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## ASPHALT REVETMENT OF DYKE SLOPES

by

COMMITTEE ON THE COMPACTION

OF ASPHALT REVETMENTS OF

DYKE SLOPES

1977



G. Laan

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The views in these articles are the authors' own.

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# 1 Introduction

## 1.1 General

The interim report of the 'Working Party on Sealed Dyke Revetments' was published in 1961 [lit. 1]; it put forward a number of arguments in favour of the choice of asphalt revetments for dyke slopes, e.g.:

- rapid completion without a large labour force; this is particularly important in the case of large-scale projects;
- watertight surface giving total protection against water penetration (the main cause of dyke collapse in the 1953 flood disaster).

The Working Party's report shows that too little was known at that time about the quality and long-term stability of asphalt revetments.

A number of setbacks were experienced in particular with structure in the tidal zone, between high and low water. After 1965 an asphaltic concrete revetment was adopted for the delta dams primarily above the tidal zone (above approx. N.A.P.\* + 5 m), while a stone revetment grouted with mastic asphalt was used in the tidal zone. Compaction figures for a number of asphaltic concrete revetments completed prior to 1965 are shown in table 1.

The 'Committee on the Compaction of Asphalt Revetments of Dyke Slopes' (C.V.A.) was set up in 1965 at the initiative of the Delta Project Department of the Public Works Department. This Committee was given the task of investigating how the quality and stability of asphalt revetments could be still further improved by achieving higher compacting densities. The Committee's work was based in part on the results obtained in earlier projects (see table 1).

The authors of the report of the Committee were as follows:

M. J. Loschacoff and J. C. Huis in 't Veld, engineers in the Delta Project Department,

D. Hogervorst, engineer in the State Road Research Laboratory,

W. J. Heijnen, engineer in the Soil Mechanics Laboratory,

R. E. Kerkhoven, engineer in the Royal Dutch Shell Laboratory, Amsterdam,

Professor H. J. Th. Span and G. Kruij, engineer, Koninklijke Maatschappij Wegenbouw N.V.,

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\* N.A.P. = Amsterdam Ordnance Datum

## **1.2 Terms of reference**

It was assumed that the quality and long term stability of new asphalt revetments could be improved by achieving higher compaction densities.

The task of the Committee was to verify this assumption and determine under what conditions the improvement could be achieved. Bituminous binders and structures are liable to be damaged not only by mechanical influences but also by chemical, physical or biological factors (atmospheric oxygen, sea water, ultraviolet radiation, oil, algae etc.).

The principal phenomena which may occur are:

- a. Oxidation as a result of which oxygen present in the air or dissolved in the water combines chemically with the bituminous binder causing it to harden and become brittle.
- b. Stripping: differences in surface tension cause the binder to be displaced by water; the adhesion between the bituminous binder and mineral aggregate is reduced or completely eliminated as a result.

The extent to which these phenomena occur depends in large measure on the surface exposed to such attack, i.e. on the accessibility of the interior of the asphalt structure to water and air.

This accessibility can be limited by reducing the voids content through adequate compaction of the asphalt mix.

The Committee studied ways of achieving the greatest density and hence maximum long-term stability. The following factors were considered: composition, processing and compacting techniques, compacting conditions, structural aspects such as layer thickness and slope gradient, and finally the subsoil characteristics.

Durability as such being difficult to measure, *the density (voids content) of the processed asphalt mix was taken as the principal criterion of quality or long-term stability of the asphalt structure.*

## **1.3 Procedure followed by the Committee**

The Committee investigated the compacting of asphalt revetments of dyke slopes both under practical conditions (1.3.1) and in the laboratory (1.3.2).

### **1.3.1 Practical study**

A distinction may be drawn according to the type of mix studied:

- a. The 'conventional' mix as generally adopted for projects completed in the reporting period.

Table 1 Compaction figures for asphalt revetments of a number of older projects.

Year	Project	No. of core samples	Average voids	Standard deviation
1954	Schelphoek ring dyke	9	4.5	1.6
1955	Schelphoek ring dyke	24	5.1	1.4
1957	Haringvliet construction pit	21	5.2	1.9
1958	Haringvliet construction pit*	14	11.4	3.8
1958	Veerse dam*	19	10.1	3.5
1958	Veerse dam	10	6.2	2.3
1959-1961	Veerse dam	84	6.3	2.3
1959-1961	Veerse dam*	38	4.9	1.7
1958-1960	Hellegat closure*	25	8.5	2.1
1958-1960	Hellegat closure	24	6.2	1.6
1964	Hellegat closure	9	7.5	2.0
1961	Grevelingen dam	90	7.6	2.7
1965	Grevelingen dam	48	7.1	2.4

\* projects completed with gravel asphaltic concrete

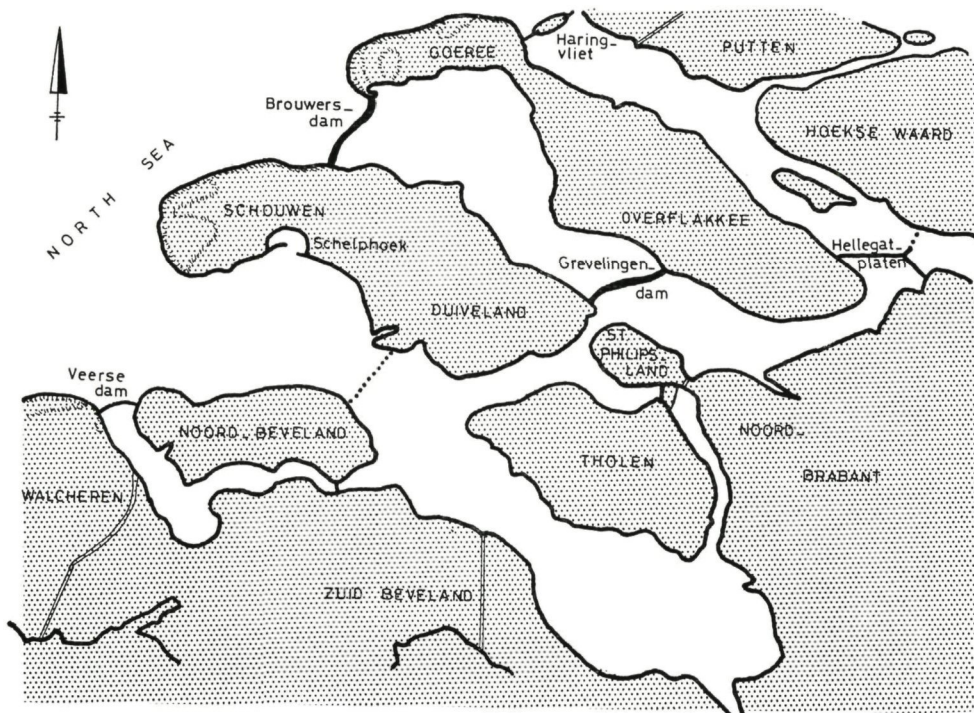


Figure 1. Delta area.



In the first place data were gathered from the results of the project acceptance tests and through extended observations and measurements on the actual structures:

Lauwerszeedam 1965,

Dirkslanddyke 1965,

Brouwersdam 1965–1971.

Although these structures were of similar design, it was sometimes possible to compare in situ the influence on compacting of a number of local conditions such as subsoil type and rigidity, slope gradient, revetment thickness, processing temperature and processing and compacting method. For the structures listed above and for the Enkhuizen/Markerwaard project it was possible to introduce locally a number of variables in respect of the subsoil compacting method, layer thickness and bitumen content.

b. Special mixes; the compacting of these mixes was studied in test sections, in sections of the structure having a temporary function and in the Volkerak dam as part of the normal structure.

A mixture with sand fractions consisting solely of fine sand (shoal sand) and an adjusted amount of filler was examined (see section 6.2.1.). The compaction results for different 'gap graded' mixtures (see section 6.2.2) and mixtures with crushed limestone as the stone fraction and with crushed limestone sand ('Dunkirk' mixture, see section 6.2.3) were also investigated.

### 1.3.2 *Laboratory investigation*

The composition of asphalt mixes for use in hydraulic engineering was originally based largely on methods known from road-building, supplemented by experience gained in the hydraulic sector.

The Committee defined a method of determining the composition of asphalt mixes in hydraulic engineering based on the requirement of a minimum voids content. The determining criteria here were the optimum filler/sand ratio and the appropriate bitumen content.

### 1.3.3 *Reporting*

The Committee set out the results of its findings in the 'Final report of the Committee on Compaction of Asphalt Revetments of Dyke Slopes, March 1975', [lit. 5].

Only a summary of the data relating to the studies carried out will be found in this report.

The content of this Communication coincides largely with the 'Final report' written in Dutch. However, the description of the measurements, sub-divided by project and test structure, has been omitted.



## 2 Asphalt dyke revetments — design and functional requirements

### 2.1 Design

For a given storm surge level and wave height, the wave run-up and hence the crest height of a dyke are determined by the slope gradient and the width of the wave-breaking berm (if a berm is built — see Fig. 11 for an example).

Other conditions being equal, the necessary crest height of a dyke with a faint slope on the seaward side, broken by a berm will be less than for a dyke with an uninterrupted or steep slope. A dyke with a slope interrupted by a berm will generally be the more economical solution.

A faint slope is favourable for asphalt revetment and better compaction results can be achieved (see section 7.4). It is most economical to make the inward slope as steep as possible although the gradient should not exceed 1 : 3 to avoid difficulty in compacting the asphalt. By adopting a high inward berm it is possible to keep the steep inward slope short.

Asphaltic concrete for the revetment of sea dykes is only adopted above the tidal zone because in the tidal zone:

- high pressures may be encountered requiring great thickness;
- steam formation resulting from the emergence of tidal and pressure water from the warm asphalt concrete mixture may cause the asphalt revetment to become porous and lose its density. This increases the likelihood of stripping and aging.

The appropriate asphalt structure in the tidal zone is a stone revetment grouted with mastic asphalt.

In the area where the revetment is exposed to severe wave loads, strength considerations will make a thick layer of asphaltic concrete necessary. Little experience has been gained so far of very thick asphalt revetments. Stone revetments grouted with mastic asphalt have been the preferred solution up to now in this area.

In the area above the highest wave crests (roughly above storm flood level under Dutch conditions), the asphaltic concrete only has a lining function: no heavy pressures or severe wave loading are encountered above storm flood level.

It is desirable to apply the asphaltic concrete revetment in a single layer for the following reasons:

- even if a special adhesive layer is provided, good adhesion between two or more layers is difficult to ensure because of fouling by blown sand and water;
- where a single layer is used, more height is available for a good edge joint.



Stone revetment grouted with mastic asphalt in the tidal zone.

In practice when two or more layers were used in the past, the lower layer was often built to a more porous and hence cheaper design, making it very likely that this layer would perform no function in absorbing any high pressures.

To seal the whole structure, the surface is generally treated with a 'sealing layer' consisting of an asphalt bitumen emulsion with distributed chippings. The asphalt bitumen emulsion is usually a fast-breaking emulsion ('Public Works Department Specifications – Type 0').

## **2.2 Functional requirements**

In general, mixes must meet the following requirements:

1. Adequate impermeability to air and water to ensure
  - a) long-term stability;
  - b) maintenance of maximum weight against uplift water pressures.
2. Ease of processing (easy to spread and compact) to allow application on slopes and having regard to the low subsoil rigidity.
3. Adequate stability on slopes in the warm and cold state.
4. Adequate mechanical strength, especially to resist wave attack.

The first requirement can easily be met by opting for a high bitumen content.

However, a high bitumen content may cause problems in respect of ease of processing and stability on slopes (requirements 2 and 3).

A compromise must therefore be found which will depend on the relative priority given to the different requirements. The thickness of the asphalt layer is determined by stability requirements and by the force of the wave attack. Under the conditions prevailing in the Netherlands, the stability factor has a decisive bearing on applications above the tidel zone. No further study has therefore been made of wave attack on an asphalt revetment.

In road building high stability is the most important requirement in connection with the traffic load. This factor partly determines the bitumen content which must not be too high. In order to obtain sufficient density, heavy rollers must then be used and a rigid base is essential. In hydraulic engineering on the other hand, the requirements placed on mix stability are not high. The static stresses resulting from the location on a slope are so low that in the final state they do not cause any risk of flow; moreover there are no heavy traffic loads. The low stability requirements are a favourable factor for the use of asphalt mixes in hydraulic engineering. By using a higher bitumen content than is usual in highway engineering, it is possible to meet the requirement of ease of compaction and hence also the important criterion of impermeability to water and air.

In conclusion the central problem is to obtain a sufficiently dense, but nevertheless easy to process mix, with the minimum required stability.



### 3 Process of compaction of asphalt dyke revetments

#### 3.1 Theoretical approach

In order to analyse the behaviour of an asphalt layer under compaction, the mechanical model must be at least approximate known.

In the most elementary form, this model consists of a two layer system on which external, primarily vertical loads are exerted by the compacting machine which is moved forwards at a given speed  $V_w$  (figure 2).



Compaction of a asphalt revetment on a berm.

These loads must be large enough for the material of the bituminous layer to be compacted, thus reducing its thickness. For this purpose it is obviously necessary for the underlying layer to offer sufficient resistance to the exerted loads, so that deformation in this layer remains low and no non-recoverable plastic movement occurs. Broadly speaking, this layer must only undergo elastic deformation as a result of the

weight of the compacting equipment. This elastic deformation may be characterized by attributing a modulus of elasticity  $E_2$  to the layer.

In the case of the bituminous layer the situation is quite different. Plastic changes of shape must be brought about here in order to obtain the desired compaction effect. This means that the applied stresses must exceed a given stress level which forms the boundary between the elastic and plastic deformation area of the material of the layer to be compacted. In this case the deformation characteristics of the bituminous layer cannot be characterized solely by a modulus of elasticity.

To gain an impression of the deformation characteristics, the composition of asphalt mixes must be considered in more detail.

An asphalt mix consists of the following components:

- a. chippings, graded or ungraded;
- b. sand, well or less well graded;
- c. filler;
- d. bitumen.

The composition of an asphalt mix can be accurately characterized as a frame consisting of the stone component with the intervening spaces wholly or partially filled by a mixture of sand, filler and bitumen: i.e. a viscous mortar or mastic asphalt (see section 4.1). This mixture also ensures good binding or cohesion between the stones in the frame. The viscous nature of this mortar is determined by overfilling with bitumen the voids in the sand/filler mixture.

When natural sand is used, the threshold value of internal shear above which this viscous characteristic sets in is negligibly low. With broken sand this is not the case. There is then a given (if small) threshold value.

On the basis of the above brief description of the asphalt mix, the deformation characteristic may be defined as follows: where the stone matrix is not entirely filled with mastic, deformation will in the first instance be determined mainly by the degree of elasticity of the stone matrix.

When the shear resistance of this matrix is exceeded, plastic deformation will occur resulting in compaction of the stone framework.

Where the stone framework is entirely filled with mastic, the level of this shear resistance will be determined by the properties of the mastic and by the angle of internal friction  $\varphi$  and the angular resistance  $\tau_h$  of the stone framework.

It may be expected that with a still uncompacted asphalt mix (low  $\varphi$  and  $\tau_h$ ) plastic deformation will already occur with very low external static pressures exerted locally. Under dynamic loading the viscosity of the mastic provides the greater part of the resistance to shear.

This latter factor will thus be decisive in the dynamic compacting process.

If, as a result of compaction, the voids between the stones have been reduced to such

an extent that they are completely or almost completely filled with mastic, changes of shape may still occur during compaction; however, this will not result in any change in the volume of the mix. The compaction process can then be considered complete because the practical limit has been reached.

It follows from this observation that the bituminous layer behaves during the compaction process as a primarily viscous material in which shear stress opposed to compaction develops as a consequence of the deformation taking place at a given speed.

The potential shear resistance of the stone framework ( $\tau_h$  and  $\phi$ ) ensures that the stability of the warm slope revetment is maintained but it is of little importance in relation to the forces exerted during compaction, at least to begin with.

The level of shear resistance in the mastic depends on the viscosity and deformation speed gradient. Figure 2 shows the approximate deformation speed gradient in the vertical direction below the wheel.

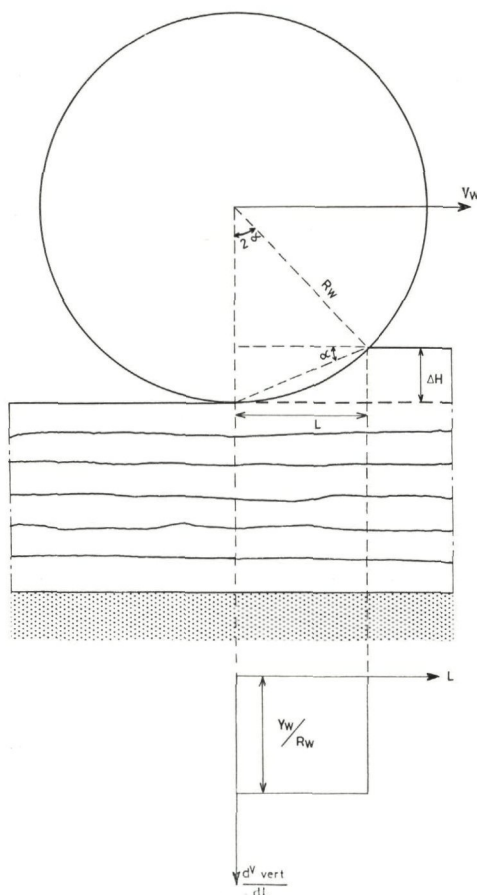


Figure 2. Representation of gradient of deformation speed vertically below the wheel.



This can be calculated very simply from the geometry of the moving wheel.

It may be expressed as  $V_w/R_w$  where  $V_w$  is the speed of advance of the roller and  $R_w$  its radius.

We thus obtain the following expression for the mean shear resistance in the mastic:

$$\bar{\tau}_m \approx \frac{V_w}{R_w} \eta_m$$

in which  $\eta_m$  is the viscosity of the mastic. At the processing temperature,  $\eta_m$  may range from 1000 to 10,000 Poise.

For  $V_w = 80$  cm/sec,  $R_w = 40$  cm and  $\eta_m = 3000$  Poise we obtain:

$$\bar{\tau}_m \approx 6000 \text{ dyne/cm}^2 (= 6.10^{-4} \text{ N/mm}^2).$$

This is a very low value measured against the bearing capacity of the subsoil.

In outline form, the compaction process can now be represented by the following dimensionless expression of Nijboer [lit. 2]:

$$V = \frac{(\sigma_w - C_1 \tau_h - C_2 \tau_m), n}{\eta_m \cdot \frac{V_w}{a}}$$

in which:

$V$  = percentage reduction in voids content,

$C_1, C_2$  = constants,

$\sigma_w$  = rolling pressure,

$n$  = number of roll passes,

$a$  = longitudinal dimension in relation to deformation speed gradient (= radius of roll  $R_w$  or thickness of asphalt layer  $h$ ),

$\eta_m$  = mastic viscosity.

The percentage of stone and its grading clearly also play a part. However, for greater simplicity they will not be considered here. The numerator of the above expression shows the actual pressure  $\sigma_w$  exerted by the roller, less the tensions opposed to compaction, i.e.  $\tau_h$  and  $\tau_m$ ; the dimensionless constants  $C_1$  and  $C_2$  must be introduced in order to bring the effect of these shear stresses to the correct value. The difference between these stresses is decisive for the external influence that must be exerted by compaction. This difference must also be multiplied by the number of roll passes to obtain the total influence. While the numerator shows the total external influence, the denominator of the function shows the resistance caused by the viscosity of the mastic, i.e.  $\eta_m$  multiplied by the speed gradient  $V_w/a$ ; this resistance is also opposed to compaction and is in the form of tension.

The above consideration of the compaction mechanism did not take account of the nature of the subsoil. It was assumed that the subsoil would not undergo too much deformation, in other words that it must have a reasonable bearing capacity. The increase in tension due to the weight of the roller could be the only cause of deformation. The roller pressures are in the order of  $0.5 \text{ kgf/cm}^2$  ( $5 \cdot 10^{-2} \text{ N/m}^2$ ). Even if the spread of this load is in practice very low, the pressure exerted on the sand will still be well below the bearing capacity limit, even if the sand bed is very loosely packed. Thus the deformation which occurs will remain slight.

It may be concluded from this that the influence of the rigidity of the soil under the revetment can have little influence on compaction of the asphalt mix.

This was confirmed by a number of tests in which the nature of the subsoil under the asphalt layer to be compacted varied (see section 7.3).

### 3.2 Influence of the various physical parameters on compaction

Nijboer's formula enables some insight to be gained into the influence of the various physical parameters on ease of compaction.

The function must reach a given maximum value if adequate compaction is to be obtained.

In order to attain the required minimum stability, the shear stresses ('angular resistances')  $\tau_h$  and possibly also  $\tau_m$  must have a given minimum value. This requirement also determines the minimum value of  $\tau_w$  and hence of the roller characteristics (weight, diameter and length of roll, vibration effect).

Since  $\sigma_w$  must be kept as low as possible (light roller) because rolling is done on a slope and because of the low rigidity of the subsoil, the compaction function value can only be increased by:

- a. reducing  $\eta_m$  and  $V_w/a$ , i.e. by using a layer of viscous mastic and a low roller speed  $V_w$ ;
- b. increasing the number of roller passes  $n$ .

It follows from the above that:

1. A layer of viscous mortar or mastic must be used in the mix;
2. A given angular resistance  $\tau_h$  must be present in the asphalt mix to obtain sufficient stability;
3. An adequate number of passes must be made with the specified (light)rollers.

N.B. 1. The theoretical considerations were based on a homogeneous layer (the entire layer thickness  $h$  is at a constant temperature). In practice this theoretical model no longer applies in the case of thicker layers.

2. It should be stressed that the above concepts of the theoretical consideration are based on the peripheral values shown in figure 2, namely a roller acting practically at right angles on the surface to be compacted.

## 4 Mix composition

### 4.1 Introduction

The mix composition for asphalt used for the revetment of dyke slopes was in the early stages largely based on experience gained with asphalt mixtures in road-building. For empirical reasons, slight changes were rapidly made in order to better meet the requirements of watertightness and ease of processing referred to in section 2.

The following 'conventional' mixture (or a similar mix) was used for a long time:

chippings 5/15 mm	48% by weight
sand A	45% by weight
very weak filler	7% by weight
<hr/>	
100% by weight	
<hr/>	
bitumen 80/100 'to'	7.5% by weight

— in principle the stone component may consist of river gravel, broken river gravel ('chippings') or limestone. However, because of the poor compaction results (table 1), the low initial resistance (see section 3) and the perhaps lower adhesion, the use of gravel has been abandoned.

Limestone is likely to give better adhesion because of the physical affinity of limestone and bitumen.

Some experience has been acquired of the use of limestone aggregate. The compaction results for asphalt concrete prepared with limestone differ only slightly from those obtained with 'conventional' mixes.

In addition, under conditions prevailing in the Netherlands, broken limestone is rather expensive;

- sand A, conforming to the 'Public Works Department Standards — 1967' falls within the 'ideal' range in the sand triangle (Figure 16);
- very weak filler, conforming to the 'Public Works Department Standards — 1967';
- bitumen 80/100 was found to be most suitable for Netherlands conditions, one important consideration being the desired flexibility of the revetment.

(The bitumen percentage will always be expressed below as a percentage of 100% mineral aggregate).

The most important differences from the composition of mixtures used in road building are as follows: the almost total absence of the 2/5 mm stone component, light



gap-grading making for ease of processing and compaction and the use of a very weak filler (low voids content) giving a more full-bodied ('richer') mixture.

For reasons of stability in the processing phase it was not always possible to fully incorporate into the mixture the normal bitumen percentage of 7.5. Despite the use of increasingly modern compaction equipment adequate compaction of the mix could still not be obtained (mean voids contents of 5 to 6% by volume were measured). This led to a design method being developed which enables the composition of different asphalt mixes to be compared. The criterion for comparison is the voids content at a compaction intensity corresponding approximately to the compaction intensity adopted in practice.

#### **4.2 Design method**

It was already recognized some time ago (1965) that asphalt mixes for slope revetment applications consist essentially of a stone framework in which the stones are coated and the spaces between them filled with a viscous mortar — mastic asphalt — whose viscosity can easily be determined (see lit. 3).

The stone component itself has no voids content, but even with ideal compaction of the asphalt mix when the voids between the stones are completely filled with mastic, some spaces still remain. It may therefore be assumed that this residual voids content is attributable to the mastic itself and results from its being enclosed in a highly viscous medium. These finely distributed spaces which cannot be eliminated by applying external forces are defined as the residual voids content and are primarily dependent on the compaction of the mastic asphalt.

Apart from the compaction factor, the voids in an asphalt concrete mix are thus dependent on the stone-mortar ratio and on the residual voids content in the mortar.

##### **STONE-MORTAR RATIO**

The voids content of compacted 5/15 mm chippings totals some 40% by volume. The volume portion of the mortar must be higher than this voids percentage in order to ensure complete filling. However, when more mortar is used the residual voids content increases again.

This phenomenon is illustrated in figure 3 where the calculated example shows the voids content of the total, ideally compacted mixture as a function of the percentage by weight of chippings. This function consists of two 'branches' relating to the stone structure over-filled with mastic in one instance and under-filled in the other. The minimum value is achieved at the point of intersection of the two 'branches', generally coinciding with a chipping percentage of some 65% by weight. The limited sensitivity of the voids content function in the over-filled area to the dispersion of the mix composition (i.e. the chipping percentage and the grading of the chippings) which always occurs in practice, generally leads to the choice of a proportion of approximately 50% chippings in the mix. As a result of this dispersion in the composition,

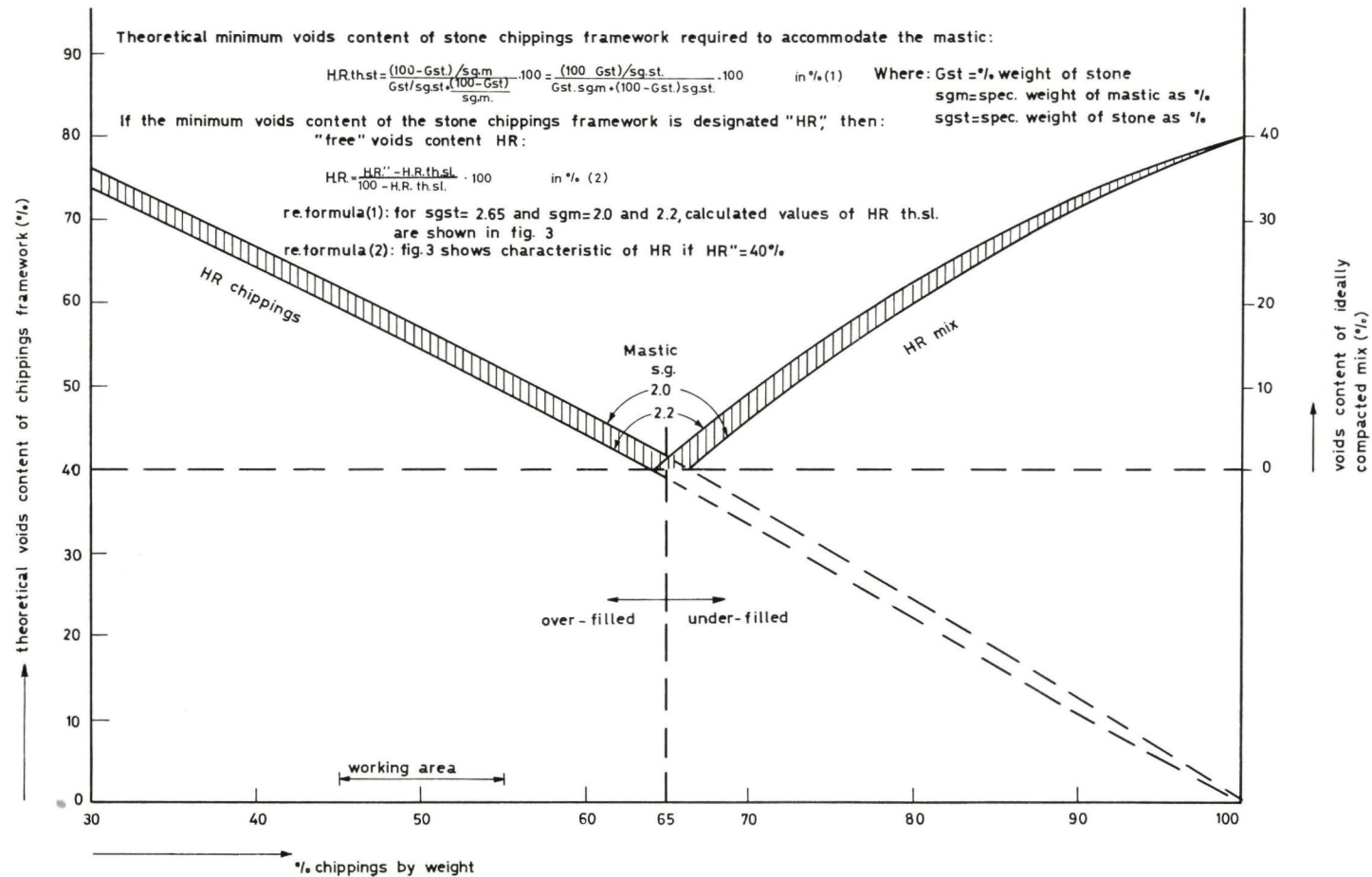


Figure 3. Voids content of stone and ideally compacted mix as a function of % chippings by weight.

a broad safety margin must always be allowed to prevent the highly sensitive under-filled range of this function from being reached.

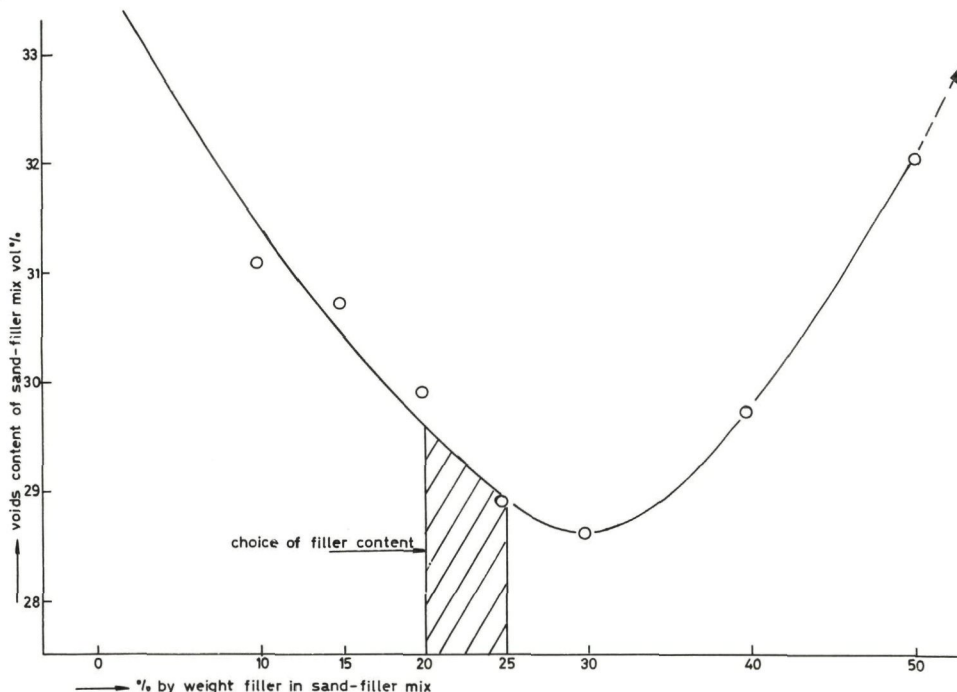


Figure 4. Example of sand-filler compaction according to Engelsmann.

#### RESIDUAL VOIDS CONTENT OF THE MASTIC ASPHALT (MORTAR)

The residual voids content is determined by the characteristics of the components and their mixture ratio. One of the principal influences on the voids content is the sand-filler ratio and the voids content in the sand-filler mixture. This is illustrated in Figure 4.

The type of sand and filler determine the location of the curve representing the sand-filler mix/voids content ratio according to Engelsmann's sand-filler compaction test. In principle the filler content should be chosen to provide a minimum voids content in the sand-filler mix.

Filler contents which exceed this minimum can be disregarded because of an undesirable 'opening' effect. In the area to the left of the minimum, an increase in filler content results in a reduction in the voids content of the sand-filler mix. This means that with a constant bitumen content, an increase in the filler content in principle makes the asphalt mix richer. Since, in the vicinity of the minimum, changes in the filler content cause only slight changes in the voids content, a lower filler content is



chosen in practice for economic considerations than would in fact correspond to the minimum voids content.

Moreover because of the dispersion of the filler contents, a certain margin must be allowed in the practical mixtures, just as with the chippings content.

To obtain a good viscosity, the design method shows the desirability of including a filler percentage of 8-9% in the total asphalt mix. This is higher than the 7% or so generally used to begin with. Comparative practical tests have in fact shown a reduction in the voids content when the filler content is increased (test sections on the Brouwersdam, 1966).

With the stone chippings content determined as described above and the filler-sand ratio, the composition of the mineral aggregate is clearly defined.

Marshall blocks are then produced which enables the voids content of the total mix to be determined as a function of the bitumen content and the compaction energy (Figure 5).

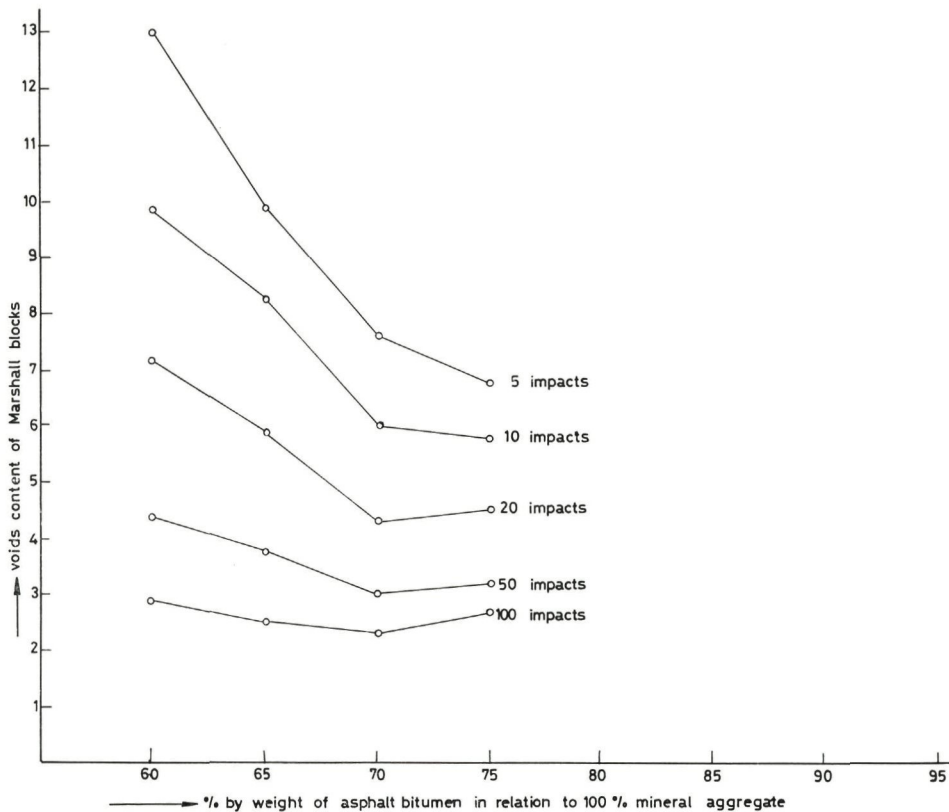


Figure 5. Example of unilateral compaction of Marshall blocks.

The choice of bitumen content is dependent firstly on the shape of the curves which indicate the relationship between the voids content and the bitumen content and secondly on the required voids content in comparison with the voids content achieved in the Marshall blocks, compacted on one side by 20 to 100 impacts (Figure 5). It has been found that this gives a good approximation to compaction under practical conditions.

It will then be necessary to determine in practice whether the stability of the asphalt mix in the application phase with the bitumen content chosen in this way is sufficient to allow satisfactory processing and compaction on a slope. Laboratory flow tests on a slope can only provide limited information on this characteristic.

If the stability of the asphalt mix proves adequate in the construction phase there is no reason to suppose that it will not still be adequate on the accessible slopes in the final phase.

Compaction results for a number of projects completed in recent years where the composition was determined with the aid of the design method for mixes, now also enable a provisional prediction to be made of the likely absolute compaction level.

## 5 Projects studied

As indicated in section 1.3.1, in the period 1965–1972 an analysis was made of the results achieved with projects in which asphalt revetments using ‘conventional’ mixes were applied to dyke slopes.

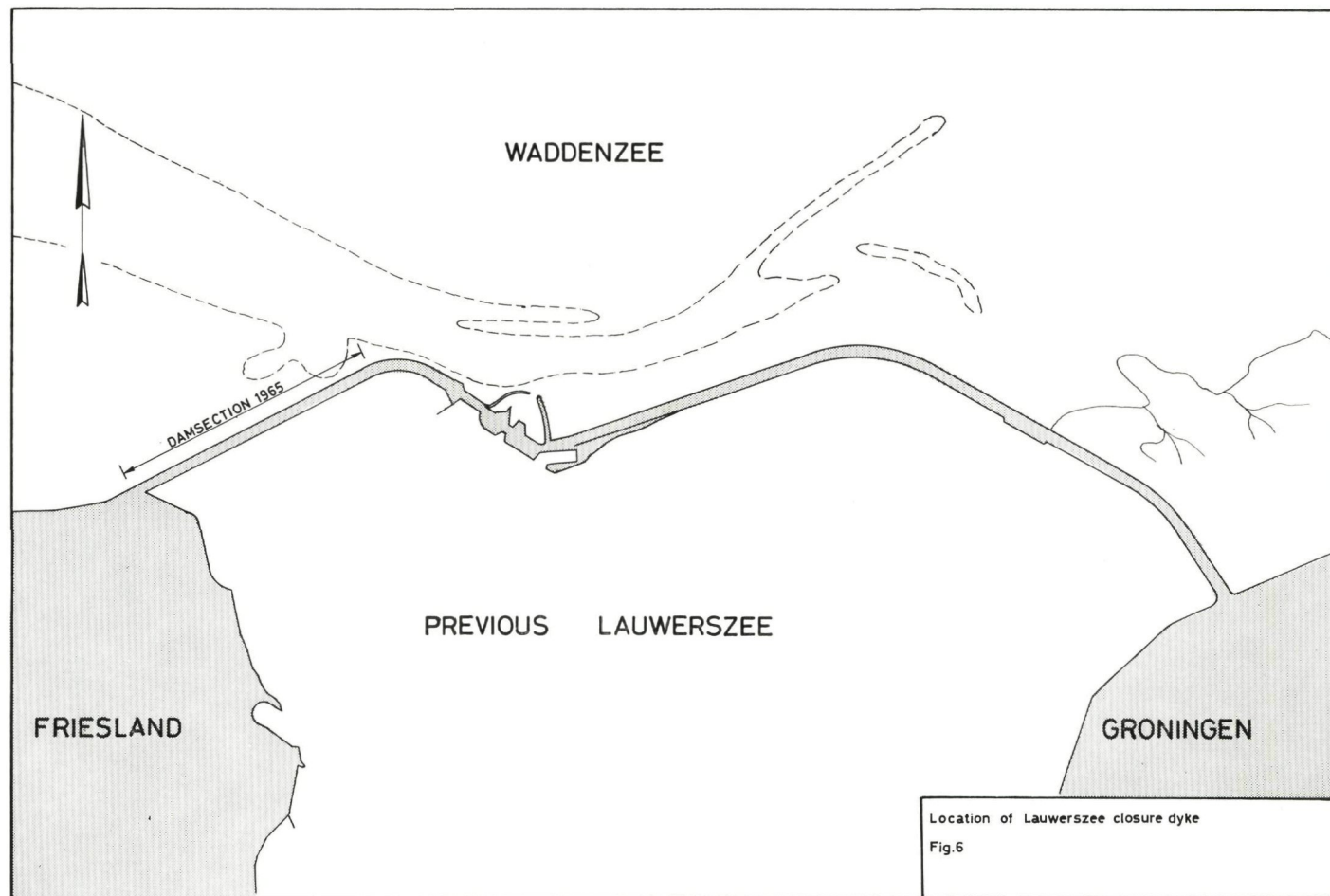
In addition to extensive observations and measurements of the normal structure, it was also possible to introduce on special test sections a number of variables which are liable to influence compaction.

The following projects were involved:

- Lauwerszeedam 1965. Figure 6 shows the location of the structure and Figure 7 the dyke cross-section.
- Dirkslanddam 1965. Figure 8 shows the location of the structure and Figure 9 the dyke cross-section.
- Brouwersdam 1965–1971. Figure 10 shows the location of the structure and Figure 11 the dam cross-section.

In addition to the above structures, it was possible to undertake compaction studies on the Markerwaard dyke (near Enkhuizen) by arranging test sections. Figure 12 shows the location of the structure and Figure 13 the dyke cross-section.

Finally, various asphalt compositions were investigated at the Volkerak dam as part of the normal structure. Figure 14 shows the location of the project and Figure 15 the corresponding cross-sections.



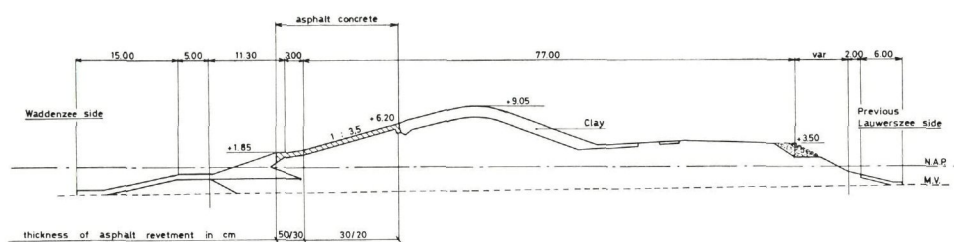


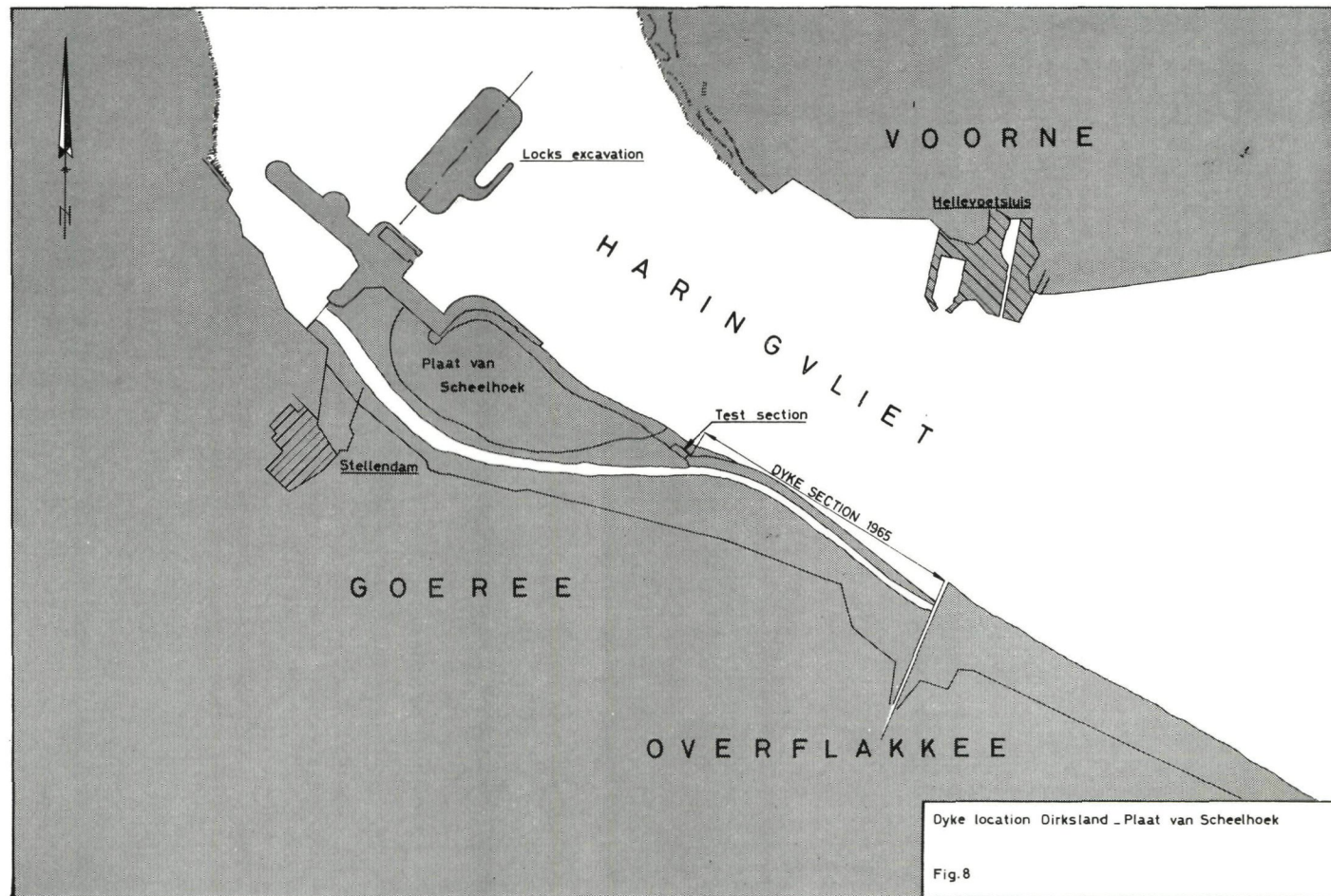
Figure 7. Cross-section of Lauwerszee closure dyke.

Composition of asphalt:

Chippings	45%
Sand A	48%
Very weak filler*	7%
Bitumen 80/100 (% of 100%)	8%
Total processed 23,000 tonnes	

\* Limestone





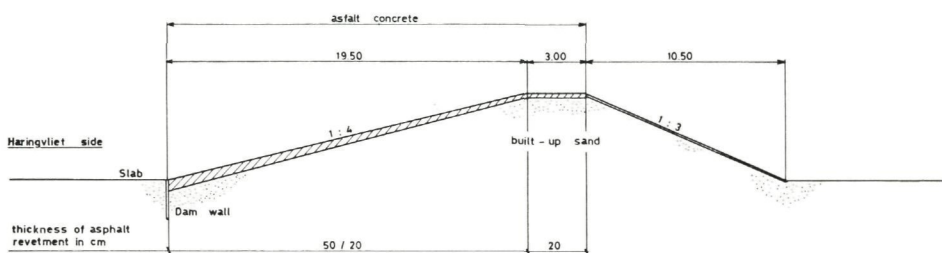
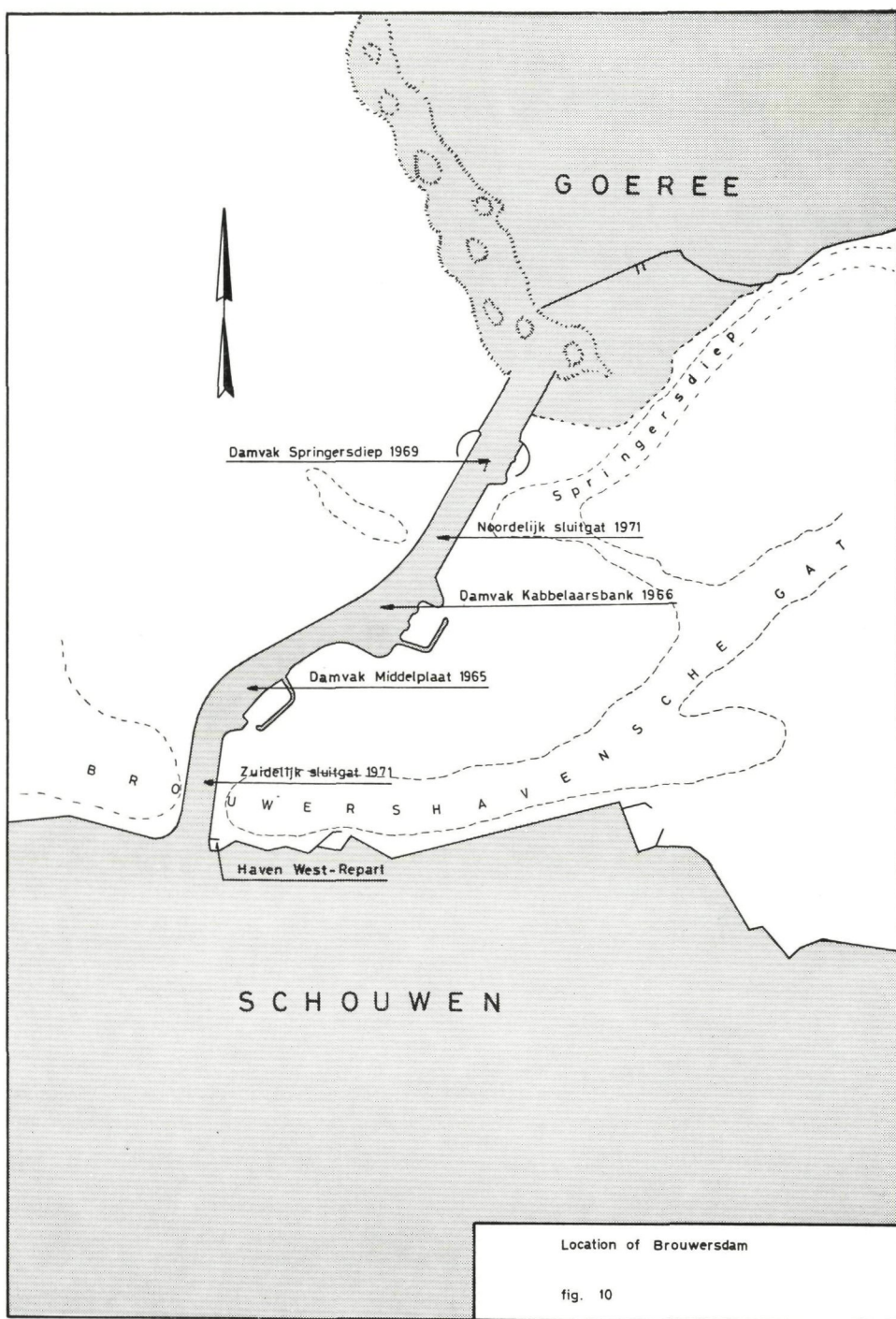


Figure 9. Cross-section of Dirksland Plaat van Scheelhoek dyke.

Composition of asphalt:

Chippings	49.5%
Sand A	43 %
Weak filler*	7.5%
Bitumen (% of 100%)	7.5%
Total processed 39,900 tonnes of asphalt	

\* Combustion dust



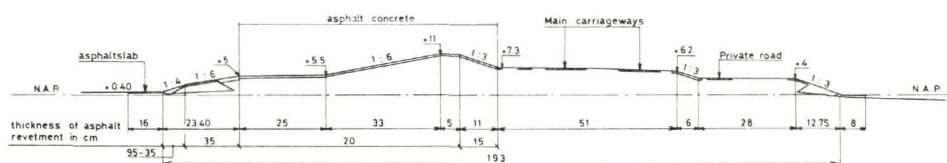


Figure 11. Brouwersdam cross-section.

Composition of asphalt:

Chippings	48 %
Sand A	45 %
Very weak filler*	7 %
Bitumen 80/100 (% of 100%)	7.5%

\* Limestone

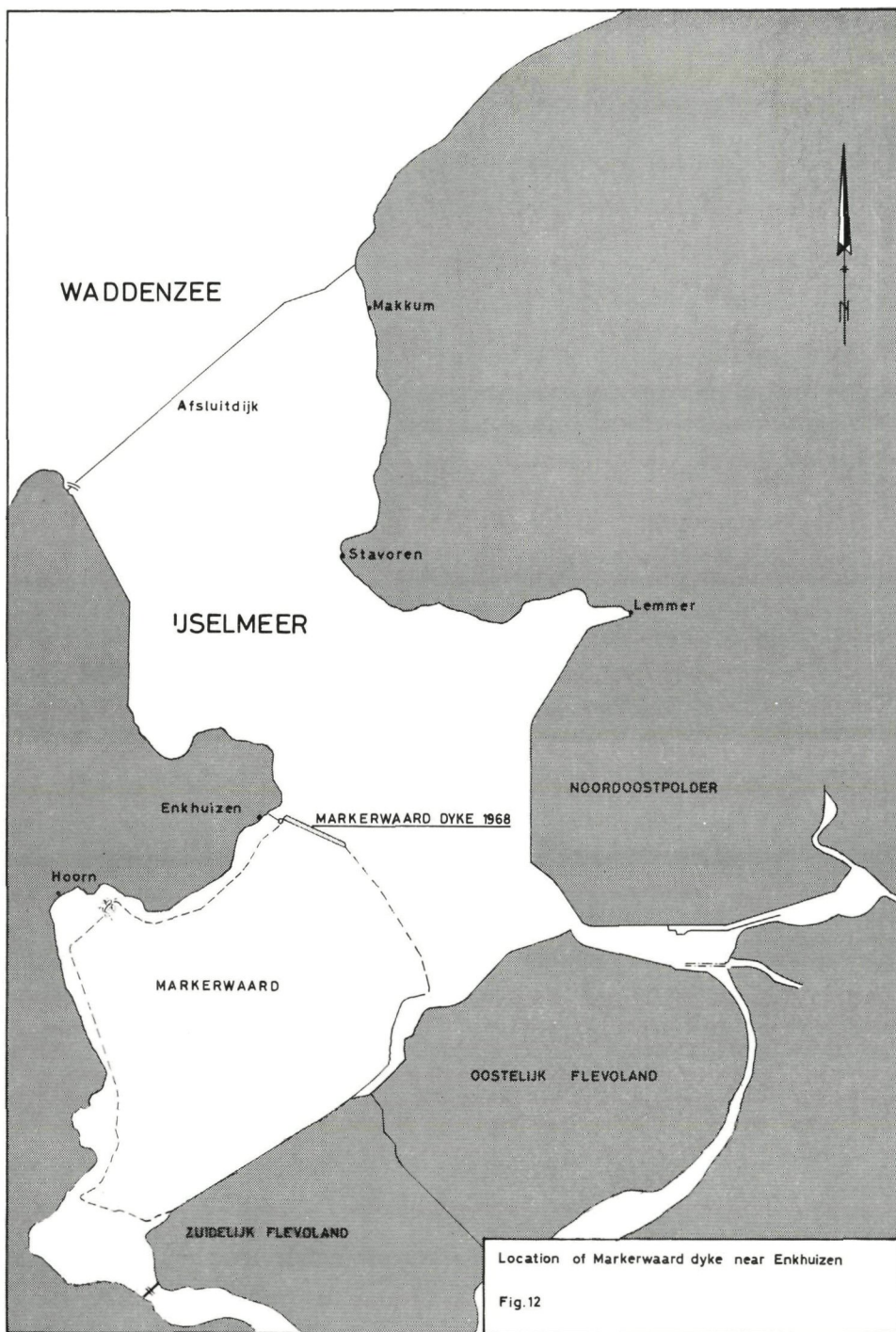
Processed quantities of asphalt concrete in tonnes:

1965	1966	1969	1970	1971	1971
				(north)	(south)
33,200	72,900	5,700	13,900	51,500	55,200

In the light of prior studies, the filler content was set at 9% in 1970 (43% sand) and at 8.5% in 1971 (43.5% sand).

The bitumen content was changed to 7.2% in 1970 and to 7.0% in 1971.





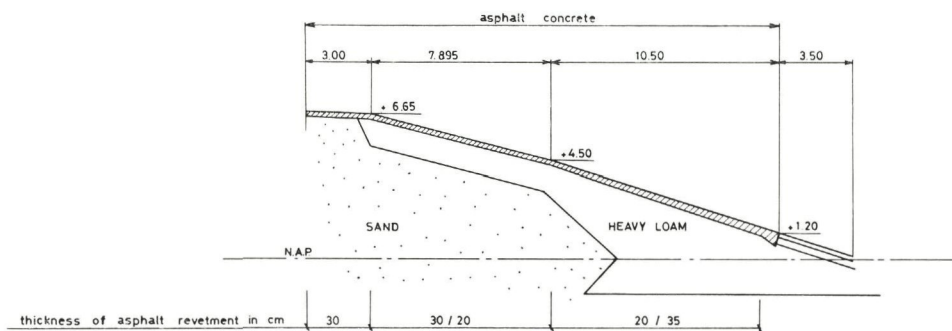
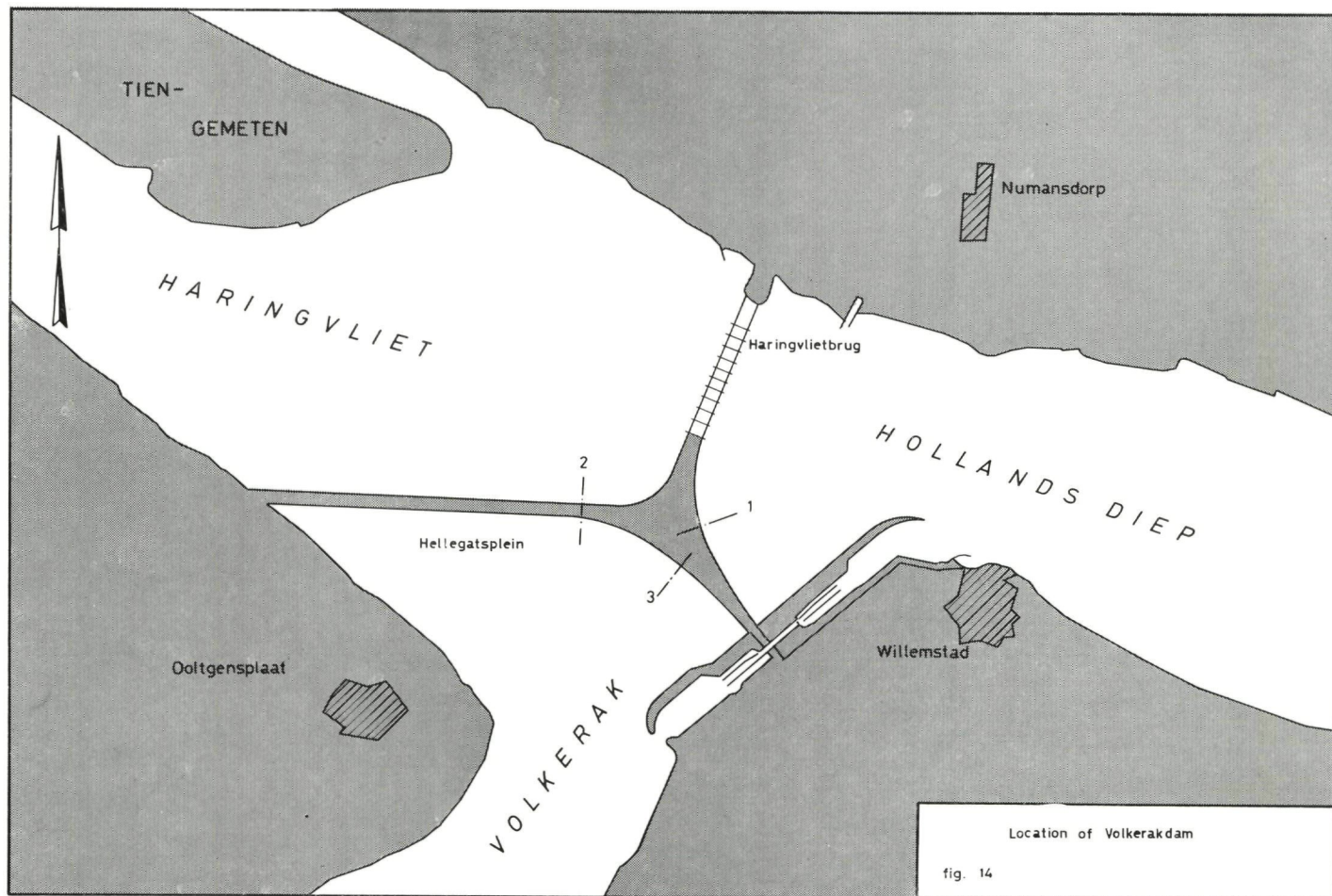
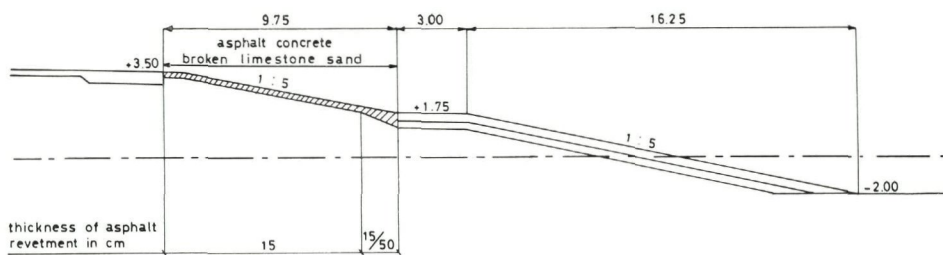


Figure 13. Cross-section of Markerwaard test areas.

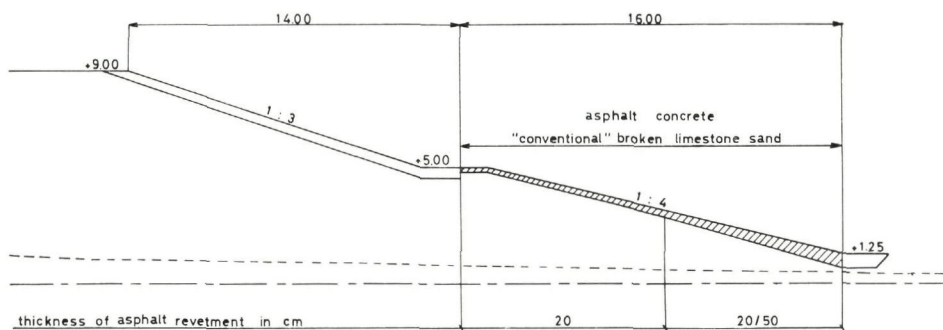
Composition of asphalt:

Chippings	43 %
Sand A	50 %
Very weak filler	7 %
Bitumen (% of 100%)	7.2 %

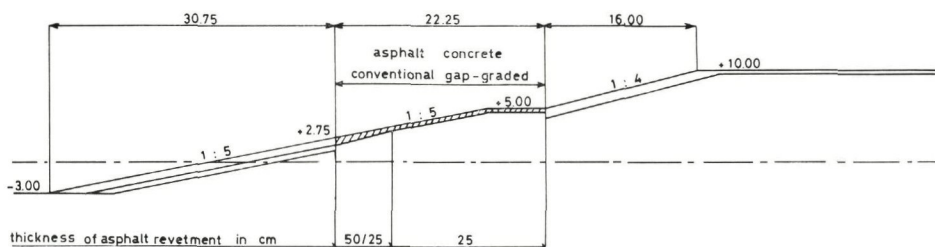




CROSS-SECTION 1



CROSS-SECTION 2



CROSS-SECTION 3

Figure 15. Cross-sections of Volkerakdam test areas.



## 6 Asphalt mix composition (influence on compaction)

### 6.1 Mixture according to design method

The following guideline was drawn up for the composition on the basis of the design method discussed in section 4.2:

chippings 5/15	50%
sand	42%
filler	8%
bitumen 80/100	7%

With good compaction, this composition was found to give (Brouwersdam 1970–71, see section 5) an average voids content in the asphalt mix of 2 to 3% by volume (standard deviation approximately 1%).

– Depending on the layer thickness, the maximum size of the chippings may be increased to 40 mm.

Although the content of chippings was not intentionally varied, the large number of observations of the normal structure showed that a lower percentage of chippings gives higher voids contents in the mixture (see also section 4.2).

– Theoretically any kind of sand may be used.

Some grading of the sand is desirable but it is not imperative for the sand point to fall within the specified range for sand A. The following observations may be made on the location of the sand point in the sand triangle (Figure 16). Sands whose sand point falls within the 'ideal' zone are particularly suitable.

No experience has been gained of sands located in the first quadrant and outside the ideal zone.

As far as is known, natural sand does not occur with characteristics of that kind. Sand in the second quadrant outside the ideal zone — consisting largely of the finest grain size — is less suitable. The filler does not fill the voids satisfactorily and the mastics produced in this way are difficult to work with (thixotropic) and also result in a higher voids content in the overall mix.

Sands whose sand point is located in the third quadrant outside the ideal zone give good results if the filler content is increased; this is confirmed by the results of the sand-filler compaction test. 'Fine sand' mixtures of this kind are dealt with in the next section.

Sands whose sand point is located in the fourth quadrant outside the ideal zone (e.g. 'river sand') have not been studied in detail.

In the light of past experience (Schelphoek 1955), it seems likely that sand of this kind will give good results.

— The filler should preferably be of the ‘very weak’ type, i.e. with minimum voids content, enabling a smaller quantity of bitumen to be used. In addition, constant quality and composition are necessary to prevent undesirable variations in processing characteristics.

Fillers containing limestone should be given preference. As indicated above, a reduction in filler content will lead to an increase in the voids content of the asphalt mix (see section 4.2). The normal dispersion of the filler content in the mix in the normal structure confirms this tendency.

— Various tests (Dirksland 1965, Brouwersdam 1965 and 1966) have shown that a reduction in bitumen content (with an accompanying increase in viscosity) leads to a higher voids content. If the bitumen content is too high the mixture will be over-filled and thus difficult to process because of being insufficiently stable to allow compaction on a dyke slope.

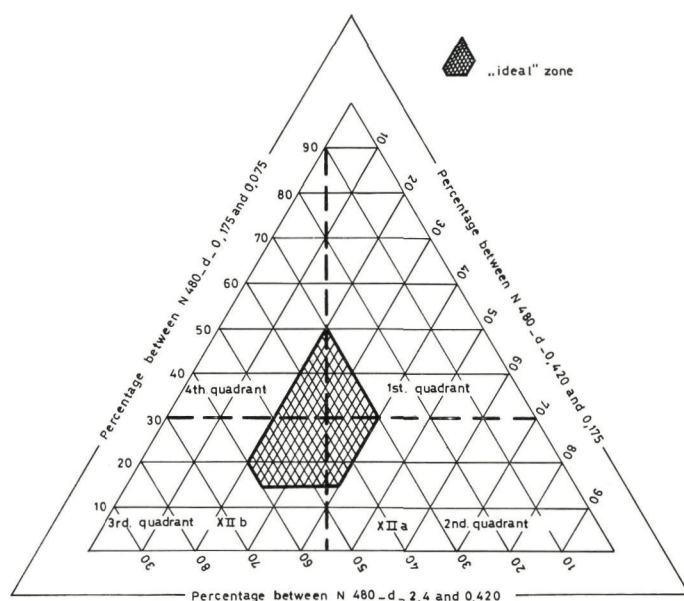


Figure 16. Sand triangle.

## 6.2 Special mixes

Apart from the usual (‘conventional’) mix, mixtures with a different composition are possible. The following special mixes have been studied:

### 6.2.1 Mixture with fine sand

Economic considerations may make it desirable to use locally available fine sand (e.g. shoal sand) instead of sand A. The filler content must then be increased (to

approx. 10%) in the asphalt mixture in accordance with the results of the sand-filler compaction test.

Comparative laboratory and practical studies have shown that compaction results similar to those obtained with asphalt mixes containing sand A can be achieved in this case, provided that the grading of the fine sand which is used contains sufficient average-sized grains, thus corresponding to a sand point in the third quadrant (see figure 16). Mixtures of fine sand with the sand point in the second quadrant (consisting primarily of the finest grains) and filler have a very unbalanced grading which reduces the voids-filling effect of the filler.

#### 6.2.2 'Gap-graded' mixture

It has been found that slight gap-grading, i.e. omission of the 2/5 mm component, is favourable from the point of view of compaction results (see section 4.1).

Replacement of 5/15 mm chippings by 20/40 mm chippings results in a wide gap-grading in the mineral aggregate. Using the design method described earlier, gap-graded mixtures with 20/40 mm chippings and a relatively low mastic viscosity have been produced with which it is possible to achieve a low voids content throughout the layer with only light compaction [lit. 4]. Tests on a practical scale have been carried out with the following composition (Volkerak 1969/70, see Figure 15):

chippings 20/40	59 %
sand A	33.5%
very weak limestone filler	7.5%
bitumen	7 %

The mean voids content in the Volkerak is 4% by volume (standard deviation 1.2%). While these compaction results can be described as good, processing of the mixture proved difficult. The mixture was delivered in a fully compacted state to the site which meant that discharge from the trucks and distribution of the asphalt with the spreader machine were difficult (the fully mechanical processing method was used, see 7.1). The warm flow in the processing phase was also found to be seriously affected by slight deviations in the mixture.

#### 6.2.3 'Dunkirk' mixture

During the construction of a harbour dam in Dunkirk, good results (especially as regards processing and compaction) were obtained with a mixture in which part of the natural sand was replaced by broken sand.

Using the design method described earlier, the following mixture was calculated:

chippings 5/15*	45%
broken sand*	28%
fine sand	19%
very weak limestone filler	8%
bitumen 80/100	7%

\* limestone material was used.

Processing on the practical scale (Volkerak 1969/1970) has in fact shown that the behaviour of the mixture on the slope was very stable in the processing phase; this enabled the vibratory roller to be used for further compaction at an early stage, giving a low voids content. The average measured voids content was 3.5% by volume (standard deviation 1.1% by volume).



## **7 Application of asphalt dyke revetments (influence on compaction)**

### **7.1 Application and spreading**

The asphalt is produced in normal asphalt mixers and delivered to the site by truck. Processing — application and spreading — of the asphalt for slope revetments can be effected in various ways depending on the local conditions and available equipment. A distinction can be drawn between the following types of working:

- a. manual (only limited mechanization);
- b. semi-mechanical;
- c. fully mechanical.

#### **CASE A**

On delivery, the asphalt is discharged into a steel reception tank on the site. It is then spread on the slope with the aid of a crane or dragline bucket. Tracked loaders have also been used for this purpose in which case the loader is driven down the slope to spread the asphalt.

The material is then spread manually (with forks and rakes) and shaped into strips with a width of 3 to 10 m, depending on the local situation, between completed asphalt concrete sections and, if necessary, wooden shuttering.

#### **CASE B**

In the semi-mechanical method, the asphalt is applied directly on the site to the shaped slope profile and spread with a hydraulic crane equipped with a broad bucket; in this way it can be brought roughly into shape, the final shaping being effected with rakes.

#### **CASE C**

In the fully mechanical method, an asphalt slope spreading machine is used.

The bulk asphalt is tipped laterally into the reception tank of the spreading machine by means of raised loader containers. The machine then distributes the asphalt over the entire length of the slope to the desired layer thickness, using an enclosed chamber warmed by hot air to prevent the asphalt from cooling during application.

The asphalt layer is applied with some initial compaction obtained by means of a vibratory beam fitted on the machine; there is generally also a roller on the machine which performs upward and downward movements during the application process.

The semi-mechanical method described in B has been the most frequently used in the Netherlands in recent years. Under normal weather conditions, good results

are obtained with this method. However, when the ambient temperature is low the surface cools rapidly causing surface flaws, i.e. cracks which require extra attention when the sealing layer is applied; at this low ambient temperature the likelihood of inadequate compaction is also increased.

The fully mechanical method of processing enables the asphalt to be applied with some initial compaction and offers almost complete protection against atmospheric influences. However, the variations in slope lengths, slope gradients and working dimensions etc. often prevent this method from being economically viable.

## 7.2 Compaction

The compaction procedure can be subdivided into two phases, initial compaction and subsequent compaction.

Initial compaction should be effected immediately after spreading with a light roller. On faint slopes up to gradients of 1 : 4, lightweight static vibratory rollers (with a twin drive) and a weight of approximately 800 kg are used. Specially built tandem rollers can also be used.

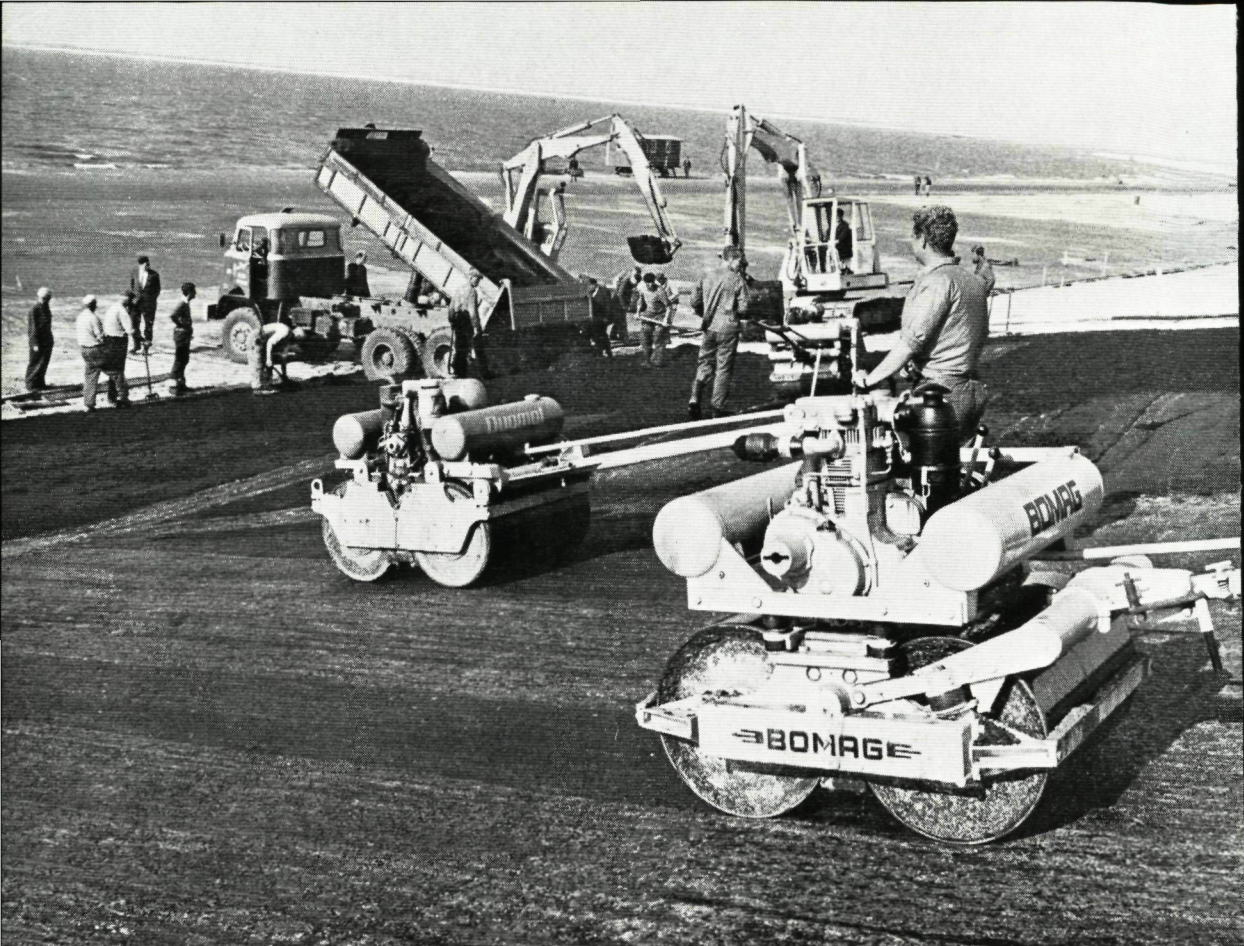
On gradients steeper than 1 : 4 the roller is generally replaced by a lose roller attached by cable to a winch mounted on a truck (or lift-truck). By winching the cable in and out, the roller can be made to move up and down the slope. Rollers weighing 200 to 800 kg with widths of 100. to 1.20 m and a diameter ranging from 0.60 to 1.00 m are used.

Rolling should always take place with a small overlap (about 15 to 20 cm in the longitudinal plane of the dyke). The asphalt should then be treated 6 to 10 times

Fully mechanical method.







Asphalt compaction on a slope with a gradient of 1 : 6 on the Brouwerdam.

with the roller. However, if the profile requires further correction after the first roll passes (e.g. when spreading is effected manually), it is preferable to work with half the roller width, to repeat the rolling operation several times and to adjust the profile in between by raking or additional spreading.

In addition to individual rollers, tandem rollers secured to a cable and with a steering facility are also used on steeper gradients.

As already indicated, initial compaction is coupled with the processing machine in the case of fully mechanized operation. The compaction equipment then consists of a 40 cm wide vibratory beam (centrifugal force 140 kgf/m'\*) and a roller suspended on the machine and designed to perform upward and downward movements during

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\* Varying the centrifugal force (from 0 to 300 kgf/m') proved to have no influence on the compaction results with normal subsequent compaction.

machine progression (weight 350 kg, width 1.20 m, diameter 0.80 m); this roller works with an overlap of 15 to 20 cm at each pass (thus completing on average 6 roller passes).



Asphalt compaction on a slope with a gradient of 1 : 3 on the Brouwersdam.

Subsequent compaction should begin as soon as the stability of the asphalt material allows, depending on the rate at which the asphalt layer cools (which depends in turn on the thickness of the layer and on the atmospheric conditions) and also on the compaction equipment used.

Twin-drive, tandem vibratory rollers with a static weight of 800 to 1300 kg are generally used for this purpose (effective static roller pressure some 2 to 3 times higher than in the initial compaction operation). The interval between initial and subsequent compaction will be between thirty minutes and several hours. To shorten this period as far as possible, the tandem roller should initially be applied statically. In general the roll passes will overlap by half the roller width.

The rolling operation should be repeated several times, tests being made to determine when the mixture is stable enough for the roller to be set to vibrate. On gradients steeper than 1 : 4 considerations of stability of the asphalt material might make it undesirable to vibrate the roller at all on the slope. Rolling is of course continued until all traces of the roller have disappeared. This method will always give a profile sufficiently true to level.



The various tests in which the intensity of compaction was altered all showed the dominant influence of temperature during compaction. It is vital for the compaction (both initial and subsequent) to begin at the highest possible temperature (having regard to the internal stability of the asphalt mix and the stability of the roller on the material). The tests showed that with intensive initial compaction (many passes with a light roller at high temperature) it is even possible in some cases to achieve satisfactory compaction without subsequent compaction using heavier tandem vibratory rollers (see measurement results in table 2).

Table 2 Influence of compaction technique, Lauwerszee test sections; slope 1 : 3.7, layer thickness 20 cm; roller 700 kg; roller width and diameter 1.00 m; vibratory roller BOMAG D18, static operation.

Asphalt compaction			No. of core samples	Mean voids content in % by volume
No. of roller passes		Mean rolling temperature in °C at 5 cm below surface		
Roller	Vibratory roller			
2 to 3	4 to 8	100 to 130	16	5.0
10	—	120 to 150	3	3.6

### 7.3 Influence of subsoil

Tests were carried out at four different project sites to determine the influence of the rigidity of the subsoil on compaction of the asphalt mixture (sounding board effect). Generally the sand bed is simply levelled and compacted with a bulldozer.

This only gives a very slight increase in the original density. The additional compaction of the sand bed by means of a static roller effected during the tests also only gave a slight compaction effect.

For loosely packed sand immediately below the surface, cone resistance values of 1–3 kgf/cm<sup>2</sup> were measured increasing after compaction with a static roller to 3–5 kgf/cm<sup>2</sup>.

It was also striking to note that at all the test sites the original density of the sand — even at a relatively great depth — was, relatively speaking, very low (at depths of about 1 m, cone resistance values of 4–7 kgf/cm<sup>2</sup> are measured).

The investigation of the influence of the subsoil was effected under extremely varied conditions, ranging from loose sand (cone resistance 1–3 kgf/cm<sup>2</sup>) to a cement stabilized bed. The tests showed that the influence of the compaction temperature is the dominant factor as far as voids content is concerned.

Only if the mix is difficult to compact, e.g. as a result of a deviant composition or low

rolling temperature, does improvement of the subsoil rigidity assume any importance. This is illustrated by measurement results in table 3.

Table 3 Influence of subsoil at Dirksland test sections. Slope 1 : 3, layer thickness variable.  
Initial compaction: vibratory beam (centrifugal force 140 kgf/cm<sup>2</sup>); roller weight 350 kg; width 1.20 m; diameter 0.80 m; 6 roll passes.  
Subsequent compaction: vibratory roller; weight 800 kg; 6 roll passes.

Subsoil	Layer thickness 15 cm		Layer thickness 40 cm	
	No. of core samples	Mean voids content %	No. of core samples	Mean voids content %
Sand, normally levelled with bulldozer	9	6.6	3	7.2
15 cm cement stabilization	11	6.0	3	6.9

#### 7.4 Influence of design (layer thickness, slope gradient)

The influence of the structure of the asphalt revetment can be examined from the angles of layer thickness and slope gradient.

##### LAYER THICKNESS

At some test sections, the influence of the layer thickness (variation from 15 to 40 cm) on the voids content was studied. No clear influence was ascertained in the 15–40 cm thickness range studied.

There appears to be a tendency towards higher voids contents at increasing thickness up to approx. 40 cm with mixes which are difficult to process (see table 3).



Berm and 1 : 6 slope gradient on the Brouwersdam.

By sawing the drilled core samples into several parts (generally an upper, central and lower part) during the density test, it was possible to ascertain the voids content distribution over the entire height of the asphalt layer. It was found that with an average compaction level (low voids content) the differences in voids content over the vertical section of the asphalt layer are only small. Generally there is a slight tendency towards a rather lower voids content in the centre of the asphalt layer and a rather higher content at the bottom.

Were the mix is difficult to process, the total voids content will be higher and the differences in the distribution of voids over the vertical section of the layer will increase. The voids at the bottom of the layer will increase particularly.

Table 4 shows the distribution of voids content over the vertical section of the asphalt layer for a number of different structures which were studied (collated measurement results for the normal structures).

Table 4 Distribution of voids over the vertical section of the asphalt layer at several different sites.

	Lauwerszee			Dirksland			Brouwersdam 1970		
	<i>n</i>	mean voids, %	stand. dev., %	<i>n</i>	mean voids, %	stand. dev., %	<i>n</i>	mean voids, %	stand. dev., %
Total	54	5.5	1.5	130	5.6	1.8	89	2.6	0.8
Top	54	5.1	1.5	122	4.9	1.7	81	2.7	0.8
Centre	39	4.4	1.3	100	4.8	2.0	24	2.7	1.1
Bottom	54	7.0	2.2	122	7.1	2.2	80	2.5	0.8

#### SLOPE GRADIENT

In general a steeper slope tends to give a higher voids content.

This influence will of course be all the more apparent, the more difficult the mix is to compact.

The overall conclusion is as follows: in the case of faint slopes with gradients of up to 1 : 4 the influence is only slight but with steeper slopes the tendency for the voids content to be greater is increased (table 5).

Table 5 Influence of slope gradient on voids content.

Slope gradient	Brouwersdam 1965			Brouwersdam 1970		
	<i>n</i>	mean voids, %	stand. dev., %	<i>n</i>	mean voids, %	stand. dev., %
berms	31	3.6	1.0	31	2.3	0.7
1 : 6	17	3.9	0.9	29	2.5	0.5
1 : 4	—	—	—	6	2.2	0.5
1 : 3	7	4.7	0.9	20	3.1	1.0

The fact that the voids content is higher in the case of steeper slopes can be attributed to:

- lower gradient stability of the asphalt mix so that compaction begins later (i.e. at lower temperature and higher mortar viscosity);
- limitation of compaction to static rolling by suspended roller.



## 8 Conclusions

1. In the period during which the Committee performed its work, knowledge was increased to such an extent that it proved possible to reduce the voids content of 'conventional' mixes used for dyke structures from 5–6% to 2–3%.
  2. To reach the low voids content, the asphalt mix should be designed by the method described in section 4. This method makes the correlation between composition and density very much clearer.
  3. The viscosity of the mix during compaction was found to have a dominant influence on the voids content. The aim should be to compact in all cases with the lowest possible viscosity. This can best be done by commencing compaction at the highest temperature at which the mix is still stable on the slope. Compaction at a lower temperature cannot be compensated by heavier and/or longer compaction. A lower viscosity can also be achieved by increasing the bitumen content. However, with a high bitumen content the mix is difficult to process.
  4. Constant grading of the mineral aggregate is important to achieve a mix with constant processing characteristics. In particular, the filler must be of constant quality and composition.
  5. Locally available sand — e.g. fine shoal sand — can be used instead of sand A with favourable compaction results if the filler content is increased sufficiently. In this case the sand point of the sand used must not be located in the 2nd quadrant of the sand triangle (uniformly graded very fine sand).
  6. Favourable compaction results can be achieved both with the 'conventional' mix and with a 'gap-graded' mix or 'limestone-broken sand mix' ('Dunkirk' mix). Under the conditions prevailing in the Netherlands, the 'conventional' mix is the most economical so there is less need for special mixes.
  7. If the mix is well designed and compacted at the exact (low) viscosity, there is no point in further compacting the subsoil. It will be sufficient to grade the sand bed normally with a bulldozer.
  8. The semi-mechanical spreading methods used widely in recent years are perfectly satisfactory.
- Because of the variations in slope lengths and gradients, working dimensions and so on which occur in practice, mechanization was not always attractive.
9. In the design, allowance should be made for the fact that with slope gradients steeper than 1 : 4 the voids content may be higher than on slopes of 1 : 4 or less. With steeper slope gradients it may be desirable to use broken sand mixes, in which case the stability of the mix on the slope is greater in the processing phase and compaction can begin sooner.

10. If the asphalt concrete has a simple covering function, a layer thickness of 15 to 20 cm will be sufficient; it is desirable to apply the asphalt concrete revetment in one single layer. No clear influence of a greater layer thickness on the voids content was observed. The bottom section of the layer generally has a higher voids content than the central and upper sections.

11. It is desirable in the specifications to allow for the possibility of easy adaptation of the mixes to local conditions and available materials. The Final Report, written in Dutch, includes as an annex a number of detailed specifications based on the research conducted by the Committee and also in part on practical findings of the Delta Project Department.

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