sigma_b^{-a}$ (see Appendix I.1.2.) this can be written as:

$$h = \sqrt[5]{\frac{P_s^4}{\sigma_b^4} \frac{27S}{16(1-\upsilon^2)c}}$$

The breaking strength, σ_s, can be used as an input parameter for this latter formula. In this case σ_s should be a function of the number of loading cyles.
This size of the wave impact, P_s, can be deduced using the method given in Section 20.2.1.

b. *Determination of the fatigue factor in the wave impact formula*
The fatigue factor is represented by the parameter, *f*, in the formula.
It is determined as follows:

1. Select the revetment, or part, in an area which, under average conditions is not or is only slightly subject to wave attack but which can be attacked during superstorm conditions. This part of the revetment is then given a value of $f = 1$. This area is, for example, above the spring tide zone.
2. Select the revetment, or part, in an area which, under average conditions, is very exposed to wave attack. Then asphalt fatigue can occur and the factor *f* is definitely not equal to 1. This would apply, for example, in the spring tide zone.
   This 'fatigue area' also will be attacked, under extreme conditions, by the design loads, which could occur, for example for a dike with a deep foreshore. Then the fatigue factor is given by:

$$f=\left[\sum \frac{n_i}{n_s}\left(\frac{P_i}{P_s}\right)^5+1\right]^{4/25}$$

If the 'fatigue area' is not attacked by the design waves during extreme conditions, for example, a dike with a relatively high foreshore where the design storm surge develops on the upper part of the dike and, to a lesser extent or not at all in the normal tidal range, then is given by:

$$f=\left[\sum \frac{n_i}{n_s}\left(\frac{P_i}{P_s}\right)^5\right]^{4/25}$$

In other words: in the first case the significant design wave can strike the particular part of the revetment; in the second case it does not.

A solution for the summation-therm factor of the f is developed, below, for the areas described under Item (2) above. The analysis is carried out for a seadike but can be applied also to other situations.

$$F=\sum \frac{n_i}{n_s}\left(\frac{P_i}{P_s}\right)^5 \qquad \text{for } a = 5$$

Under Item b above it is shown that:

$$n_s = 3.75\,\frac{T}{\bar{T}}$$

in which:

T = design storm duration (s)
$\bar{T}$ = average wave period in the design storm (s)

Generally the normal wave condition in front of a dike is given in terms of significant wave height which, in turn, represents a specific wave field.
If a Rayleigh distribution of waves is assumed for the spectrum:

$$n_i^* = 3.75\,\frac{T_i^*}{\bar{T}_i^*} \qquad \text{(see Section I.5.1.)}$$

in which:

n_i^* = number of times that the significant wave height, H_i will occur in the wave field
T_i^* = duration of the wave field (s)
$\bar{T}_i^*$ = average wave period in the field (s)

The frequency of occurrence of the wave field in the lifetime of the cover layer, n_i^{**}, must also be determined.

$$n_i^{**} = p(H_i)\Delta H_i \frac{l \cdot 365 \cdot 24 \cdot 3600}{T_i^*}$$

in which:

$p(H_i)\Delta H_i$ = probability that a wave with a significant height, H_i (in a wave height range of ΔH_i) will occur

l = lifetime of the dike (years)

Now:

$$n_i = n_i^* \cdot n_i^{**}$$

From this:

$$\frac{n_i}{n_s} = \frac{3.75 \dfrac{T_i^*}{\bar{T}_i^*} \dfrac{p(H_i)\,\Delta H_i\, l \cdot 365 \cdot 24 \cdot 3600}{T_i^*}}{3.75 \dfrac{T}{\bar{T}}}$$

with:

$$T = 36 \cdot 3600$$

this becomes:

$$\frac{n_i}{n_s} = \frac{730\, p(H_i)\,\Delta H_i\, l \bar{T}}{3 \bar{T}_i^*}$$

Based again on the relationship between significant wave height and average wave period, $\bar{T}$, which can be expressed as:

$$\bar{T} = A H_{sig}^{B}$$

By substitution in the above equation:

$$\frac{n_i}{n_s} = \frac{730\, p(H_i)\,\Delta H_i\, l H_s^B}{3 H_i^B}$$

The size of the wave impact is given by:

$$P = \varrho_w g \frac{t}{b} H^2 \qquad \text{(see Appendix I.2.2b)}$$

Then:

$$\sum_{i=1}^{j} \frac{n_i}{n_s} \left(\frac{P_i}{P_s}\right)^5 = \sum_{i=1}^{j} 243{,}3\, p(H_i)\,\Delta H_i\, l \frac{H_i^{10-B}}{H_s^{10-B}}$$

with B = 0,5 this becomes:

$$\sum_{i=1}^{j} \frac{n_i}{n_s} \left(\frac{P_i}{P_s}\right)^5 = \sum_{i=1}^{j} 243{,}3\, p(H_i)\, \Delta H_i\, l \left(\frac{H_i}{H_s}\right)^{9{,}5}$$

Usually the design water-level for sea-dikes is assumed to be higher than for average conditions. Since, however, the wave height is limited by the water depth, this means that the maximum significant wave height in average conditions, H_i, is not larger than the design wave height, H_s.

In order to determine the summation factor, wave data are needed for the site at which the revetment will be used, for example, at the toe of the dike. Very often these data are not available.

In order to get an impression, calculations are made using wave data from Dutch light vessels located at various sites along the Dutch coast (see Table 3). (N.B.: use is only made of waves in those sectors which actually strike the coast).

Table 3 Probability of occurrence of wave heights $p(H_i)\Delta H_i$ at some locations off the Dutch coast (92).

	lightvessel		
H_i	Terschellingerbank	Texel	Goeree
0	0.034	0.043	0.061
0,5	0.167	0.229	0.175
1	0.164	0.220	0.225
1,5	0.104	0.165	0.180
2	0.06	0.0879	0.1116
2,5	0.928	0.0379	0.0491
3	0.011	0.0135	0.0236
3,5	0.004	0.0049	0.0079
4	0.0019	0.002	0.003
4,5	0.0007	0.0006	0.0011
5	0.0004	0.0008	0.0004
5,5	0.0005	0.0004	0.0003
6	0.0003	0.0003	0.0002
6,5	0.001	0.0003	0.0005

The calculations are then continued with the following assumptions:

- The waves, given in Table 3, although measured at some distance from the coast are assumed to act at the site of the dike. The maximum wave height, however, is restricted since the water depth at the dike is limited. Higher waves, are assumed to have broken by the time they reach the dike and have the maximum heights possible at the site have been assumed.
- Since the pricise position of the wave impact on the dike face cannot be determined, it is assumed, in fact, that the water-level does not vary. In order to compensate for this unfavourable assumption and since not every wave causes an impact the number of waves is reduced by a factor of 0.1.
- The Class 0 wave height in Table 3 are assumed to be 0.1 m.

The calculation is made for a design wave height of H_s = 2, 4, 6 and 8 m and for a maximum significant wave height (taking into account the water depth limitation) under average conditions of $H_{i\,max}$ of 0.5 H_s, 0.75H_s and H_s. Under these assumptions the basic equation becomes:

$$\sum_{i=1}^{j} \frac{n_i}{n_s} \left(\frac{P_i}{P_s}\right)^5 = 243{,}3\,/\,\frac{1}{H_s^{9,5}} \sum_{i=1}^{j} p\,(H_i)\,\Delta H_i\, H_i^{9,5}$$

An example of the calculation, for H_s = 4 m, at the Terschellingerbank lightvessel is given in Table 4.

Table 4 An example of the calculation of $\Sigma p(H_i)\Delta H_i H_i^{9,5}$ for H_s = 4 m, at the Terschellingerbank lightvessel.

H_s = 4 m, Terschellingerbank lightvessel								
$H_{i\,max} = H_s$			$H_{i\,max} = 0{,}75H_s$			$H_{i\,max} = 0{,}5H_s$		
H_i	$p(H_i)\Delta H_i$	$p(H_i)\Delta H_i H_i^{9,5}$	H_i	$p(H_i)\Delta H_i$	$p(H_i)\Delta H_i H_i^{9,5}$	H_i	$p(H_i)\Delta H_i$	$p(H_i)\Delta H_i H_i^{9,5}$
0	0.034	–	0	0.034	–	0	0.034	–
0.5	0.167	–	0.5	0.167	–	0.5	0.167	–
1	0.164	0.164	1	0.164	0.164	1	0.164	0.164
1.5	0.104	4.9	1.5	0.104	4.9	1.5	0.104	4.9
2	0.06	43.4	2	0.06	43.4	$\geqslant 2$	0.108	78.2
2.5	0.029	175	2.5	0.029	175			
3	0.001	375	$\geqslant 3$	0.019	648			
3.5	0.004	590						
$\geqslant 4$	0.004	2097						
totaal		3286	total		871	total		83

The fatigue factor, f, can be quantified, for certain places on the Dutch coast by making an estimate of the lifetime of the dike and applying Table 5.

I.3 Design of a fully grouted asphalt stone laye gainst wave impact

A fully grouted rip-rap cover layer is a layer of rip-rap the voids of which have been filled with mastic. The stones are completely coated with mastic and no account is taken of any adhesion between the stone and the mastic.
The grouted rip-rap is considered to act as a plate. If the plate bends compression forces are taken by the rip-rap and the mastic; tension forces are resisted by the mastic alone. (It is assumed that shear forces in the tension zone, between the mastic and the rip-rap, can be transmitted.

A summary of the calculated values is given in Table 5.

Table 5. Several calculated values for $\Sigma n_i/n_s(P_i/P_s)^5$ for different values of H_s en H_{imax}/H_s and for several locations along the Dutch Coast, l = lifetime in years.

	$\sum \frac{n_i}{n_s}\left(\frac{P_i}{P_s}\right)^5$		
	$H_{i\,max} = H_s$	$H_{i\,max} = 0{,}75H_s$	$H_{i\,max} = 0{,}5H_s$
H_s [m]	Terschellingerbank		
2 4 6 8	28,2 l 1,72 l 0,42 l 0,045l	3,4 l 0,42 l 0,067l 0,017l	0,13 l 0,039l 0,007l 0,002l
H_s [m]	Texel		
2 4 6 8	39 l 1,76 l 0,026l 0,022l	5,0 l 0,50 l 0,063l 0,017l	0,18 l 0,05 l 0,011l 0,002l
H_s [m]	Goeree		
2 4 6 8	51 l 2,43 l 0,28 l 0,10 l	6,0 l 0,76 l 0,08 l 0,018l	0,20 l 0,07 l 0,016l 0,003l

The various materials are assumed to react in the way shown in Figure 8.

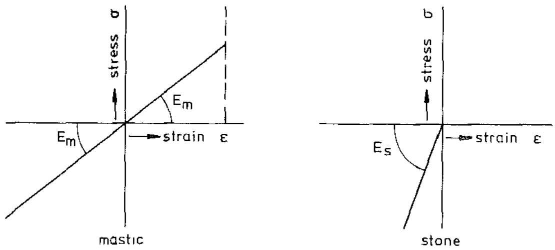

Figure 8 Schematised stress-strain diagrams for mastic.

The calculations also assume that, when bending occurs, plane sections remain plane (fig. 9).

Generally it is valid that:

$$\begin{bmatrix} \sum_{i=1}^{n} E_iA_i & \sum_{i=1}^{n} E_iS_i \\ \sum_{i=1}^{n} E_iS_i & \sum_{i=1}^{n} E_iI_i \end{bmatrix} \cdot \begin{bmatrix} \varepsilon \\ \kappa \end{bmatrix} = \begin{bmatrix} F \\ M \end{bmatrix}$$

in which: $\sum_{i=1}^{n} E_i A_i$ = the sum of the surfaces of the cross-section

$\sum_{i=1}^{n} E_i S_i$ = the sum of the static moments related to the neutral axis

$\sum_{i=1}^{n} E_i I_i$ = the sum of the moments of inertia related to the neutral axis

ε = strain

κ = de kromming van de doorsnede

F = force normal to the neutral axis

M = momentum in the cross section

In cases of simple bending, with respect to the neutral axis, $F = 0$ and $\varepsilon = 0$.

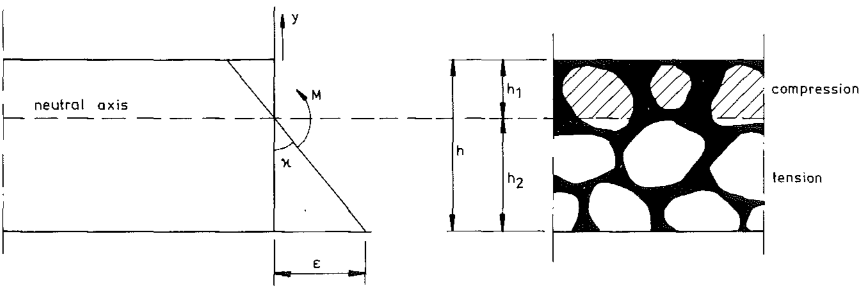

Figure 9 The problem schematised.

Rip-rap comprises about 40% voids. It is assumed that the surface of the cross section comprises 60% rip-rap and 40% mastic, uniformly distributed.

compression zone	tension zone

$$\Sigma E_i A_i = 0{,}4 \frac{E_m}{(1-\upsilon_m^2)} h_1 + 0{,}6 \frac{E_s}{(1-\upsilon_s^2)} h_1 \qquad + 0{,}4 \frac{E_m}{(1-\upsilon_m^2)} h_2$$

$$\Sigma E_i S_i = \frac{E_m}{(1-\upsilon_m^2)} \int_0^{h_1} y\, 0{,}4 \, dy + \frac{E_s}{(1-\upsilon_s^2)} \int_0^{h_1} y 0{,}6 \, dy \quad + \frac{E_m}{(1-\upsilon_m^2)} \int_{-h_2}^{0} y 0{,}4 \, dy$$

$$= \frac{E_m}{(1-\upsilon_m^2)} 0{,}2 \, h_1^2 + \frac{E_s}{(1-\upsilon_s^2)} 0{,}3 \, h_1^2 \qquad - \frac{E_m}{(1-\upsilon_m^2)} 0{,}2 \, h_2^2$$

$$\Sigma E_i I_i = \frac{E_m}{(1-\upsilon_m^2)} \int_0^{h_1} y^2 0{,}4 \, dy + \frac{E_s}{(1-\upsilon_s^2)} \int_0^{h_1} y^2 0{,}6 \, dy + \frac{E_m}{(1-\upsilon_m^2)} \int_{-h_2}^{0} y^2 0{,}4 \, dy$$

$$= \frac{E_m}{(1-\upsilon_m^2)} 0{,}4 \cdot 1/3 \; h_1^3 + \frac{E_s}{(1-\upsilon_s^2)} 0{,}6 \cdot 1/3 \; h_1^3 + \frac{E_m}{(1-\upsilon_m^2)} \cdot 0{,}4 \cdot 1/3 \; h_2^3$$

in which:

E_m = mastic modulus of stiffness
υ_m = Poissons ratio for mastic
E_s = elastic modulus for stone
υ_s = Poissons ratio for stone
h_1 = height of the compression zone
h_2 = height of the tension zone

The two following equations come from the matrix:

1. $\Sigma E_i S_i = 0$ in relation to the neutral axis
2. $(\Sigma E_i I_i)\kappa = M$ in relation to the neutral axis

Using Equation (1):

$$\tfrac{1}{2}h_1^2 \left(\frac{0{,}4E_m}{(1-\upsilon_m^2)} + \frac{0{,}6E_s}{(1-\upsilon_s^2)}\right) = \tfrac{1}{2}h_2^2 \; \frac{0{,}4E_m}{(1-\upsilon_m^2)}$$

$$h_2 = h_1 \sqrt{1 + \frac{0{,}6E_s}{0{,}4E_m}\,\frac{(1-\upsilon_m^2)}{(1-\upsilon_s^2)}}$$

with $h_1 + h_2 = h$ it follows that

$$h = h_1 + h_1 \sqrt{1 + \frac{0{,}6E_s}{0{,}4E_m}\,\frac{(1-\upsilon_m^2)}{(1-\upsilon_s^2)}}$$

that is

$$h_1 = \frac{h}{1 + \sqrt{1 + \frac{0{,}6E_s}{0{,}4E_m}\,\frac{(1-\upsilon_m^2)}{(1-\upsilon_s^2)}}}$$

and

$$h_2 = \frac{h \cdot \sqrt{1 + \frac{0{,}6E_s}{0{,}4E_m}\,\frac{(1-\upsilon_m^2)}{(1-\upsilon_s^2)}}}{1 + \sqrt{1 + \frac{0{,}6E_s}{0{,}4E_m}\,\frac{(1-\upsilon_m^2)}{(1-\upsilon_s^2)}}}$$

in which the neutral axis remains fixed

Using Equation (2):

$$M = \kappa \Sigma EI$$

in which:

$$\kappa = \frac{-\partial^2 w}{\partial x^2}$$

then:

$$M = -\Sigma EI \frac{\partial^2 w}{\partial x^2}$$

The general equilibrium equation for a plate lying on a visco-elastic subsoil, which is represented by a dashpot-spring model subjected to a load of $q(xl)$, can be derived from a vertical balance equation (Figure 10).

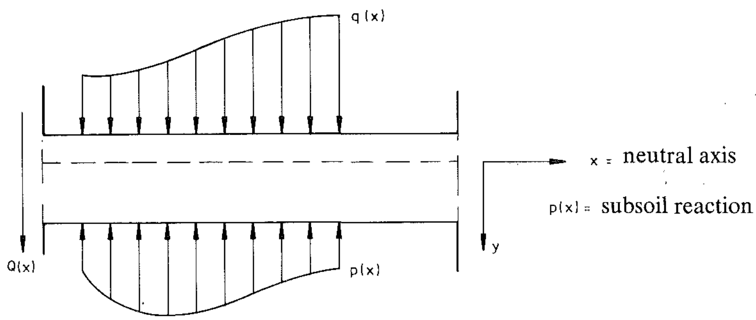

Figure 10 Plate subjected to a load $q(x)$ and a subsoil reaction $p(x)$.

The equation is:

$$\frac{\partial Q}{\partial x} + q(x) - p(x) = F$$

Q = shear force

$$\frac{\partial Q}{\partial x} = \frac{\partial^2 M}{\partial x^2} = -\Sigma EI \frac{\partial^4 w}{\partial x^4}$$

$p(x)$ = the subsoil reaction

$p(x) = cw + D \dfrac{\partial w}{\partial t}$

D = subsoil damping

c = modulus of subgrade reaction

$F = M \dfrac{\partial^2 w}{\partial t^2}$

M = plate weight + weight of the related subsoil

The equilibrium equation thus becomes:

$$-\Sigma EI \frac{\partial^4 w}{\partial x^4} + q(x) - cw - D\frac{\partial w}{\partial t} = M\frac{\partial^2 w}{\partial t^2}$$

For a step line load P, for the unloaded section, $q(x) = 0$, so that

$$\Sigma EI \frac{\partial^4 w}{\partial x^4} + M \frac{\partial^2 w}{\partial t^2} + cw + D \frac{\partial w}{\partial t} = 0$$

The solution of this differential equation is in Appendix I.1.1.
For the static solution, at $x = 0$, the term:

$$\frac{\partial^2 w}{\partial x^2} = -\frac{P}{4\Sigma EI \sqrt[4]{\dfrac{c}{4\Sigma EI}}}$$

with

$$\kappa = -\frac{\partial^2 w}{\partial x^2}$$

this becomes:

$$\kappa = \frac{P}{4\Sigma EI \sqrt[4]{\dfrac{c}{4\Sigma EI}}}$$

It is assumed, for a dimensioning criteria, that the breaking stress of the mastic, σ_m, must not be exceeded.

$$\sigma_m = \frac{E_m}{(1 - \upsilon_m^2)} \kappa h_2$$

with

$$h_2 = \frac{h \cdot \sqrt{1 + \dfrac{0{,}6E_s}{0{,}4E_m} \dfrac{(1 - \upsilon_m^2)}{(1 - \upsilon_s^2)}}}{1 + \sqrt{1 + \dfrac{0{,}6E_s}{0{,}4E_m} \dfrac{(1 - \upsilon_m^2)}{(1 - \upsilon_s^2)}}}$$

that is:

$$h = \sqrt[5]{\frac{P_m^4}{\sigma_m^4} \frac{3375E^*}{512c}}$$

and

$$E^* = \frac{E_m}{(1 - \upsilon_m^2)} \left(\frac{1}{\sqrt{1 + \dfrac{0{,}6E_s}{0{,}4E_m} \dfrac{(1 - \upsilon_m^2)}{(1 - \upsilon_s^2)}}} + 1 \right)^2$$

The original formula reads:

$$h=\sqrt[5]{\frac{P^4}{\sigma_b^4}\;\frac{27S}{16(1-\upsilon^2)c}}$$

In order to use this formula, in which mastic parameters are used as asphalt parameters, the calculated value of h must be multiplied by a correction factor, z, in order to obtain the real layer thickness.

$$z=\sqrt[5]{\frac{125}{32}\left(\frac{1}{\sqrt{1+\frac{0{,}6E_s}{0{,}4E_m}\;\frac{(1-\upsilon_m^2)}{(1-\upsilon_s^2)}}}+1\right)^2}$$

The parameter, z, is illustrated schematically in Figure 11, for several values of E_s/E_m. Application of a given value of z is based on the assumption that shear stresses which develop in the plate between the mastic and the rip-rap can be transmitted. Further research should be carried out into the validity of this assumption.

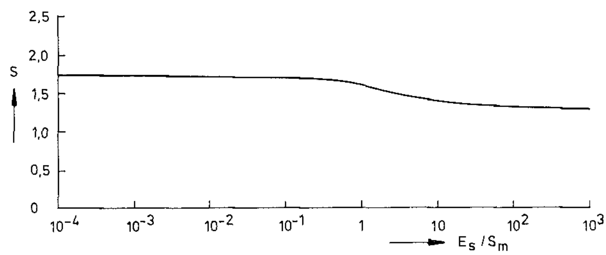

Figure 11 Values of the correction factor, z, plotted against elastic modulus of rip-rap/elastic modulus of mastic, E_s/E_m.

Appendix II

The lifting of a relatively watertight bed protection by wave action

Consider an impermeable plate, for example a mastic slab lying on a sandy bed under water, over which there are standing or translatory wave. The plate must not be lifted by the wave action.

Two cases can be considered:

- the wave length longer than the bed protection;
- the wave length shorter than the bed protection.

II.1 Wave length longer than the bed protection

The system can be schematized two dimensional. The bed protection perpendicular to the paper, is considered to be stiff and long. The width of the plate is l, and the thickness h.

The thickness of the waterbearing sand layer is B. The wave height is H and the wave length L (Figure 1).

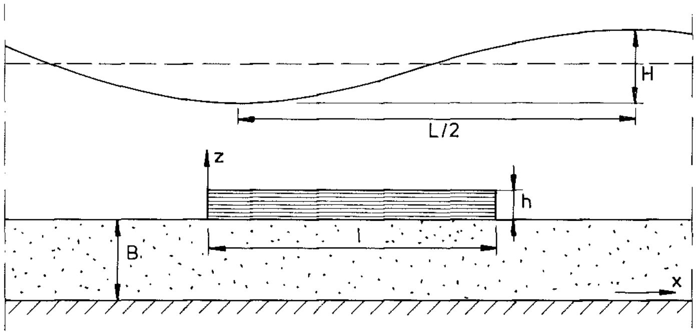

Figure 1 Watertight revetment under water.

The groundwater pressure under the plate will vary differently to the water pressure resulting from the wave action above the plate. This can result in an upward force. Barends gives a solution to this problem (79). The groundwater flow can be expressed as a linear storage equation:

$$(K/\gamma_w)\nabla^2 p = n\beta' \cdot \partial p/\partial t + \partial \varepsilon/\partial t$$

In addition there is an equilibrium equation:

$$(K_s + G/3)\nabla(\nabla u) + G\nabla^2 u = \nabla p$$

in which:

K = hydraulic permeability (m/s)
γ_w = specific weight of water (N/m^3)
p = pore water pressure (N/m^3)
n = porosity
β' = fluid/gas compressibility (m^2/N)
K_s = modulus of compression of the subsoil (N/m^2)
G = shear modulus of the subsoil (N/m^2)
u = displacement vector (m)
t = time (s)
ε = strain

For furter discussion of this problem reference should be made to (79).
An expression for the maximum uplift pressure, p_m, is developed below, and plotted below against $2\pi l^2/(c_v T)$ for a Poisson ratio $\vartheta = 1/3$.

T = wave period(s)
c_v = consolidation coefficient (m^2/s)

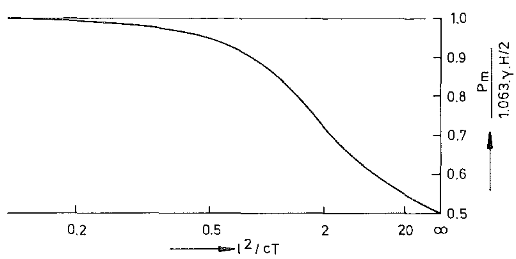

Figure 2 Maximum uplift pressure under the bed protection (79).

In order to prevent the plate being lifted the uplift pressure must be balanced by the weight of the plate:

$$p_m = h\gamma_a$$

where:

γ_a = volume weight of bottom protection (N/m^2)

The maximum excess pressure is given by:

$$p_m = 1{,}063\gamma_w \frac{H}{2} \frac{1}{\cosh(\lambda d)}$$

in which:

d = water depth (m)
γ = $2\pi/L$
L = wave length (m)
H = wave height (m)

Thus if:

$$h > 1{,}063\ \frac{\gamma_w}{\gamma_a}\ \frac{H}{2}\ \frac{1}{\cosh(\lambda d)}$$

II.2 Wave length much shorter than the bed protection

Consider a revetment lying under water on a bed of sand and loaded from above by a standing or translatory wave (Figure 5).
The general equilibrium equation for the plate reads (41):

$$K_p \frac{\partial^4 w}{\partial x^4} + \varrho h \frac{\partial^2 w}{\partial t^2} + 2D \frac{\partial w}{\partial t} + cw = q(x,t) + \varrho hg$$

in which:

K_p = bending stiffness of the plate (Nm)
ϱ = density of the plate (kg/m^3)
h = plate thickness (m)
$2D$ = subsoil damping (Ns/m^3)
c = modulus of subgrade reaction (N/m^3)
t = time (s)
x = location vector
q = loading due to wave action (N/m^2)
g = acceleration due to gravity (m/s^2)
w = vertical deflection of the plate (m)

The subsoil can be schematized as a dashpot-spring model (Figure 3). In this model the spring represents the elastic reaction of the bed and the dashpot the damping effect resulting from pore water movement. It is assumed that plate bending can be neglected, an assumption which is valid if the plate is very stiff (Kl very large) or very flexible ($K \rightarrow 0$).

The differential equation then becomes:

$$\varrho h \frac{\partial^2 w}{\partial t^2} + 2D \frac{\partial w}{\partial t} + cw = q(x,t) + \varrho gh$$

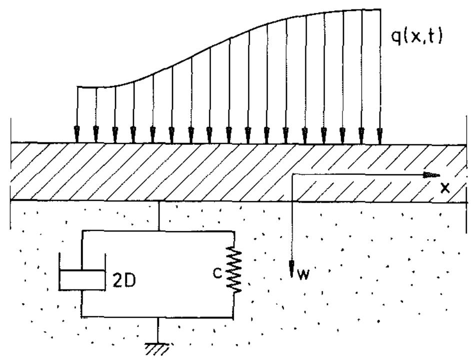

Figure 3 Schematization of the subsoil.

The wave action is assumed to be in the form of a cycli loading:

$$q(x,t) = q_0 \sin(\alpha t) \qquad (t \geqslant 0)$$

q_0 = loading amplitude
α = angular velocity of the loading

The solution of the differential equation reads:

$$w = A \exp\left[\frac{-D+\sqrt{D^2-\varrho hc}}{\varrho h}\right] t + B \exp\left[\frac{-D-\sqrt{D^2-\varrho hc}}{\varrho h}\right] t + \frac{\varrho hg}{c} +$$

$$+\frac{(c-\varrho h\alpha^2)q_0 \sin(\alpha t) - 2D\alpha q_0 \cos(\alpha t)}{(c-\varrho h\alpha^2)^2 + 4D^2\alpha^2}$$

(A and B are constants).

Boundary conditions: the loading $q(x, t)$ commences at $t = 0$, thus

$$t=0: \qquad w = \frac{\varrho hg}{c}$$

$$\frac{\partial w}{\partial t} = 0$$

Using the boundary conditions the integration constants A and B, can be found. The complete solution of the differential equation then reads:

$$w = \frac{2\alpha q_0 e^{-\frac{D}{\rho h}t}[(2D^2-(c-\varrho h\alpha^2)\varrho h) \sinh u + (2D\sqrt{D^2-\varrho hc}) \cosh u]}{2\sqrt{D^2-\varrho hc}[(c-\varrho h\alpha^2)^2+(2D\alpha)^2]} +$$

$$+\frac{\varrho hg}{c} - \frac{2D\alpha q_0 \cos(\alpha t) - (c-\varrho h\alpha^2)q_0 \sin(\alpha t)}{(c-\varrho h\alpha^2)^2 + 4D^2\alpha^2}$$

in which: $= \dfrac{\sqrt{D^2-\varrho hc}}{\varrho h} t$

By substituting:

$$\operatorname{tg}\phi = \frac{c - \varrho h\alpha^2}{2D\alpha}$$

The equation can be written as:

$$w = \frac{2\alpha q_0 e^{-\frac{D}{\rho h}t}[(2D^2 - (c - \varrho h\alpha^2)\varrho h)\sinh u + (2D\sqrt{D^2 - \varrho hc})\cosh u]}{2\sqrt{D^2 - \varrho hc}[(c - \varrho h\alpha^2)^2 + (2D\alpha)^2]} +$$

$$+ \frac{\varrho hg}{c} - \frac{q_0\cos(\alpha t + \phi)}{\sqrt{(c - \varrho h\alpha^2)^2 + 4D^2\alpha^2}}$$

The first term in the equation is an ignition term which eventually damps out. This term is neglected when its value is 1%, or less, of its initial value, that is:

$$e^{-\frac{D}{\rho h}t_1} < 0{,}01 e^{-\frac{D}{\rho h}t_0} = 0{,}01 \qquad (t_0 = 0)$$

$$t_1 > \frac{4{,}61\varrho h}{D}$$

Since, in many cases $D \gg \rho h$ zal t_1 is very small, the ignition term quickly damps out. As a dimensioning criterion it is assumed that the plate must not be lifted ($w<0$). Since $D^2 > \rho hc$ first term will have a positive effect on deflection and, for this reason the ignition term can be neglected.

Thus:

$$w = \frac{\varrho hg}{c} - \frac{q_0\cos(\alpha t + \phi)}{\sqrt{(c - \varrho h\alpha^2)^2 + 4D^2\alpha^2}}$$

With the dimensioning criteria $w \geq 0$ it follows that:

$$\frac{\varrho hg}{c} \geqslant \frac{q_0\cos(\alpha t + \phi)}{\sqrt{(c - \varrho h\alpha^2)^2 + 4D^2\alpha^2}}$$

that is:

$$\frac{\varrho hg}{c} \geqslant \frac{q_0}{\sqrt{(c - \varrho h\alpha^2)^2 + 4D^2\alpha^2}}$$

The term $\rho h\alpha^2$ can often be neglected with respect to c whereby:

$$h \geqslant \frac{q_0}{\varrho g \sqrt{1 + \left(\frac{2D\alpha}{c}\right)^2}}$$

q_0 is the loading on the plate due to wave action

$$q_0 = \frac{\varrho_w g H}{2} \frac{1}{\cosh(\lambda d)}$$

in which:

d = water depth (m)
λ = $2\pi/L$
L = wave length (m)
H = wave height (m)

From this it follows that:

$$h \geqslant \frac{H\varrho_w}{2\varrho_a \sqrt{1 + \left(\frac{2D\alpha}{c}\right)^2}} \frac{1}{\cosh(\lambda d)}$$

As stated above damping is caused by the resistance of the pore water flow. The asphalt plate can be displaced a vertical distance Δw if a quantity of water V flows in or out (Figure 4).

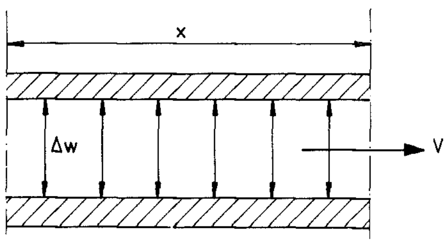

Figure 4 Displacement of a plate as a result of water flowing in the subsoil.

$$V = Q\Delta t$$

in which:

Q = inflow/outflow of water (m^2/s)
Δt = time period over which the plate is displaced
V = $\Delta w x$ (m^2)
x = length of plate lifted (m)

or:

$$Q\Delta t = \Delta w x$$

$$Q \quad = x\frac{\Delta w}{\Delta t}$$

The edge of the plate will remain in contact with the subsoil over a length R (Figure 5). This length R can be related to the length of penetration of the pore water movement resulting from the wave pressure under the plate (79).

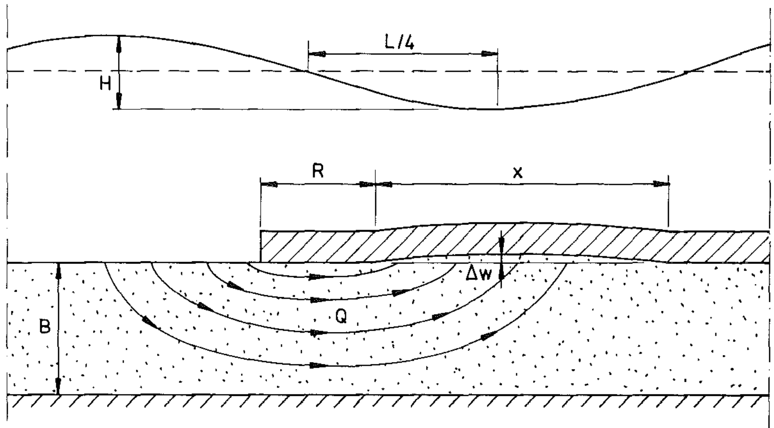

Figure 5 Lifting of the bottom protection.

If the revetment is lifted over a distance x then water must flow in.
The groundwater flow can be estimated using (42, 43):

$$\frac{Q}{kH'} = \frac{K'(b)}{2K(b)}$$

$$K(b) = \int_0^{\pi/2} \frac{d\theta}{\sqrt{1-b^2\sin^2\theta}}$$

$$K'(b) = \int_0^{\pi/2} \frac{d\theta}{\sqrt{1-(1-b^2)\sin^2\theta}}$$

$$b = \text{tgh}\left(\frac{\pi R}{4B}\right)$$

in which:

B = thickness of waterbearing layer (m)
k = permeability of the subsoil (m/s)
H' = wave pressure difference on the particular part of the plate (m)
H' = $H/\cosh(\lambda d)$

or:

$$Q = kH' \frac{K'(b)}{2K(b)}$$

by substitution in:

$$Q = x\frac{\Delta w}{\Delta t}$$

is obtained:

$$H' = \frac{2K}{K'}\frac{x}{k}\frac{\Delta w}{\Delta t}$$

The quantity $\gamma_w H'$ is the driving force fot the groundwater flow and, thus, also for the displacement Δw:

$$\gamma_w H' = \frac{2K}{K'}\frac{x}{k}\gamma_w\frac{\Delta w}{\Delta t}$$

The factor:

$$\frac{2K}{K'}\frac{x}{k}\gamma_w$$

thus represents, as it were, the damping factor due to pore water movement.

$$2D = \frac{2K}{K'}\frac{\gamma_w}{k}x$$

In this x is the length over which the plate is lifted. If the plate is stiff x is large; if the plate is more flexible it will, more or less, follow the wave action and will tend to $x \Rightarrow L/2$ (L = wavelength).

The distance R can be determined using:

$$R = -L \ln f/2\pi(\tfrac{1}{2}+\tfrac{1}{2}\sqrt{1+a^2})^{-1/2}; \quad a = L^2/(2\pi c_v T) \quad [79]$$

in which:

f = fraction of the wave penetration effect which is noticeable at a distance, l, from the edge of the plate
T = wave period (s)
c_v = subsoil consolidation coefficient (m^2/s)

In practice if: $f = 0{,}1$ en $a \gg 1: R = 1{,}3\sqrt{c_v T}$; for $a \ll 1: R = 0{,}366 \cdot L$

Calculations for some typical site conditions are carried out below.

Assumptions:

Wave periods:	T	= 6, 8, 10 and 12 s
Water depths:	d	= 2, 5, 10, 20 and 40 m
Subsoil permeability:	k	= 10^{-4}, 10^{-5} and 10^{-6} m/s

	Modulus of compression K_s (N/m²)	Shear modulus G (N/m²)
loose sand	8.5×10^6	3.5×10^6
medium compacted sand	25×10^6	11×10^6
density compacted sand	50×10^6	24×10^6

The wavelength is derived from the wave periods and water depths:

	water depth d (m)				
wave length (m)	2	5	10	20	40
$T =$ 6 s	27	38	48	56	56
$T =$ 8 s	36	53	71	89	100
$T =$ 10 s	45	71	96	120	156
$T =$ 12 s	54	85	120	152	192

In order to calculate R the consolidation coefficient of the subsoil c_v must be known.

$$c_v = \frac{k}{m_v \gamma_w}$$

in this:

m_v = related compression coefficient

$$m_v = \frac{1}{(K_s + \frac{4}{3}G)}$$

Thus:

	permeability		
consolidation coefficient	10^{-4}	10^{-5}	10^{-4}
loose sand	0.13	0.013	0.0013
medium compacted sand	0.40	0.040	0.0040
dense compacted sand	0.82	0.082	0.0082

with: $\gamma_w = 10^4$ N/m³

a en R can then be calculated for all these combinations of conditions (for f = 0,1). Since this is very involved only a few extreme values have been estimated.

permeability (m/s) period T (s)		loose sand				densely packed sand			
		waterdepth (m)							
		2		40		2		40	
		a	R	a	R	a	R	a	R
$k = 10^{-4}$	$T=6$	149	1.15	640	1.15	24	2.9	100	2.9
	$T=12$	297	1.62	3.8×10^3	1.62	47	4.1	595	4.1
	$T=6$	1.49×10^4	0.11	6.4×10^4	0.11	2.4×10^3	0.29	1.01×10^4	0.29
$k = 10^{-6}$	$T=12$	2.97×10^4	0.16	3.8×10^5	0.16	4.7×10^3	0.41	5.96×10^4	0.41

R/B is plotted against K/K' in Figure 6.

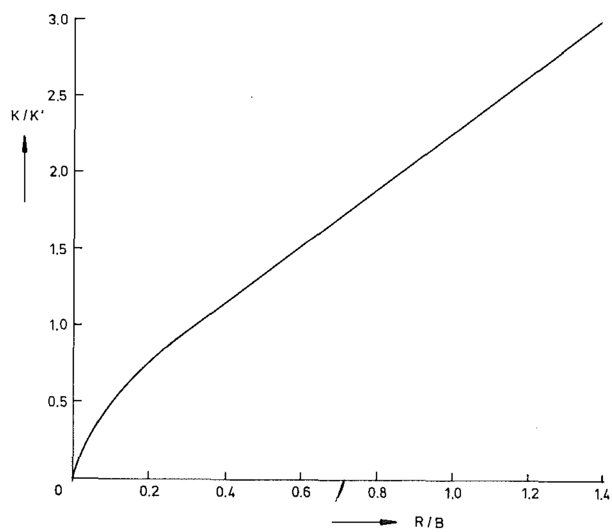

Figure 6 Values of K/K' plotted against R/B, based on Figure 11h (79).

The damping $2D$ has been calculated below for the variables given in the above tables for values of B equal to 10 and 100 m.
The uplifted length, x, is taken as $L/2$:

$$2D = \frac{2K}{K'} \frac{\gamma_w}{k} \frac{L}{2}$$

The maximum wave height in a water depth of 2 m is about 1.60 m and in a depth of 40 m, 31 m (breaking index = 0.78 m).

Damping, 2D (Ns/m^3)		loose sand				densely packed sand			
		waterdepth (m)							
		2		40		2		40	
		width of loaded surface (m)							
B (m) →		10	100	10	100	10	100	10	100
$k = 10^{-4}$ (m/s)	$T = 6$ s	1.6×10^9	2.7×10^8	3.4×10^9	5.6×10^8	3.1×10^9	5.4×10^8	6.4×10^9	1.1×10^9
	$T = 12$ s	3.5×10^9	4.5×10^8	1.2×10^{10}	1.5×10^9	6.2×10^{11}	1.4×10^{11}	2.2×10^{12}	4.8×10^{11}
$k = 10^{-6}$ (m/s)	$T = 6$ s	1.3×10^8	1.1×10^7	2.8×10^8	2.2×10^7	3.2×10^{10}	2.2×10^9	6.7×10^{10}	4.5×10^9
	$T = 12$ s	2.7×10^8	2.7×10^7	9.6×10^8	9.6×10^7	1.4×10^{11}	1.4×10^{10}	4.8×10^{11}	4.8×10^{10}

The modulus of subgrade reaction, C, can be estimated very generally using:

$$c = \frac{E}{I_s B(1 - \upsilon^2)} \qquad \text{(see Appendix V)}$$

in which:

E = the subsoil modulus of elasticity = $3K_s 1 - 2\upsilon)$
B = width of the loaded surface (m)
I_s = form factor

The modulus of subgrade reaction has been calculated for the various variables using $B = L/2$, $I_s = 2.5$ and $\upsilon = 0.33$.

Thus:

Modulus of subgrade c (N/m^3)	loose sand	densely packed sand		
	waterdepth			
	2	40	2	40
$T = 6$ s	2.9×10^5	1.4×10^5	1.7×10^8	8.2×10^5
$T = 12$ s	1.4×10^5	4.1×10^4	8.5×10^5	2.4×10^5

By using the values in the formula for determining the layer thickness, h extremely low values are obtained. The damping is very large related to the bedding constant.
The conclusion from this is that the uplift criterium, in most cases, is not a determining factor for the layer thickness.

Remarks

1. In the derivation of the formula for uplift plate bending has been neglected. Bending will resist deflection.
2. Inflow of water at the edges of a bed protection, when this has been lifted, will tend to lead to greater displacements. There is, however, no indication that the edges will be lifted because there the pressure differences due to wave penetration into the bed will be very small.

3. Local instability, related to scouring at the edges, can be produced by wave action. At sites where the sand is unstable under wave action, it is recommended that the watertight bed protection at the edges is replaced by a sandtight bed protection.
   This does not apply to a mastic slab at the toe of a dike, since this is used to protect the toe from scouring.

Appendix III

Designing an impermeable asphalt dike revetment against hydraulic uplift pressure

III.1 Designing an impermeable asphalt dike revetment against hydraulic uplift pressure

The maximum value of the hydraulic uplift pressure, σ_{w0}, which acts at the outside water-level, is assumed to be variable and should be determined by the designer.

The system of forces which acts on a revetment is shown in Figure 1 (53, 54).
For general equilibrium:

$$N_2 - W - K - F = 0$$

1. If the frictional resistance, W, is equivalent to the component of the weight of the revetment down the slope, N_2, the equilibrium equation becomes:

   $N_2 = W$ and $K = 0$; $F = 0$

2. If N_2 is larger than W then the revetment will hang from the upper section and rest on the lower section, where $W \geq N_2$. In this situation an internal tensile force, K, will develop in the revetment.

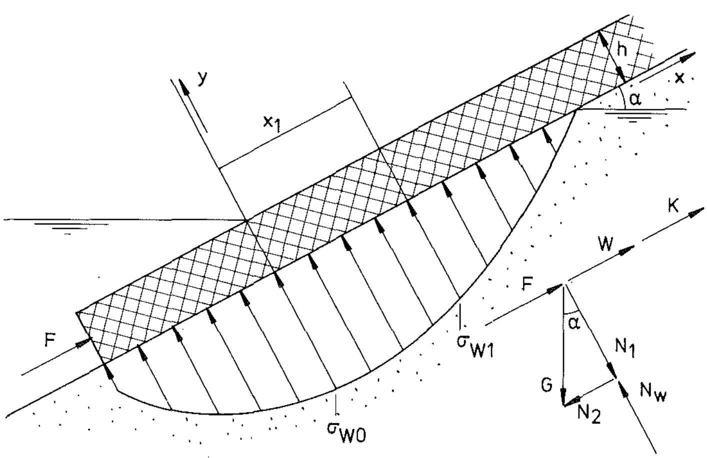

Figure 1 System of forces acting on a revetment.

G = the unit weight of the revetment (N/m)
N_1 = the weight component of the revetment vertical to the slope (N/m)
N_2 = the weight component of the revetment parallel to the slope (N/m)
W = frictional resistance (N/m)
N_w = upward force due to hydraulic uplift pressure (N/m)
K = internal tensile force in the revetment (N/m)
h = revetment thickness (m)
a = slope angle
F = force due to a support, for example, a toe construction, or water pressure (N/m)

III.1.1 *The sliding criterium*

This criterium relates to the opposing tensile forces and pressures in the revetment. (The construction does not bend and there are no moments or shear forces.
Nowhere under the revetment must the friction forces be exceeded.
The equilibrium equation becomes:

$$N_2 = W$$

$$W = fN_k$$

f = coefficient of friction
f = $\tan\theta$ if $\theta < \phi$
f = $\tan\phi$ if $\theta \geq \phi$
θ = angle of friction between the revetment and the subsoil
ϕ = angle of internal friction of the subsoil
N_k = force exerted by the subsoil grains on the revetment (N/m)

Consider a small element Δx in the plate, see Figure 2.

$$N_k = N_1 - N_w = \rho_a gh \cos \alpha \Delta x - \sigma_{wl} \Delta x$$

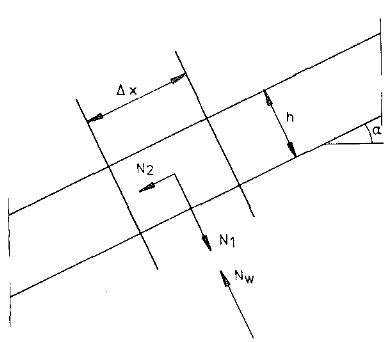

Figure 2 Forces acting on a small element revetment.

in which:

ρ_a = density of the revetment material (kg/m^3)
g = acceleration due to gravity (m/s^2)
h = thickness of the revetment (m)
σ_{wl} = hydraulic uplift pressure at $x = x_l$ (N/m^2)
N_2 = $\rho_a g h \sin \alpha \Delta x$ (N/m)

The small element will not slide if:

$$N_2 \leqslant W$$

that is:

$$\varrho_a gh \sin \alpha \Delta x \leqslant f(\varrho_a gh \cos \alpha \Delta x - \Delta x \sigma_{wl})$$

$$h \geqslant \frac{\sigma_{wl} f}{\varrho_a g (f \cos \alpha - \sin \alpha)}$$

at the place of the maximum uplift pressure ($x = 0$)

$$h \geqslant \frac{\sigma_{w0} f}{\varrho_a g (f \cos \alpha - \sin \alpha)}$$

From the formula it follows also that, to guarantee the general stability of the construction, the part of the revetment under which there is groundwater, and also when there is no difference in water level between must lie on a slope with an angle smaller than a certain critical value, see also Section III.3.

III.1.2 *The buoyancy criterium*

The basis of the buoyancy criterium is to prevent the revetment from lifting. In other words no forces should be allowed to develop which can push up the revetment.
This can only be achieved if the maximum water pressure does not exceed the revetment weight component vertical to the slope:
That is:

$$\varrho_a gh \cos \alpha \geqslant \sigma_{w0}$$

$$h \geqslant \frac{\sigma_{w0}}{\varrho_a g \cos \alpha} \qquad (\text{at } x = 0)$$

Since here the sliding criterium is not satisfied stresses and strains develop in the material. If the revetment is not supported, for example, by a toe construction, it will hang from the section above where $W \geq N_2$.
Tensile forces develop in the material which must not exceed the maximum allowable value.

The tension force which develops is given by: $K = N_2 - W - F$.

where $F = \varrho_w g h \cos \alpha$

(supporting force by water)

If it is assumed that the tensile force σ_t, is uniformly distributed over the cross-section of the plate:

$$\sigma_t = \frac{F}{h} \leqslant \sigma_{t\,max}$$

with $\sigma_{t\,max}$ the allowable tension.

III.2 Application of an electrical analogue

The variation of water pressure under a relatively impermeable revetment can be determined using an electrical analogue.
With an electrical analogue, differences in electric potententials can be determined. These potential differences are related to the differences in hydraulic head between the groundwater and the free water on the revetment. The differences in hydraulic head in the groundwater between the top and the bottom of the revetment can also be determined. These hydraulic heads which, eventually, cause the uplift pressures are described in meter water column, related to the surface of the revetment, see Figure 3.

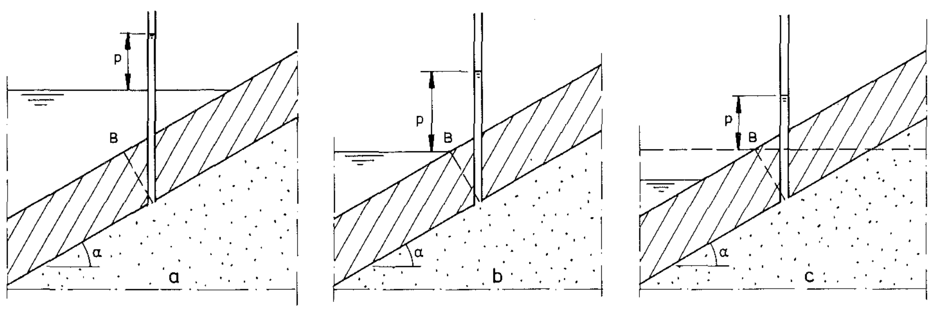

Figure 3 The potential differences for different exterior water levels (52).

The following relationship is valid:

$$\sigma_w = \varrho_w g (p + h \cos \alpha)$$

The revetment thickness required at several points on the slope, must be determined from the maximum potential difference which can develop at each point. Becuase the maximum potential difference develops at each point at different moments in time, the envelope of the maximum potential differences is usually plotted, see Figure 4.
This subject is discussed further in (52, 53, 85).

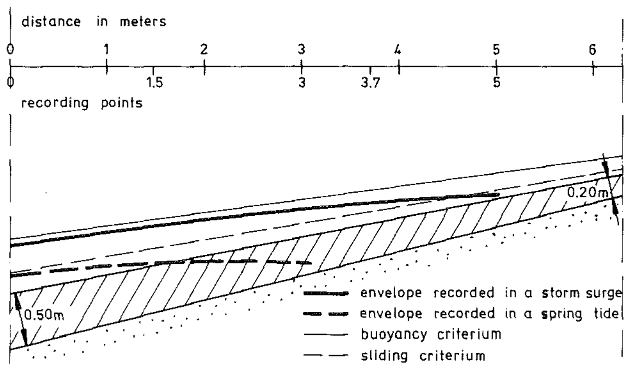

Figure 4 Envelope of maximum potential differences for different conditions (52).

III.3 Slope stability

Groundwater in the dike body cannot flow out through a sealed revetment. It is assumed that the water is forced to flow parallel to and below the revetment, see Figure 5 (55).

The equilibrium equations for a small element, dx, are:

– vertical to the slope:

$$\gamma_a h \, dx \cos\alpha + \gamma_n y \, dx \cos\alpha = N_k \, dx + N_w \, dx$$

– parallel to the slope:

$$\gamma_a h \, dx \sin\alpha + \gamma_n y \, dx \sin\alpha + \gamma_w iy \, dx = W \, dx + H_w \, dx$$

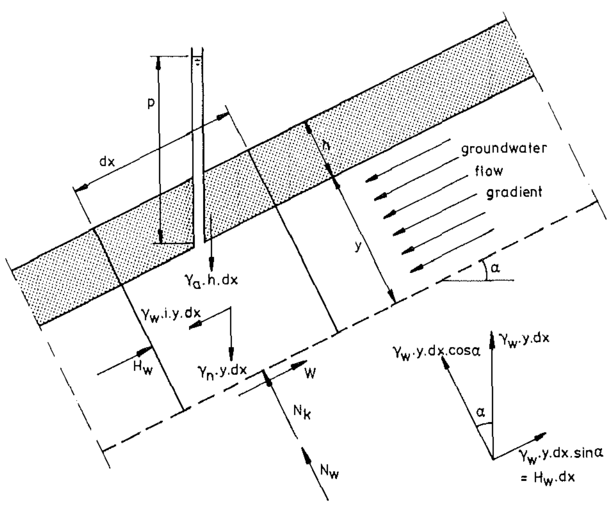

Figure 5 Groundwater flow under the revetment, parallel to the slope.

in which:

γ_a = specific mass of the revetment (N/m^3)
h = thickness of the revetment (m)
dx = width of small element (m)
α = slope angle
γ_n = specific mass of wet subsoil (N/m^3)
y = thickness of subsoil element (m)
N_k = grain stress in subsoil (N/m^2)
N_w = resultant upwards hydraulic pressure vertical to the slope
H_w = resultant upwards hydraulic pressure parallel to the slope
i = gradient
W = sliding force (N/m^2)

To prevent sliding along the line:

$$W \leq N_k \tan \phi$$

ϕ = the angle of internal friction of the subsoil

Also:

$N_w = (p + y \cos \alpha) \gamma_w$
p = hydraulic head at $y = 0$ related to the lower surface of the revetment. This is not the same as in Section III.2 where p was measured from the upper surface of the revetment.

Thus:

$$\gamma_a h \, dx \cos \alpha + \gamma_n y \, dx \cos \alpha = N_k \, dx + (p\gamma_w + \gamma_w y \cos \alpha) \, dx$$
$$N_k = (\gamma_a h + \gamma_n y - \gamma_w y) \cos \alpha - p\gamma_w$$

In addition:

$$(\gamma_a h + \gamma_n y - \gamma_w y) \sin \alpha + \gamma_w i y = W \leqslant N_k \operatorname{tg} \phi$$

Thus:

$$(\gamma_a h + \gamma_n y - \gamma_w y) \sin \alpha + \gamma_w i y \leqslant \{(\gamma_a h + \gamma_n y - \gamma_w y) \cos \alpha - p\gamma_w\} \operatorname{tg} \phi$$

$$\operatorname{tg} \phi \geqslant \frac{(\gamma_a h + \gamma_n y - \gamma_w y) \sin \alpha + \gamma_w i y}{(\gamma_a h + \gamma_n y - \gamma_w y) \cos \alpha - p\gamma_w}$$

Putting: $\gamma_n - \gamma_w = \gamma_0$

$$\operatorname{tg} \phi \geqslant \frac{(\gamma_a h + \gamma_0 y) \operatorname{tg} \alpha + \gamma_w y \dfrac{i}{\cos \alpha}}{\gamma_a h + \gamma_0 y - \dfrac{p\gamma_w}{\cos \alpha}}$$

For $y = 0$:

$$\operatorname{tg} \phi \geqslant \frac{\gamma_a h \operatorname{tg} \alpha}{\gamma_a h - \frac{p\gamma_w}{\cos \alpha}} = \operatorname{tg} \alpha \left[1 + \frac{\frac{p\gamma_w}{\gamma_a h \cos \alpha}}{1 - \frac{p\gamma_w}{\gamma_a h \cos \alpha}} \right]$$

for $y \to \infty$:

$$\operatorname{tg} \phi \geqslant \frac{\gamma_0 \operatorname{tg} \alpha + \gamma_w \frac{i}{\cos \alpha}}{\gamma_0} = \operatorname{tg} \alpha \left[1 + \frac{\gamma_w}{\gamma_0} \frac{i}{\sin \alpha} \right]$$

Which is the determining condition?

$$\frac{\frac{p\gamma_w}{\gamma_a h \cos \alpha}}{1 - \frac{p\gamma_w}{\gamma_a h \cos \alpha}} \overset{?}{><} \frac{\gamma_w}{\gamma_0} \frac{i}{\sin \alpha}$$

$$\frac{p\gamma_w}{\gamma_a h \cos \alpha} \overset{?}{><} \frac{\gamma_w}{\gamma_0} \frac{i}{\sin \alpha} \left[1 - \frac{p\gamma_w}{\gamma_a h \cos \alpha} \right]$$

$$\frac{p\gamma_w}{\gamma_a h \cos \alpha} \left[1 + \frac{\gamma_w}{\gamma_0} \frac{i}{\sin \alpha} \right] \overset{?}{><} \frac{\gamma_w i}{\gamma_0 \sin \alpha}$$

The strongest condition is:

$$\operatorname{tg} \phi \geqslant \operatorname{tg} \alpha \left[1 + \frac{\gamma_w}{\gamma_0} \frac{i}{\sin \alpha} \right]$$

if:

$$\frac{p\gamma_w}{\gamma_a h \cos \alpha} \left[1 + \frac{\gamma_w}{\gamma_0} \frac{i}{\sin \alpha} \right] < \frac{\gamma_w}{\gamma_0} \frac{i}{\sin \alpha}$$

that is:

$$p < \frac{\frac{\gamma_a}{\gamma_0} hi \frac{\cos \alpha}{\sin \alpha}}{1 + \frac{\gamma_w}{\gamma_0} \frac{i}{\sin \alpha}}$$

The maximum value of the gradient, i, (see Figure 6) is:

$$i = \sin \alpha - \frac{p \cos \alpha}{l}$$

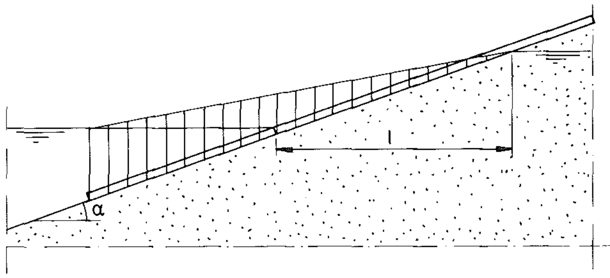

Figure 6 Hydraulic uplift pressure under an impermeable asphalt revetment.

By substitution in:

$$\operatorname{tg} \phi \geqslant \operatorname{tg} \alpha \left[1 + \frac{\gamma_w}{\gamma_0} \frac{i}{\sin \alpha}\right]$$

it follows that:

$$\operatorname{tg} \phi \geqslant \operatorname{tg} \alpha \left[1 + \frac{\gamma_w}{\gamma_0} \frac{\sin \alpha - \dfrac{p \cos \alpha}{l}}{\sin \alpha}\right]$$

The most unfavourable situation develops when $p/l = 0$ or $p = 0$; $l \neq 0$ than $i = \sin \alpha$. Thus:

$$\operatorname{tg} \phi \geqslant \operatorname{tg} \alpha \left[1 + \frac{\gamma_w}{\gamma_0}\right]$$

That is:

$$\operatorname{tg} \alpha \leqslant \operatorname{tg} \phi \left[1 - \frac{\gamma_w}{\gamma_n}\right]$$

III.4 A revetment acting as a plate

Another criterium which can be applied is that the revetment is allowed to lift by hydraulic uplift pressures which only occur very rarely. As a result of this uplift tensile forces, moments and shear forces develop in the revetment which must be absorbed by its bending resistance. Figure 7 shows how hydraulic uplift pressures act on the revetment.

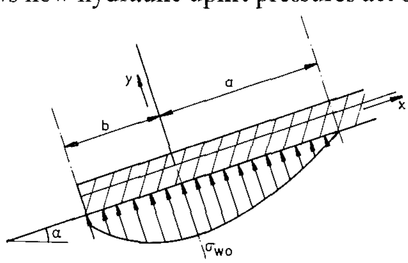

Figure 7 Hydraulic uplift pressure under an impermeable asphalt revetment.

It is assumed that displacements are so small that the tensile forces resulting from bending moments can be neglected.

Figure 8 shows a schematised loading situation. Since the revetment is lifted locally, it will be partially yes or no elastic supported by the subsoil. (It is assumed that the loading duration is so long that time effects can be neglected). It is assumed, in the uplift, that there is no elastic support from the subsoil.

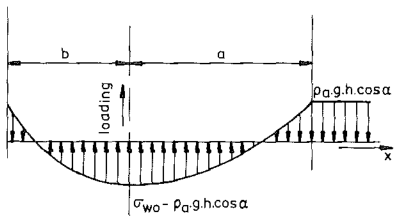

Figure 8 Schematised loading situation on the revetment.

For an elastically supported slab the following is valid:

$$K \frac{\partial^4 w}{\partial x^4} + cw = q(x) \qquad \text{(fig. 9)}$$

in which:

K = bending stiffness of the slab (N/m^2)
w = deflection (m)
x = horizontal plate vector (m)
c = modulus of subgrade reaction (N/m^3)
q = pressure on the slab (N/m^2)

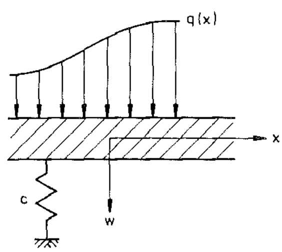

Figure 9 An elastically supported plate.

For $w >$ c = constant
For $w <$ c = 0; so that

$$K \frac{\partial^4 w}{\partial x^4} = q(x)$$

The above differential equation must be solved in order to determine the deflection of the revetment.

The following relationships are also valid:

– angular distortion $\phi = \frac{\partial w}{\partial x}$

– moments $M = -K \frac{\partial^2 w}{\partial x^2}$

– shear forces $D = -K \frac{\partial^3 w}{\partial x^3}$

The boundary conditions are:

1. $x = -b$: $M = 0$; $D = 0$
2. If the revetment has a length: at $x = l - b : M = 0$; $D = 0$
   If the revetment is 'infitely' long: at $x \rightarrow \infty : w = 0$; $\phi = 0$

At points where $w = 0$ the equations have to be linked together.
In this case:

$$w_1 = w_2;\ \phi_1 = \phi_2;\ M_1 = M_2;\ D_1 = D_2$$

The solution of the equations is very protracted and a numerical approach is more suitable.
Investigation into the applicability (the desirability of uplifting) is advisable.

Appendix IV

The effects of settlement and scouring

IV.1 Settlement

If irregular settlement occurs under an asphalt revetment, because if its viscous properties, it will try to adjust. A strongly schematized approximation is given below which can be used to determine the amount of irregular settlement which can be allowed. This should not be seen as definitive but more as an example of a calculation.

It is assumed that the settlement under the asphalt slab is circular in plan. The velocity of the settlement is assumed to be constant at v m/s vertically and u m/s horizontally, see Figure 1.

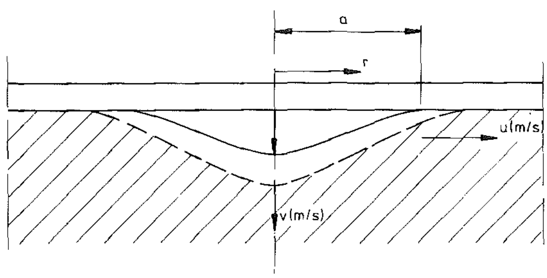

Figure 1 Settlement under an asphalt slab.

The revetment will bend under its own weight. The bending of a circular area, fixed around its circumference and loaded under its own weight is given by:

$$w = \frac{q}{64K}(r^2 - a^2)^2$$

$$K = \frac{Eh^3}{12(1-\upsilon^2)}$$

in which:

q = uniformly distributed load due to its own weight (N/m²)
q = $\rho_a g h$
w = maximum displacement
r = see Figure 1 (m)
a = see Figure 1 (m)
E = stiffness modulus of the revetment material (N/m²)

h = thickness of the revetment (m)
υ = poissons ratio for the slab material
ρ_a = density of the revetment material (kg/m^3)
g = acceleration due to gravity (m/s^2)

at $r = 0$:

$$w = \frac{12q(1-\upsilon^2)a^4}{64Eh^3}$$

The slab will try to adjust to the settlement. If the rate of settlement is not too fast or if the slab does not break before, it will, after some time, reach a position where it is, once again, supported by the subsoil.
Assuming that the amount of settlement at $r = 0$ is z, the slab will adjust to the subsoil if:

$$w \geqslant z$$

that is:

$$\frac{3}{16}\frac{\varrho_a g(1-\upsilon^2)a^4}{Eh^2} \geqslant z$$

with $a = ut$ and $z = vt$ (t = time; the time at which the settling begins is $t = 0$) it follows that:

$$\frac{3}{16}\frac{\varrho_a g(1-\upsilon^2)u^4t^4}{Eh^2} \geqslant vt$$

This can also be written as

$$\frac{S(t,T)}{t^3} \leqslant \frac{3\varrho_a g(1-\upsilon^2)u^4}{16vh^2}$$

$S(t, T) = E$ = the stiffness modulus of the asphalt, which is time and temperature dependent.
The value of t, to estimate $S(t, T)$, can be taken, for example, at about half of the duration of the total settlement.

The time taken to adjust to the subsoil can be found using the formula:

$$\frac{S(t,T)}{t^3} = \frac{3\varrho_a g(1-\upsilon^2)u^4}{16vh^2}$$

Before the revetment has adjusted completely onto the subsoil it will hang 'free' and bending stresses can develop which should not exceed the maximum allowable stress.

On the horizontal x-axis the moment in the unsupported slab is given by:

$$M_{xx} = -\frac{\varrho_a g h}{16}\{(3+\upsilon)x^2 - (1+\upsilon)a^2\}$$

$$M_{xy} = 0$$

$$M_{yy} = -\frac{\varrho_a g h}{16}\{(1+3\upsilon)x^2 - (1+\upsilon)a^2\}$$

at $x = 0$:

$$M_{xx} = \frac{\varrho_a g h}{16}(1+\upsilon)a^2$$

$$M_{yy} = \frac{\varrho_a g h}{16}(1+\upsilon)a^2$$

at $x = a$:

$$M_{xx} = -\frac{\varrho_a g h}{8}a^2$$

$$M_{yy} = -\frac{\varrho_a g h}{8}\upsilon a^2$$

The maximum moment is:

$$M = \frac{\varrho_a g h}{8}a^2$$

with $a = ut$ this becomes:

$$M_{max} = \frac{\varrho_a g h}{8}u^2 t^2$$

The maximum moment occurs after the maximum time t, just before the revetment comes to rest again on the subsoil. The maximum bending stress the revetment is given by:

$$\sigma_b = \frac{6M}{h^2} = \frac{3\varrho_a g u^2 t^2}{4h} \leqslant \sigma_{b\ \text{allowed}}$$

A wave impact can also cause settlement. Because of its elasticity, initially the revetment will recover to its original position; the subsoil, however, will not, or, at least, only partly recover. (The deflection, resulting from wave impacts, can be determined using Appendix I.) The revetment should, preferably, be able to settle onto the subsoil in the period between two wave impacts. The bending of an unsupported revetment, fixed at its circumference and loaded by its own weight, is given by:

$$w_{x=0} = \frac{1}{384}\frac{ql^4}{EI} = \frac{1}{32}\frac{ql^4}{Eh^3}$$

In order for the slab to settle in time (i.e. between two wave impacts), the following must be valid:

$$S(t,T) < \frac{1}{32}\frac{ql^4}{h^3 w} = \frac{\varrho_a g l^4}{h^2 w}$$

in which:

S = E is the stiffness modulus of the asphalt (N/m^2)
w = deflection due the wave impact (m)
l = length of the deflection (Appendix I) (m)

If the revetment does not settle in time, it will be loaded, while unsupported, by the following wave impact.
The bending stresses are given by:

$$M_{\text{rand}} = \tfrac{1}{8}Pl + \tfrac{1}{12}\varrho_a g h l^2$$

$$\sigma_b = \frac{6}{h^2}\left(\tfrac{1}{8}Pl + \tfrac{1}{12}\varrho_a g h l^2\right) \leqslant \sigma_{b \text{ allowed}}$$

IV.2 Scouring

The displacement of the end of an unsupported cantilevered slab is given by:

$$w = \frac{\varrho_a g h l^4 12(1-\upsilon^2)}{8Eh^3}$$

Let the scouring proceed at a horizontal speed, v m/s, see Figure 2:

$$l(t) = vt$$

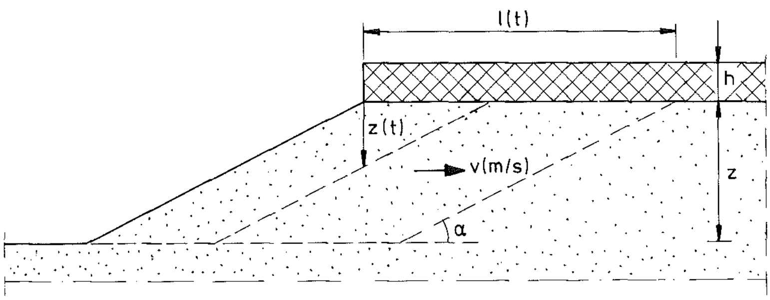

Figure 2 Scouring of an asphalt slab.

The slab will try to adjust to the scouring and, if the speed of undermining is not too great or if the slab does not brake, it will, after a certain time, settle again onto the subsoil. For this to happen:

$$w \geqslant z(t)$$

That is: $$\frac{\varrho_a g l^4 12(1-\upsilon^2)}{8Eh^2} \geqslant z(t)$$

The horizontal scouring speed is v m/s; the vertical speed v tan α m/s (α = slope of scoured subsoil).

Now $z(t) = vt \tan\alpha$ up to a maximum value of $z(t) = z$.
Thus for:

$$t \leqslant \frac{z}{v \operatorname{tg} \alpha} : \frac{S(t,T)}{t^3} \leqslant \frac{3\varrho_a g(1-\upsilon^2)v^3}{2h^2 \operatorname{tg} \alpha}$$

$$t > \frac{z}{v \operatorname{tg} \alpha} : \frac{S(t,T)}{t^4} \leqslant \frac{3\varrho_a g(1-\upsilon)^2 v^4}{2h^2 z}$$

Using these formulas the time taken to settle on to the subsoil can be determined.
Before settling bending stresses develop in the unsupported slab which should not exceed the maximum allowable values.
The maximum moment is given by:

$$M_{\max} = \tfrac{1}{2}\varrho_a g h l^2 = \tfrac{1}{2}\varrho_a g h v^2 t_1^2$$

where t_1, is the time taken for the slab to settle onto the subsoil

The maximum bending stress is:

$$\sigma_b = \frac{3\varrho_a g v^2 t_1^2}{h} \leqslant \sigma_{b \text{ allowed}}$$

With asphalt mastic the viscous component will predominate with larger loading durations. Then the modulus of stiffness can be replaced by $S = 3\eta/t$ in which η is the viscosity in Pa · sec. (5).

Appendix V

Boussinesq approach for monolithic constructions

The distribution of stresses caused by surface loads in an elastic, homogeneous, isotropic, semi-infinite mass were analysed for the first time by Boussinesq (45, 46, 47, 57).

V.1 A single layer system

The following situations can be considered when determining the stresses in a dike body (of earth and/or lean sand asphalt):

– The stress distribution under a line load P per unit length, see Figure 1, is:

$$\sigma_z = \frac{2P}{\pi} \frac{z^3}{(x^2+z^2)^2}$$

$$\sigma_x = \frac{2P}{\pi} \frac{x^2 z}{(x^2+z^2)^2}$$

$$\tau_{xz} = \frac{2P}{\pi} \frac{x z^2}{(x^2+z^2)^2}$$

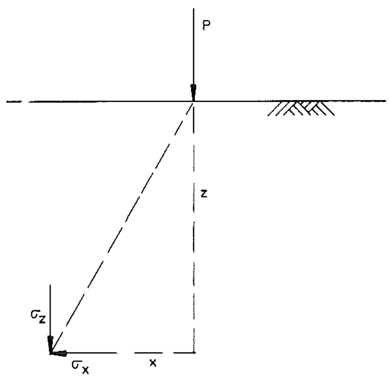

Figure 1 Stress distribution under a line load.

– The stress distribution under a strip with uniform loading, see Figure 2, is:

$$\sigma_z = \frac{q}{\pi} \{\alpha + \sin\alpha \cos(\alpha + 2\beta)\}$$

$$\sigma_x = \frac{q}{\pi} \{\alpha - \sin\alpha \cos(\alpha + 2\beta)\}$$

$$\tau_{xz} = \frac{q}{\pi} \{\sin\alpha \sin(\alpha + 2\beta)\}$$

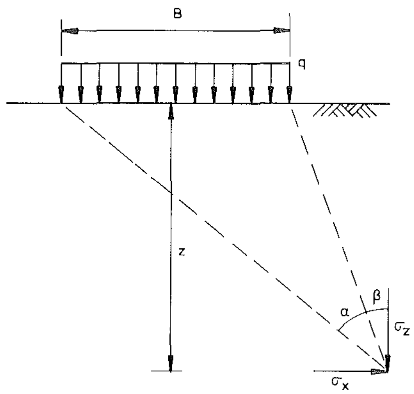

Figure 2 Strip with uniform loading.

– The vertical stress under a rectangular area (length L, breadth P) with a uniform loading, see Figure 3, is:

$$\sigma_z = \frac{q}{4\pi}\left[\frac{2mn(m^2+n^2+1)^{1/2}}{m^2+n^2+m^2n^2+1}\;\frac{m^2+n^2+2}{m^2+n^2+1} + \mathrm{tg}^{-1}\left\{\frac{2mn(m^2+n^2+1)}{m^2+n^2-m^2n^2+1}\right\}\right] = qI_\sigma$$

$$m = \frac{B}{z}; \qquad n = \frac{L}{z}$$

The value I_σ is given in Figure 4 for different values of m and n.

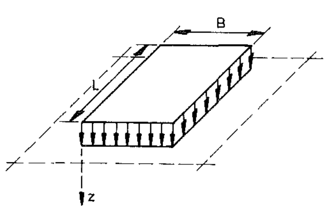

Figure 3 Rectangular area with uniform loading.

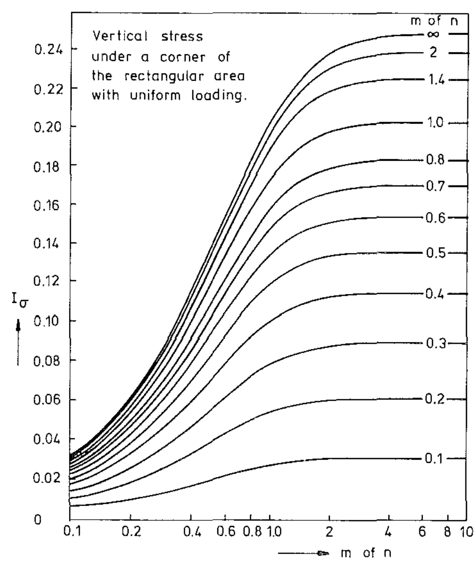

Figure 4 Values of I_σ for different values of m and n.

The volumetric strain of the subsoil can be determined using:

$$\frac{\Delta v}{v} = e = \frac{1-2\upsilon}{E}\,(\sigma_1 + \sigma_2 + \sigma_3)$$

The settlement of the surface, for various type of surface loading can be determined using the formula:

$$S_d = I_s q B \left(\frac{1-\upsilon^2}{E}\right)$$

in which:

S_d = settlement of the surface (m)
I_s = a coefficient related to the form of the surface referred to as the Form and Stiffness Factor
q = uniformly distributed load (N/m^2)
B = breadth of loading area (rectangular area) (m)
diameter of loading area (circular area (m)
υ = poissons ratio for the subsoil
E = elastic modulus of the subsoil

Various values of I_s are given in Table 1.

A subsoil constant which is often used in dimensioning models is, what is referred to as, the modulus of subgrade reaction: *c*.

Table 1 Form and stiffness factor, I_s, for calculating the settlement at points under the loaded *elastic areas of an semi-infinite mass*

form	middle	corner	middle short side	middle long side	mean
circle	1.00	0.64	0.64	0.64	0.85
circle (stiff)	0.79	0.79	0.79	0,79	0.79
square	1.12	0.56	0.76	0.76	0.95
square (stiff)	0.99	0.99	0.99	0.99	0.99
Rectangle L/B					
1.5	1.36	0.67	0.89	0.97	1.15
2	1.52	0.76	0.98	1.12	1.30
3	1.78	0.88	1.11	1.35	1.52
5	2.10	1.05	1.27	1.68	1.83
10	2.53	1.26	1.49	2.12	2.25
100	4.00	2.00	2.20	3.60	3.70
1000	5.47	2.75	2.94	5.03	5.15
10000	6.90	3.50	3.70	6.50	6.60

The modulus of subgrade reaction is, in fact, not a direct property of the subsoil but a parameter which varies from situation to situation. The modulus of subgrade reaction defines the relationship between the uniformly distributed loading q, and the settlement of the subsoil s:

$$c = \frac{q}{s}$$

From the expressions given earlier can be written as:

$$c = \frac{E}{I_s B(1-\upsilon^2)} \quad \text{for a uniformly distributed load}$$

Some principle values for subsoil parameters are given in Table 2.
The compression modulus $K = E/3(1-\upsilon^2)$, and the shear modulus $G = E/2(1+\upsilon)$.
The Poissons ratio, for cohesion less soils, lies approximately between 0.25 and 0.35 and, for soils with cohesion, between 0.35 and 0.40.

Table 2 Some principle values for the elastic properties of soil. (All values in MN/m^2).

Soil type	Compression modulus K	Shear modulus G
loose sand	8.5	3.5
medium sand	25.0	11.0
densely sand	50.0	24.0
weak clay	0.8	0.25
medium clay	2.5	1.0
strong clay	5.0	2.3
organic clay	1.0	0.2
weak peat	0.67	0.05
strong peat	1.33	0.10

In most cases the loading is not uniformly distributed and the subsoil parameters are somewhat different to those given in Table 2. It is, therefore, advisable to determine the modulus of subgrade reaction by (dynamic) loading tests, for example, a plate-bearing test or a C.B.R.-test.

V.2 Multilayer systems

Burmister developed expressions for the stresses and settlement of elastic two-layer systems subjected to uniform circular loading, see Figure 5. For the bending of the upper layer he gives the following formula:

$$s = \frac{1{,}5pa}{E_2} F_w$$

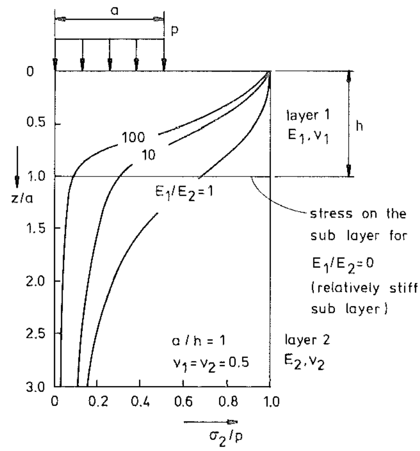

Figure 5 Stress variation in a two-layer system (Burmister).

Figure 6 gives some values of F_w, the displacement parameter.

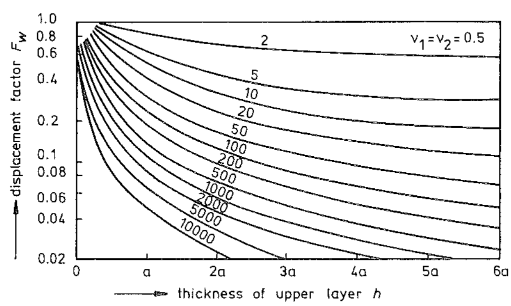

Figure 6 Theoretical displacements in an elastic two-layer system.

Palmer and Barber have also investigated the displacements in a two-layer system. They assumed that the relative stiffness of the upper layer to the subsoil is:

$$\left\{\frac{E_1(1-\upsilon_2^2)}{E_2(1-\upsilon_1^2)}\right\}^{1/3}$$

With this relationship the thickness of the upper layer h can be converted into an equivalent thickness of material of the subsoil h_e.

$$h_e = h\left[\frac{E_1(1-\upsilon_2^2)}{E_2(1-\upsilon_1^2)}\right]^{1/3}$$

The vertical displacement Δz, at a depth z, of a semi-infinite elastic subsoil is

$$\Delta z = \frac{1{,}5pa^2}{E(a^2+z^2)^{1/2}} \quad \text{for} \quad \upsilon = \tfrac{1}{2}$$

By converting E to E_2 and z to h_e the vertical displacement of the surface of the subsoil becomes:

$$\Delta s = \frac{1{,}5pa^2}{E_2\left[a^2+h^2\left(\frac{E_1}{E_2}\right)^{2/3}\right]^{1/2}} \qquad (\upsilon_1 = \upsilon_2 = 0{,}5)$$

The displacement of the upper layer is

$$\Delta p = \frac{E_2}{E_1}\left\{\frac{1{,}5pa}{E_2} - \Delta s\right\}$$

The total displacement of the upper surface is, therefore:

$$\Delta = \Delta p + \Delta s = \frac{1{,}5pa}{E_2}\left[\frac{a}{\left(a^2+h^2\left(\frac{E_1}{E_2}\right)^{2/3}\right)^{1/2}}\left(1-\frac{E_2}{E_1}\right)+\frac{E_2}{E_1}\right] = 1{,}5\,\frac{pa}{E_2}\,F'_w$$

Table 3 gives some values of F_w and F'_w.

Table 3 Comparison between the Burmister coefficient and the Palmer/Barber coefficient.

E_1/E_2	a/h	F_w	F'_w
10000	10	0.40	0.42
10000	5	0.22	0.23
10000	1	0.05	0.05
100	10	0.91	0.91
100	5	0.76	0.74
100	1	0.23	0.22
2	1	0.80	0.81

N.B.: It must be stressed that the above formulas are only valid for static loads.

Detailed calculation models are also available for determining the stresses and strains in two or multilayer systemd. An example of such a model is the Shell Bisar program.

Appendix VI

A soil mechanics model for calculating a slope constructed from lean sand asphalt

The conventional way to determine the stability of an earth slope is the slip circle analysis of Bishop. In this method it is assumed that the slope will slide along a slip circle if the maximum shear strength on the slip surface is exceeded by the loading due to the weight of earth above the surface and the external conditions, see Figure 1. In the Bishop method it is assumed that the resultant of all lamina forces on each lamina is horizontal. The horizontal equilibrium is not considered, so this resultant play no role.

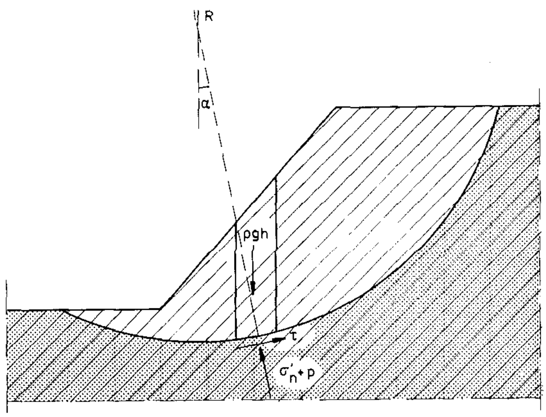

Figure 1 Slip surface according to Bishop.

Consider the following:

a. Moments equilibrium

$$\Sigma \varrho g h \sin \alpha = \Sigma \frac{\tau}{\cos \alpha}$$

b. Vertical equilibrium

$$\varrho g h = \sigma'_{\mathrm{n}} + p + \tau \frac{\sin \alpha}{\cos \alpha}$$

in which:

ϱ = density of the soil (kg/m^3)
h = height of the lamina (m)
σ'_n = soil particle stress (N/m^2)
p = water pressure (N/m^2)
τ = sliding resistance (N/m^2)
α = see Figure 1
g = acceleration due to gravity (m/s^2)

In contrast to soil when sliding develops the strength of lean sand asphalt increases due to its viscosity.
A modified slip circle analysis has been developed by Calle (58) for banks of lean sand asphalt which includes a relationship between the shear strength mobilised and the shear deformation which develops.
For sand:

$$\tau_s = f_s(\gamma)\sigma' \operatorname{tg} \phi_s$$

For lean sand asphalt:

$$\tau_a = f_a(\gamma)\sigma' \operatorname{tg} \phi_a + \eta\dot{\gamma}$$

In which:

τ = developed shear stress (N/m^2)
σ' = normal stress on the cross-section on which the shear stress acts (N/m^2)
ϕ = angle of internal friction
f = a mobilization factor, see Figure 2
η = viscosity (N/m^2)
γ = shear strain
$\dot{\gamma}$ = derivative of shear strain γ in time (l/s)
s = sand index
a = lean sand asphalt index

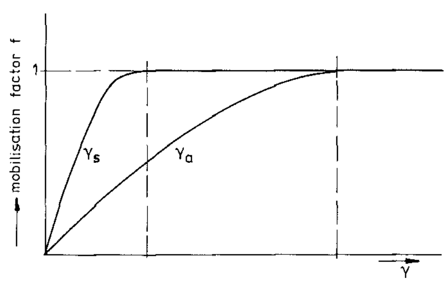

Figure 2 Mobilization factor.

The equilibrium method, according to Bishop, has been developed for a bank of lean sand asphalt, see Figure 3.

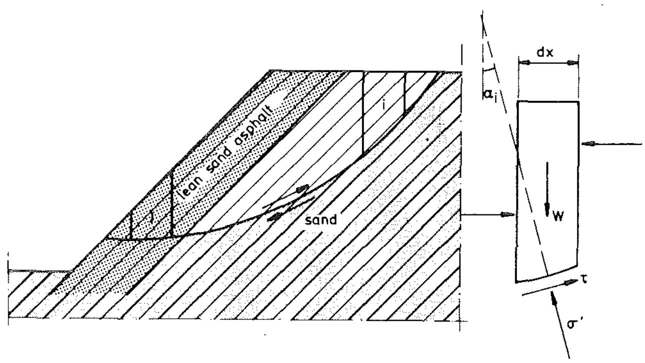

Figure 3 A bank of lean sand asphalt.

For a sand lamina:

Resolving vertically

$$w_i = \sigma_i' f_s(\gamma) \text{ tg } \phi_s \sin \alpha \frac{dx_i}{\cos \alpha_i} + (\sigma_i' + p_i)\, dx_i$$

w_i = weight of lamina

dx_i = breadth of lamina

The contribution of a lamina to the resisting moment is:

$$M_i^R(\gamma) = R\tau \frac{dx_i}{\cos \alpha_i} = R \frac{(w_i - p_i\, dx_i)}{\cos \alpha_i + f_s \text{ tg } \phi_s \sin \alpha_i} f_s(\gamma) \text{ tg } \phi_s$$

R = slip circle radius

For a lean sand asphalt lamina:

Resolving vertically

$$w_j = \sigma_j' f_a(\gamma) \text{ tg } \phi_a \sin \alpha_j \frac{dx_j}{\cos \alpha_j} + \eta \dot{\gamma} \sin \alpha_j \frac{dx_j}{\cos \alpha_j} + (\sigma_j' + p_j)\, dx_j$$

The contribution to the resisting moment

$$M_j^R(\gamma, \dot{\gamma}) = \frac{R(\eta \dot{\gamma}\, dx_j + (w_j - p_j\, dx_j) f_a(\gamma) \text{ tg } \phi_a}{\cos \alpha_j + f_a(\gamma) \text{ tg } \phi_a \sin \alpha_j}$$

that is

$$M_j^R(\gamma, \dot{\gamma}) = C_j(\gamma) + \dot{\gamma} D_j$$

The moment equilibrium results in:

$$\sum_{i} M_i^R(\gamma) + \sum_{j} C_j(\gamma) + \dot{\gamma}\Sigma D_j = M^\circ$$

M° = the moment (due to the weight of the sand lean asphalt)

1. If $\dot{\gamma} \rightarrow 0$ when $t \rightarrow \infty$

   $$\sum_{i} M_i^R(\gamma_{eq}) + \sum_{j} C_j(\gamma_{eq}) = M^\circ$$

   γ_{eq} = final value of the deformation

2. If the following is true for all values of γ:

   $$\sum_{i} M_i^R(\gamma) + \sum_{j} C_j(\gamma) \leqslant M^\circ$$

   then the deformation ratio for large values is given by

   $$\dot{\gamma} = (M^\circ - \sum_{i} M_i^R(\gamma_a) - \sum_{j} C_j(\gamma_a))/\sum_{j} D_j$$

   in which γ_a is the value of the shear deformation at which the shear strength for sand and lean sand asphalt is fully mobilized ($f_s = 1; f_a = 1$).

Appendix VII

Sliding along a membrane

A membrane is generally covered with a revetment material. The following situations can develop:

1. The revetment slides off because the membrane is too smooth.
2. The shear stress transmission in the membrane leads to such large deformations that there is either a complete change of geometry or damage to the cover layer.

VII.1 Cover layer sliding

In the case of a bitumenous revetment which has no built-in stability the friction component along the membrane must be greater than the component of its weight along the membrane or else the revetment can slide off, see Figure 1.

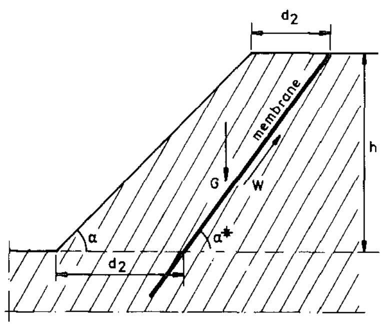

Figure 1 Forces which develop when sliding occurs.

The weight of the cover layer is:

$$G = \frac{\varrho g}{2}(d_1 + d_2)h$$

The friction, W, along the membrane is:

$$W = fG \cos \alpha^*$$

in which:

ρ = density of the revetment material (kg/m^3)
g = acceleration due to gravity (m/S^2)

α	= revetment slope
α^*	= membrane inclination
d_2	= horizontal thickness of revetment-top (m)
d_1	= horizontal thickness of revetment-bottom (m)
h	= height of slope
f	= an ϕ if $\phi < \Theta$; others tan Θ
tan ϕ	= angle of internal friction of the subsoil
tan Θ	= coefficient of friction between membrane and subsoil

To prevent sliding:

$$G \sin \alpha^* < fG \cos \alpha^*$$

that is

$$\mathrm{tg}\ \alpha^* < f$$

When $W < G \sin \alpha^*$ the revetment will be stable on the subsoil, see Figure 2.

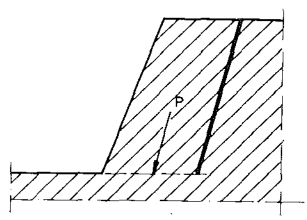

Figure 2 Force exerted by the revetment on the subsoil.

$$P = G (\sin \alpha^* - f \cos \alpha^*)$$

- If the membrane ends at the bottom or is retained at an angle α^* it should not be possible for the force P to cause the toe material to slide off, see Figure 3.
  This can be studied using the Prandtl wedge approach, see Figure 4 (48): The requirements which must be satisfied so that the soil mass does not slide off can be defined by determining the related Mohr circles and ensuring that these lie within the envelope given by the cohesion c, and the angle of internal friction Θ, see Figure 5.

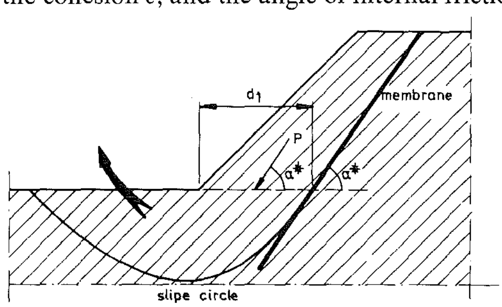

Figure 3 The toe material sliding due to the force P.

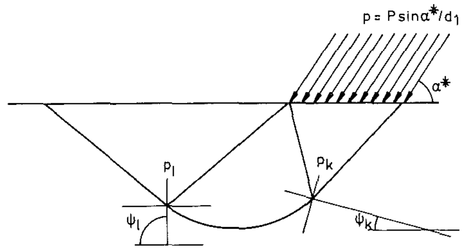

Figure 4 The Prandtl Wedge.

The Mohr Circle, with a centre point q_k, and moving through vector $\bar{p}$, must, in order for there to be no sliding, lie within the envelope; q_k and Ψ_{jk} can then be determined.

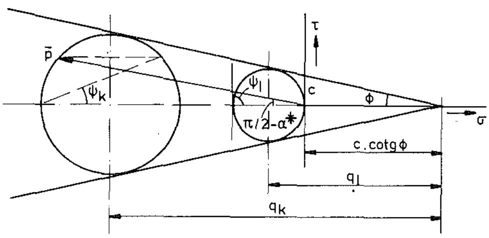

Figure 5 Mohr Circles related by the Prandtl wegde.

The relationship between q_l and q_k is given by:

$$q_l = q_k e^{[-2(\Psi_l - \Psi_k)\,\mathrm{tg}\,\phi]}$$

in which:

$$\Psi_l = \frac{\pi}{2}$$

The Mohr Circle with a centre point q_l, calculated by the above formula and with a principle stress $\sigma = 0$ must lie within the envelope, that is:

$$\text{ofwel } q_l \leqslant \frac{c\,\mathrm{cotg}\,\phi}{1 - \sin\phi}$$

– If the membrane is laid on the bed, see Figure 6, then:

$$P\cos\alpha^* < fP\sin\alpha^*$$

that is:

$$\frac{1}{f} < \mathrm{tg}\,\alpha^*$$

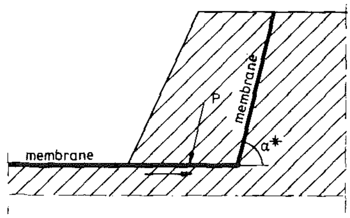

Figure 6 A membrane laid on the bed.

VII.2 Shear stress failure

Because it is viscous an asphalt membrane can creep due to the shear forces exerted on it. In the situation shown in Figure 7 the shear force acting on the membrane is given by $G \sin\alpha^*$.

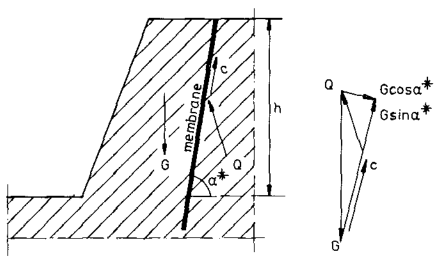

Figure 7 The shear force on a membrane.

The shear stress on the membrane is:

$$\tau = \frac{G \sin^2\alpha^*}{h}$$

If the membrane is laid horizontally under the slope, see Figure 8, the shear stress is given by:

$$\tau = \frac{P \cos\delta}{h} \operatorname{tg}\alpha$$

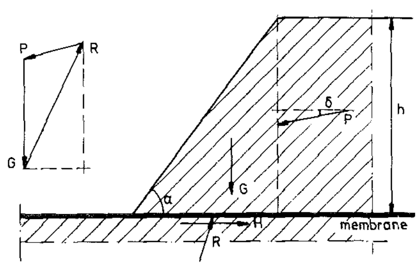

Figure 8 Forces acting on a horizontal membrane.

The size of P can be determined, using a slip surface, from:

$$P \cos \delta = \lambda_{aw} \tfrac{1}{2} \gamma h^2 \quad [48]$$

$$\lambda_{aw} = \frac{\cos \delta}{\cos^2 (\phi + \delta)} (\cos \delta + \sin \phi \sin (\phi + \delta) - 2\sqrt{\sin \phi \sin (\phi + \delta) \cos \delta})$$

$$(c = 0)$$

in which:

γ = specific weight of the soil (N/m^3)
h = height of the slope (m)
δ = angle which P makes (see Figure 8 ($\delta = \frac{2}{3}\phi$)
ϕ = angle of internal friction of the soil
λ_{aw} = active soil pressure coefficient

Deformations will develop in the membrane because of the shear forces. These must not exceed a certain limit. If the membrane comprises only a single layer of bitumen, see Figure 9, and assuming that the bitumen acts like a Newtonian fluid, the deformation can be expressed as:

$$y(h) = \frac{\tau h}{\eta} t$$

and the deformation speed as:

$$v(h) = \frac{\tau h}{\eta}$$

in which:

τ = shear stress (N/m^2)
h = thickness of the bitumen layer (m)
t = time (s)
η = viscosity (N/m^2)

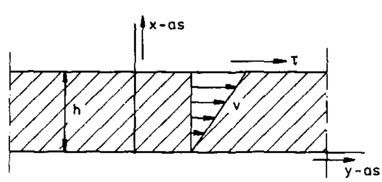

Figure 9 Membrane comprising a single layer of bitumen.

If the membrane is built up from different materials which do not slide over each other, see, for example, Figure 10, the deformation is more complicated.

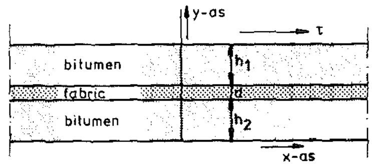

Figure 10 Membrane comprising two layers of bitumen separated by a cloth.

In this case the deformation is given by:

$$y_{tot} = \frac{\tau h_1 t}{\eta} + \frac{\tau h_2 t}{\eta} + \text{strain in the cloth}$$

(assuming that the lower layer of bitumen is fixed)

The cloth is considered to be an elastic material; the strain in the cloth is independent of t.

Strain in the cloth $= \frac{\tau d}{G}$ $\quad G$ = shear modulus of the membrane

The deformation ratio at the top is given by:

$$v(h) = \frac{\tau (h_1 + h_2)}{\eta}$$

Appendix VIII

Mastic flow through a pipe

The flow of mastic through a pipe will, because of its material properties, be laminax in which case the Reynolds number must be less than 2320.

$$Re = \frac{vD\varrho}{\eta}$$

in which:

Re = Reynolds number
v = flow velocity (m/s)
D = pipe diameter (m)
ϱ = density of the mastic (kg/m^3)
η = viscosity of the mastic ($Pa \cdot s$)

The relationship between shear stress (τ), viscosity (η) and velocity gradient (dv/dy) is, for a viscous fluid, given by:

$$\tau = \eta \frac{dv}{dy}$$

The following can be deduced for lamina flow in a pipe with a circular cross-section. The forces acting on the shaded cylinder of fluid, are shown in Figure 1.

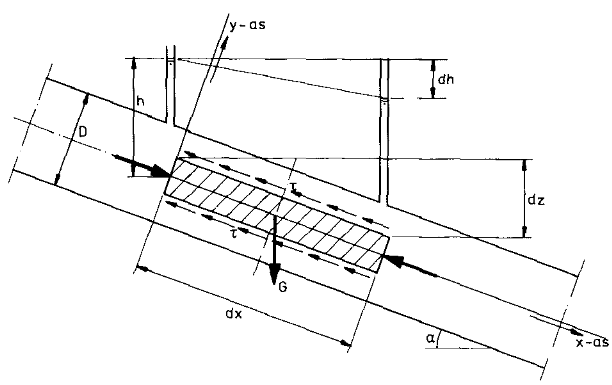

Figure 1 Laminar flow in a pipe.

a. The weight component along the x-axis

$G \sin \alpha = \varrho g \pi y^2 \, \mathrm{d}x \sin \alpha$

putting $\sin \alpha = \mathrm{d}z/\mathrm{d}x$ it follows that

$G \sin \alpha = \varrho g \pi y^2 \, \mathrm{d}z$

b. Fluid friction

$-2\pi y \tau \, \mathrm{d}x$

c. The pressure difference

$\varrho g \pi y^2 (\mathrm{d}h - \mathrm{d}z)$

All forces acting in the x-direction must be in equilibrium, see Figure 1, thus:

$$2\pi y \tau \, \mathrm{d}x - \varrho g \pi y^2 \, \mathrm{d}h = 0$$

since

$$\tau = \eta \frac{\mathrm{d}v}{\mathrm{d}y}$$

this becomes:

$$\mathrm{d}v = -\frac{\varrho g I}{2\eta} y \, \mathrm{d}y \qquad I = \frac{\mathrm{d}h}{\mathrm{d}x}$$

By integration

$$v = -\frac{\varrho g I}{4\eta} y^2 + \text{constant}$$

The boundary condition at $y = \frac{1}{2}D$ is $v = 0$
by which:

$$v = \frac{\varrho g I}{4\eta} \left[\left(\frac{D}{2} \right)^2 - y^2 \right]$$

The discharge in the pipe is:

$$Q = \int_0^{D/2} 2\pi y v \, \mathrm{d}y = \frac{\varrho g I \left(\frac{D}{2} \right)^4 \pi}{8\eta}$$

The mean velocity is given by:

$$v = \frac{Q}{\frac{1}{4}\pi D^2} = \frac{\varrho g I D^2}{32\eta}$$

For a pipe with any given cross-section

$$I = \frac{k\eta Q}{\varrho g D^2 A}$$

in which:

I = gradient of the pressure line
k = a shape factor (k = 32 for circular cross-section pipes)
η = viscosity (Pa · s)
Q = discharge (m^3/s)
ϱ = density of the mastic (kg/m^3)
g = acceleration due to gravity (m/s^2)
D = pipe diameter (m)
A = pipe cross-sectional area (m^2)

$$I = \frac{\varrho_1 - \varrho_2}{\varrho g \, dx}$$

ϱ_1 and ϱ_2 are the pressures at the beginning and end of the pipe section dx respecuvely

A PIPE WITH A CONSTANT DIAMETER

When the pipe has been filled and there is no further supply of material into it, the level will sink as result of flow through the end, until equilibrium is reached, see Figure 2 (23). This equilibrium level is at a height $\varrho_w/\varrho_w \cdot h$ above the end of the pipe, where:

ϱ_w = density of the water (kg/m^3)
ϱ_b = density of the mastic (kg/m^3)
h = depth of the end of the pipe, below water level (m)

If a certain outflow has to be achieved the mastic level in the pipe must be at a particular height b, above the equilibrium level.

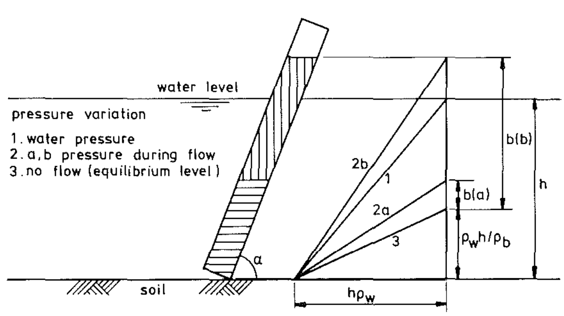

Figure 2 Pressure-diagram in a pipe of constant diameter (23).

For a particular flow through the pipe:

$$I = \frac{k\eta Q}{\varrho g D^2 A}$$

I is the gradient of the pressure variation

$$I = \frac{\left(b + \frac{\varrho_w}{\varrho_b} h\right) - h\varrho_w}{\left(b + \frac{\varrho_w}{\varrho_b} h\right) \varrho_b} \sin \alpha = \frac{b}{b + \frac{\varrho_w}{\varrho_b} h} \sin \alpha$$

From this it follows that:

$$\frac{b}{b + \frac{\varrho_w}{\varrho_b} h} \sin \alpha = \frac{k\eta Q}{\varrho g D^2 A}$$

that is:

$$b = \frac{\frac{\varrho_w}{\varrho_b} h}{\frac{\varrho_b g D^2 A \sin \alpha}{k\eta Q} - 1}$$

To prevent water entering the nozzle the pressure in the pipe must be greater than the water pressure at the same level. The mastic level in the pipe must be at least as high as the water-level outside.

$$b \geqslant h \left(1 - \frac{\varrho_w}{\varrho_b}\right)$$

$$\frac{k\eta Q}{g D^2 A} \geqslant (\varrho_b - \varrho_w) \sin \alpha$$

In the following

$$\frac{k\eta Q}{g D^2 A} = K$$

is given the value.

For the typical density values of $\rho_b = 2000$ kg/m^3 and $\rho_w = 1000$ follows $K \geq 1000 \sin \alpha$. Since it is impractical to work with too high a mastic level K, if $\sin \alpha = 1$, should be less than about 1.30×10^3.

A CONSTRICTED PIPE

In order to prevent water getting into the pipe, instead of maintaining the pressure in the pipe higher than the water pressure over its full length, the end of the pipe can be constricted over a length of ½ to 1 metre so that $K > 1000 \sin \alpha$. The remainder of the pipe should be such that the pressure loss is negligible (23).

N.B.: In the forgoing the viscosity of the mastic is assumed to be constant. In practice, however, it is pressure-dependent. If a pipe is considered with a constricted end, which is not so long, then, over the length of the constriction, the viscosity can be assumed to be constant.

Since at Point A, see Figure 3, the pressure must be at least equal to the local water pressure:

$$K_2 l_2 \geqslant l_2 \sin \alpha \, (\varrho_b - \varrho_w)$$

(The suffix '2' relates to the constricted part of the pipe).

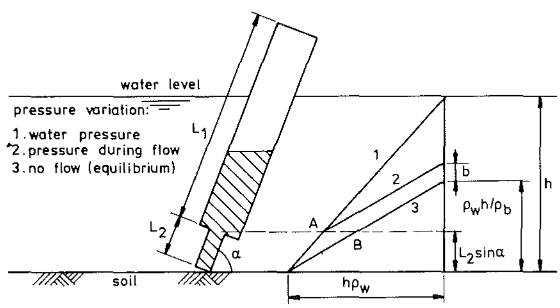

Figure 3 Pipe with a constricted end (23).

With $\rho_b = 2000$ kg/m^3 and $\rho_w = 1000$ kg/m^3 this expression becomes:

$$K_2 \geqslant 1000 \sin \alpha$$

The flow is now controlled by:

$$b = \frac{K_2 l_2}{\varrho_b}$$

that is:

$$b \geqslant \frac{l_2 \sin \alpha \, (\varrho_b - \varrho_w)}{\varrho_b}$$

In practice the value of b is determined by:

- if the pressure difference becomes large the mastic outflow jet will not spatter;
- if the pressure difference becomes low the outflow does not become irregular.

In addition, for construction reasons the factor b should not be too large.
The maximum value of b lies at abou. :he water-level.

In the table below, the maximum and minimum values of K, calculated for various water depths, are given.
It is assumed that $\sin \alpha = 1.l_2 = 0.5$, $\varrho_w = 1000$ and $\varrho_b = 2000$.

Table 1 The affect of a constriction on the variation in the K-value.

	$10^3 \cdot K$			
	constant diameter		constricted diameter	
depth (m)	min.	max.	min.	max.
3	1	1.6	1	6
10	1	1.3	1	20
20	1	1.2	1	20

From Table 1 it is obvious that a much larger variation in the K-value and thus also in ηQ is possible with a pipe with a constricted end than with a pipe with a constant diameter. Irregularities in the mastic during discharge can, therefore, be dealt more easily.

PIPE FITTED WITH A DISTRIBUTOR

To ensure that the material is spread properly a distributor with a number of outlets can be fitted to the end of the supply pipe, see Figure 4.

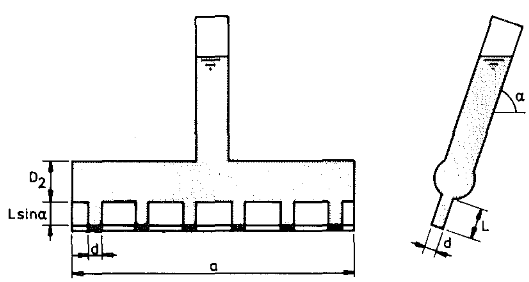

Figure 4 Asphalt distributor (23).

In order for the flow to be uniform over the full breadth of the distributor the pressure drop across the outlets must be much greater than in the distributor pipe. Kerkhoven established, for this:

$$\frac{l}{nd^4} \geqslant \frac{6{,}25a}{D_2^4}$$

in which:

l = length of the outlet pipe (m)
n = number of outlets
d = diameter of an outlet pipe (m)
a = length of the distributor pipe (m)
D_2 = diameter of the distributor pipe (m)

In general, flow contraction of the jet will occur in the mouth of the outlet. A broad opening (a split) is, therefore, only possible if the nozzle is dragged over the bed. This procedure is not recommended when the bed is formed of dumped rubble. Ideally the distributor should be kept at a certain distance above the bed so that scouring around the edges of the apparatus is minimised. This means that a number of outlets has to be applied.
To prevent water getting into the distributor the parameter K should be $\geq 1000 \sin \alpha$

The K-value of the outlets should be several times greater than $1000 \sin \alpha$ because:

- the hanging thread of mastic exerts a negative pressure on the viscous mass in the outlets
- the distributor can become out of alignment.

In view of these detrimental effects it is recommended that K is at least 3000 ($\sin \alpha = 1$).

Appendix IX

The slope test

A viscous material, lying on a slope, will deform under its own weight.
For an infinitely long slope the shear stress at a level x in the material, see Figure 1 (7) is:

$$\tau(x) = \varrho g (h - x) \sin \alpha$$

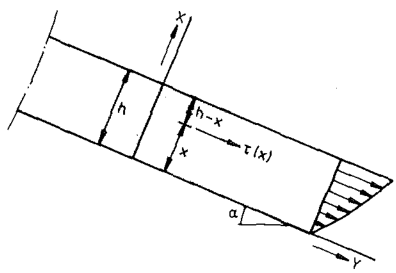

Figure 1 Viscous action on a slope.

The angular deformation is:

$$\gamma = \frac{dy}{dx}$$

In addition:

$$\gamma = \tau / G$$

so that:

$$\frac{dy}{dx} = \frac{\varrho g (h - x) \sin \alpha}{G}$$

$$y(x) = \int_0^x \frac{\varrho g (h - x) \sin \alpha}{G} dx = \frac{\varrho g \sin \alpha}{G} (hx - \tfrac{1}{2}x^2)$$

At the surface of the material ($x = h$):

$$y = \frac{\varrho g h^2 \sin \alpha}{2G}$$

For long duration loads and/or high temperatures, and if the material acts as a Newtonian fluid, the resistance to flow can be expressed in terms of the viscosity as:

$$\tau = \eta \dot{\gamma}$$

therefore:

$$y(h) = \frac{\varrho g h^2 \sin \alpha}{2\eta} t$$

and the flow velocity:

$$v(h) = \frac{\mathrm{d}y(h)}{\mathrm{d}t} = \frac{\varrho g h^2 \sin \alpha}{2\eta}$$

in which:

τ = shear stress (N/m^2)
ϱ = density of the material (kg/m^3)
g = acceleration due to gravity (m/s^2)
h = layer thickness
α = slope angle
G = shear modulus
y = linear deformation (m)
η = viscosity (Pa · s)
γ = angular deformation
v = deformation ratio (m/s)
t = time (s)
$\dot{\gamma}$ = derivative of γ with respect to time