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3rd EUROBITUME SYMPOSIUM 1985

BITUMEN, FLEXIBLE AND DURABLE

Van Dijk & Ageme
Bitumen materials in hydraulic Engineering

VOLUME II

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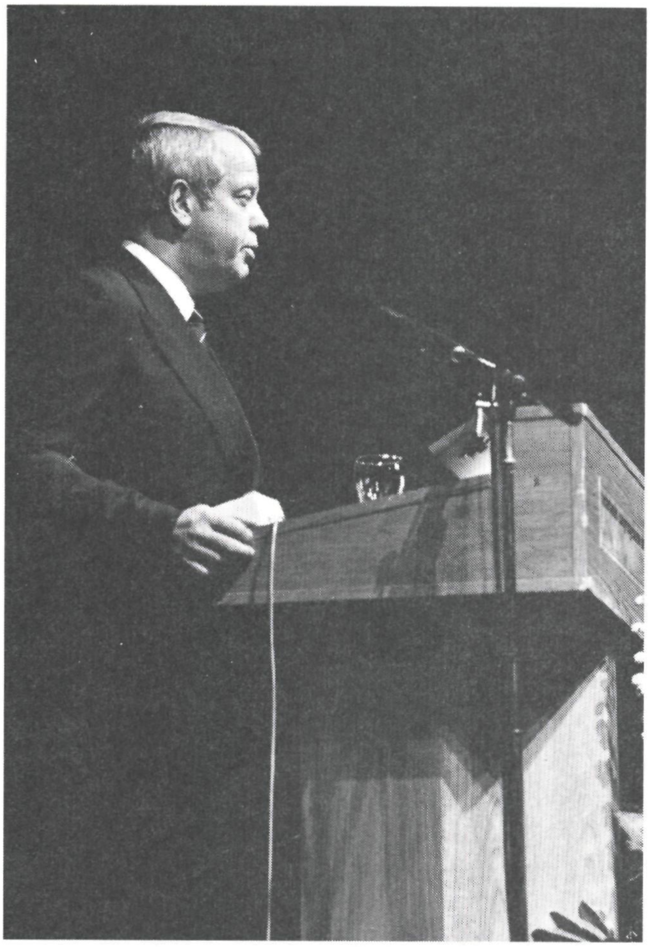
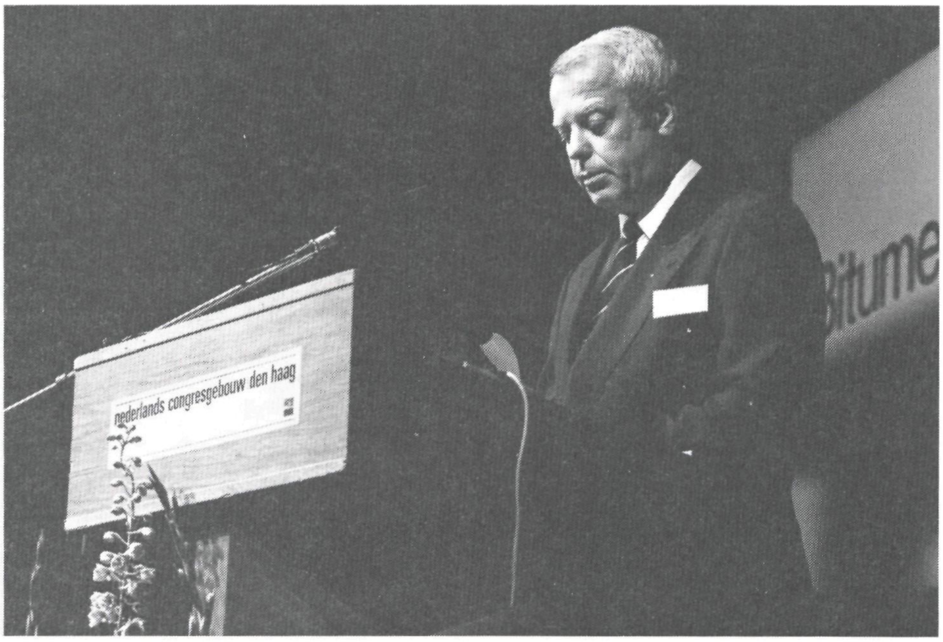
WELCOME by Mr. J.C.G. Bos,
President of EUROBITUME,
The Netherlands





Session V - 'BITUMINOUS MATERIALS IN HYDRAULIC ENGINEERING'

Moderators:

- Drs. W. van Dijk, Professor at the Delft University of Technology, The Netherlands.

- Ir. J.F. Agema, Emeritus Professor at the Delft University of Technology, The Netherlands.

SESSION V

MODERATOR: Prof. Drs. W. van Dijk

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Chairman, ladies and gentlemen,

1. Introduction

The organisation of this 3rd Eurobitume Symposium has taken a wise decision by devoting a special session to bituminous materials in hydraulic engineering. Twenty papers are presented here; this means about 14% of the total number of papers, which reflects the worldwide interest in using bituminous materials in hydraulic structures. As Holland was developed by the Dutch, as the Anglo-Saxons say, it should not supprise you that 7 reports are from the Netherlands, then France with 4, Germany 3, Belgium, Italy and Norway 2 each.
As you see, Europe has a lot of experience in handling bituminous mixes for waterdefences.
The papers in session V, Bituminous Materials in Hydraulic Engineering, are dealing with the application of asphaltic mixes for dams, reservoirs, canals, coast protection structures and storage reservoirs. My colleague and co-moderator, Prof. Agema, will pay special attention to all aspects relalated to the design and the functional aspects of the structures, while I, true to my name, will built up the dike-body by looking at the materials and their properties.
In my review of the reports I shall frequently make use of the figures as given by the authors. For the sake of efficiency I transform them and I hope that the authors will follow me carefully to notice if any mistakes are given in the presented slides. For the others of the audience, at the same time I will show on the right-side some general slides.

2. General formulation of the asphalt mixes

In general, the asphalt mixes are mixtures of mineral aggregates (gravel/stone, sand and filler) and bitumen. The bitumen binder is sometimes modified with additives (chemical or physical bonding of polymers and other fillers). The strength of the mix is mainly influenced by the dimensions and the shape of the aggregates and by the ratio of aggregates and binder in the mix. Either filling or not filling of the voids with finer aggregate and bitumen or mortar will yield asphalt mixes which are either impermeable or permeable to water. The asphalt mixes are therefore grouped in the way as given by Agema et al (paper 16), see Fig. 16,1 (composition of asphalt mixes in hydraulic engineering). The general composition of these mixes are presented in Table I.

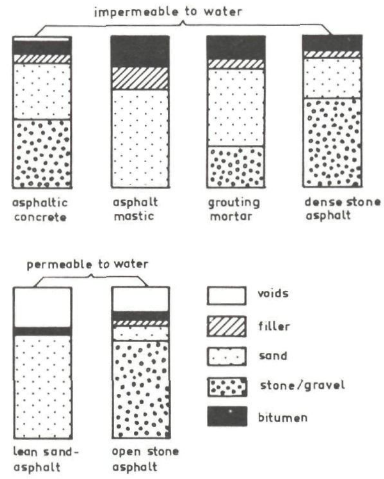


Fig. 16,1.

Table I : General composition of asphalt mixes in hydraulic engineering.

composition by % weight					
Mix	crushed stone	sand	filler	bitumen (80/100)	voids ratio (%v)
Asphaltic concrete	47	39	7.5	6.5	3-6
Asphaltic mastic	-	64	17	19	-
Grouting mortar	30	47.5	11.5	11	~
Dense stone asphalt	60	25,5	7	7,5	2-3
Lean sand asphalt	-	96	-	4	~ 30
Open stone asphalt	83	10	4	3	20-30

Some of these mixes do not need compaction, which makes them therefore suitable for application below waterlevel.
A special group is that of the bitumen membranes and mats or mattresses filled with asphalt mixes, either impermeable or permeable, prefabricated or formed in - situ.
As already said, the asphalt mixes are especially suited for hydraulical engineering-structures. The asphalt mix to be selected depends on the forces which will occur, the function of the particular construction component and the properties of the other materials that form the total structure, see Agema, (paper 16).
The main functions are revetments above water, bottom and toe protection, penetration of blocks, bottom and slope protection revetments, filler and core material, and watertight lining.

3. Asphaltic concrete

Asphaltic concrete is the most widely used type of mix in hydraulic engineering. In general it is a mixture of crushed stones or gravel, sand and filler, in which the pores are practically completely filled with bitumen.

The voids content is from 3 to 6% vol. The mix composition of the asphaltic concretes, mentioned by the reports in this session, are summarized in Table II.
The material must be compacted (essential!) and is unsuitable for applications under water and in tidal zones. By its impermeability the asphaltic concrete is applied as a watertight dike revetment and as a lining (impermeable) for canals, reservoirs and dams.

Haas (paper 3) investigated for this mix the effect of hydraulic fracture, filter stability, strength and deformation behaviour for application as core sealings in barrage dams.
These types of sealings have sofar been successfully applied in dams of up to 100 m in height.

Asphaltic concrete types	Composition by % weight: crushed stone > 2, 12/20 mm	crushed stone > 2, 6/12 mm	crushed stone > 2, 4/6 mm	sand 2 - 0.063, crushed 0/2 mm	sand 2 - 0.063, round 0/4 mm	filler <0.063 (total)	bitumen pha (pen.)	author (report)
dense	60			30		10	8	Hoffmann (4)
support layer		(4/10) 45	(0/4) 38		10	7 (12.3)	6.7 (60/70)	Herment (6)
watertight layer		45	38		10	7 (12.3)	8.2 (60/70)	
joint layer		45	38		10	7 (12.3)	9.5 (180/220)	
support layer	20	20	13	40		7 (14.3)	8 (60/70)	Duval (9)
impervious layer		36	16	30	10	8 (13.6)	8.3 (60/70)	
dense	48			40		12	7.8 (80/100)	Citroni (11)
semi-dense	55			38		7	5.8 (80/100)	
open	82			14		4	4.0 (80/100)	
5 types	49 - 53			39 - 43		6.7-8.4	6.1-6.9 (80/100)	Bondsma (14)
standard	50			42		8	6.5 (80/100)	v. Herpen (17)

Table II Composition of the asphaltic concrete types

Hoffmann et al (paper 4) pointed out that in general terms up to 60 m can be designed with a vertical central core.

To detect hydraulic fracture, Haas developed an apparatus as given in Fig. 3,1 for pressures up to 115 bar (11,5 MN/m2) and temperatures up to 60°C.

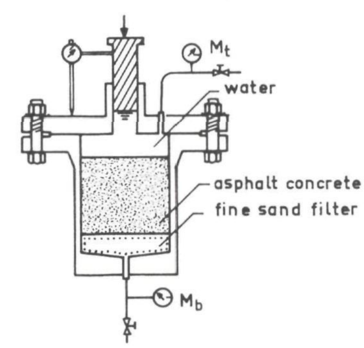


Fig. 3,1.

Can outside waterpressure (penetration of back-up water) cause leakage of the core in an asphaltic material? The viscosity at room temperature is more than a million times higher than that of water (about 2 x 10^7 centi-Stokes).
Fig. 3.3 shows that at a pressure of 75 bar and at 40°C the bitumen (type B65 with 50-70 pen) in the core of asphaltic concrete starts to flow.
Filter stability is related to the filtering laws in soil mechanics, which means that the grainsize distribution curve of the aggregate should be given by the Fuller line.

The sealing cores should have their own strength. Their residual deformation is dependent on the axial and confining pressures.

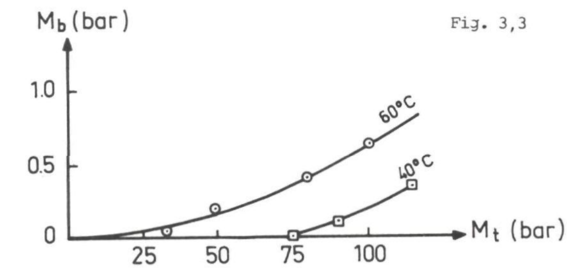


Fig. 3,3

The biaxial type of test is preferable, because in the direction parallel to the dam axis the supporting pressures are bigger and in the direction normal to that they are smaller than one would expect in comparative triaxial tests. The non-linear, stress dependent, permanent strain of a bituminous mix may be represented by a hyperbolic function.

Haas submitted the specimen to different radial supporting pressures at a constant vertical pressure and continuously registered the volume changes.

The results are given in Fig. 3.4 which shows the volume changes after attaining different radial expansions ε_3.

When the radial support is missing (σ_3 = o), the asphaltic concrete shows dilatation after initial compression; this may not exceed the limits of impermeablility.

On the other hand under equal radial pressures a specimen with a higher air void content more easily compressed than one with a lower void content.

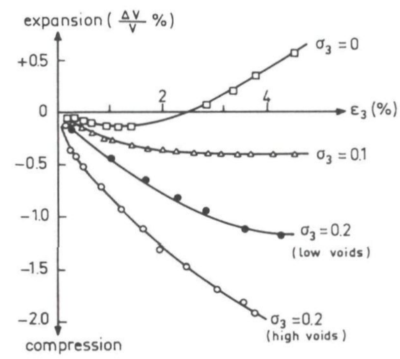


Fig. 3,4

Schönian (paper 5) prefers dense asphaltic concrete in shore protection above mean high seawater level, as it provides a longer lifetime than permeable revetments. A disadvantage is the presence of an up-lift pressure, related to the effect of backpressure of water inside the dike.

Herment et al (paper 6) consider asphaltic concretes (support-, watertight- and joint-layers) with a mineral composition of 45% w crushed stone (4/10 mm), 38% w crushed stone (0/4 mm, including 15% filler), 10% w rolled stones (0/4 mm) and 7% w limestone filler (total 12,3% w of filler).
These types of asphaltic concretes with 83% w of crushed stones are used for the Verney dam in the hydroelectrical project Grand Maison in the France Alps.
The characteristics are given in Table 6.2.

3 types of asphaltic concretes			
	support-layer	watertight-layer	special joining layer
Bitumen content (pha)	6.7	8.2	9.5
Penetration	60/70	60/70	180/220
Void content % vol	5.7	2.1	2.5
Permeability k(m/s)	5.10^{-7}	5.10^{-10}	5.10^{-10}
Retained Stability %	92	-	-

Table 6.2 Characteristics of asphaltic concretes types used in Verney (ref. Herment)

The impermeability of the dam is obtained by bituminous concrete facings with a 10 cm support layer (filter layer, semi-impermeable) and two watertight layers of 6 cm thick each. For the transition zone between the cement concrete beam, and the dam a special joining watertight layer is realized with 9,5 pha bitumen (180/220 pen).
No flow is possible, because in that region the temperature will be fixed at about 10°C.
This layer will be used to protect the cement concrete construction from thermal shocks and will limit the differential dilatation between the cement concrete and the asphaltic concrete.

Duval (paper 9) mentioned the use of a polyester layer between two layers of 6 cm thick asphaltic concrete mixes in front of the dam for reasons of watertightness (the barrier dam in Pla de Soulcem).

Measurement of the permeability of mixes occurs on different ways. Citroni et al. (paper 11) for instance present an apparatus, see Fig 11.3 in which the hydraulic pressure and deformation are registered.

Three types of asphaltic concrete mixes (dense, semi-dense and open) are investigated.
The grading curves of the mineral aggregates are given in Fig. 11,1.

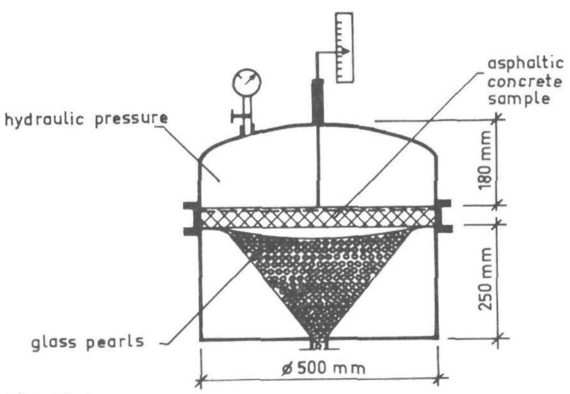


Fig. 11,3

For dense and semi-dense asphaltic concrete with 7.8 resp. 5.8 bitumen (80/100) at about 4 bar pressure and a deformation of 50 mm they measure a permeability coëfficiënt smaller than 10^{-10} m/s.
The open asphaltic concrete mix whith 4 pha bitumen (80/100) has a permeability coëff. of 8.5×10^{-4} m/s.
Normally at a void content of about 3% vol. and at a pressure of 25 bar an asphaltic concrete is impermeable, according to Hoffmann et al. (paper 4) as tested by them in a vacuum (?) apparatus.
Unfortunately no details are given.
More about permeability test methods is also given by Ballie (paper 15) in session II.

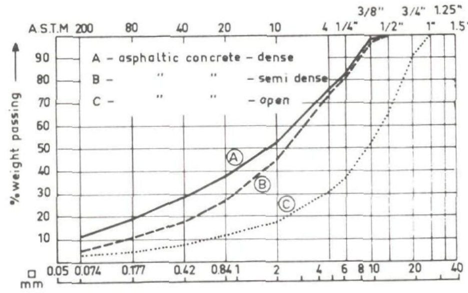


Fig.11,1

Like many others, Post et al (paper 8) is using the therm "bituminous concrete" instead of asphaltic concrete. They mention that permeability tests during and after construction are important, but unfortunately they do not report any results. The bituminous concrete facing has the following advantages:

- flexibility and, due to their visco-elastic behaviour, possibility to follow the differential settlements in the dam and the foundation
- retains its watertightness in severe climates (frost, ice etc.)
- fast construction (short working time)
- easy to maintain and repair

After compaction a permeability coëfficient of 10^{-10} m/s is easily attained, which means semi or fully impervious material. For waterimpermeability a void content of less the 3% vol. is necessary.

The stability and durability is strongly influenced by the percentage of crushed limestone (at least 50%), while addition of rounded sand and limestone filler affect the flexibility and compactibility.
Mostly a 60/70 penetration bitumen is used, bitumen 80/100 and blown bitumen 85/25 are less common. The ageing of the bitumen in the mix is comparable with road-mixes and is mainly caused by the mixing temperature (about 180 °C). The original 60/70 pen bitumen is hardened to a 40/50 pen. However, I believe that at high void contents (c.q. lean sandasphalt) and high bulk temperatures during construction the penetration value can lower much more strongly.
The asphalt concretes mentioned by Duval (paper 9) are a asphaltic concrete (0/12) as the impervious layer and an asphaltic concrete (0/20) as the support layer for the lining of the Pla de Soulcem dam, see Table II.
The properties and behaviour of these mixes, as expressed by density (Duriez-LCPC test), stability (Marshall-test), resistance to compression, flow at a slope and the coëfficient of permeability are given (see Table 9,2).
For these mixes flow stability tests at 70°C and a slope of 1.85 to 1 have also been carried out, which indicate that there is only a very little flow after several days for the dense asphaltic concrete (about4/100 mm).

The author also mentions the use of asphaltic concrete mixes for a 10 m high and 60 cm thick vertical sealing core in the alluvial foundation as an impervious layer.
The extreme forces on asphaltic concrete layers

	asphaltic concrete		
	normal (0/12)	dense (0/20)	Norm C.C.T.P.
Density (1000 kg/m^3)	2.33	2.33	-
Degree of compaction (%)	97.5	97.2	> 97
Resistance to compression at 18°C and 24 hours (bar)	48	50	> 50
Marshall Stability 5 mm à 60°C (da N)	375	365	225
Permeability k (m/s)	3 à 4.10^{-10}	5.10^{-10}	$<10^{-10}$ for (0/12) $10^{-9}<..>10^{-10}$ for (0/20)

Table 9,2 Characteristics for normal (0/12) and dense (0/20) asphaltic concrete (ref. Duval.)

can endanger the impervious nature of the material. Especially at joints, sudden change of slopes and local and differential settlements of subsoil.

Herment (paper 6) therefore reports about the effect of polyester reinforcement of bituminous mixes on the strength properties.
Figures 6,4 and 6,5 present the results for the

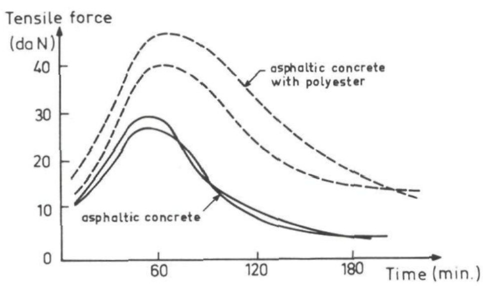
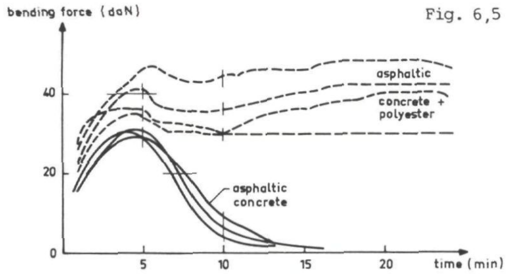


Fig. 6,4

maximum force in a tensile test with an initiated crack and in a simple bending test respectively. The effect of polyester reinforcement is explicitly visible.
Citroni (paper 11) have investigated the effect of a fabric and a non-woven membrane in asphaltic concrete mixes in relation to the shear strength

Fig. 6,5


parallel and perpendicular to the joints. By introducing these types of synthetic layers in between the bituminous mixes the stresses will be diminished. A bitumen membrane and synthetic membranes in general have their disadvantages in relation to small deformations and poor adhesion

to the mixes respectively. Better adhesion to asphaltic mixes is claimed for two particular membranes, viz:
- a fabric polyester [HATELIT]
- a co-polymer (EPDM) membrane [AGAPER-EW2].

The effect of reinforcement of asphaltic concrete by fabric and non-woven membranes is tested in specimens, as shown in Fig. 11,7.

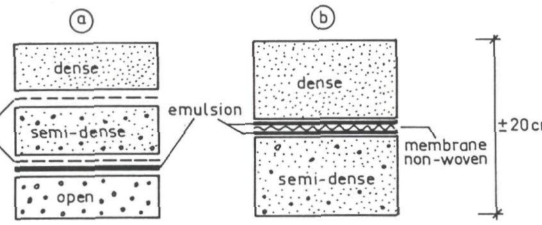


Fig. 11,7

The authors promote a direct shearing test with loading pressures in horizontal and vertical directions.
The shearforces on the specimens being parallel and normal to the direction of the membrane layers.
The principal testresults are presented in Fig. 11,10 for a non-woven membrane with the shearforce parallel and for a two-layer fabric membrane with the shearforce normal to intermediate layers.
It should be noted that the results for homogeneous asphaltic concrete in the first test condition always give higher shearforce results, while in the second case (that means the layer is perpendicular to the shearforce) a positive result is reached. The authors give many testresults ($\tau \div \sigma$) for membranes and fabric rein-

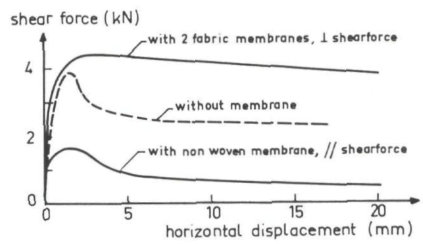


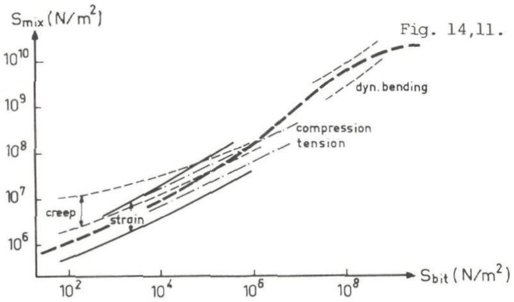
Fig. 11,10

forced asphaltic concretes in shear tests parallel and perpendicular to the layers (shear strength, cohesion and friction values).
Also the effect of an emulsion layer in-between the asphaltic concrete layers is taken into consideration. On the basis of their tests they conclude that the use of 2 layers in-between, dense, semi-dense and open asphaltic concrete will increase the internal friction value, which is partly influenced by the effect of the aggregate of the open asphaltic concrete.

Bandsma et al (paper 14) emphasize that for the structural design of hydraulic bituminous constructions more information is desired about the mechanical properties at different loading conditions.
They stipulate splitting, creep and fatigue test in relation to the ageing, stripping and erosion effect of the materials.

Because in hydraulic applications the loads in time and frequency are different from the stresses in roads, it is important to execute tests under conditions related as close as possible to the circumstances in hydraulic engineering applications.

The authors investigated five different asphaltic concrete mixes, including a mix with partially recycled asphaltic concrete (25% old asphalt). The mix composition is mentioned in Table II. The void contents depend on the degree of compaction and vary between 4 and 6 vol%. The stiffnessmodulus of a mix (S_{mix}) can be expressed by a mastercurve, which gives S_{mix} as a function of the stiffnessmodulus of the bitumen (S_{bit}), and which is dependent on the type of mix composition (mineral, bitumen and void content). The measured stiffness modulus value is stress dependent, which means that the curves S_{mix} versus S_{bit} given by the creeptest will not coincide with the results measured by the tensile and compression straintest.
This is expressed in Fig. 14,11.

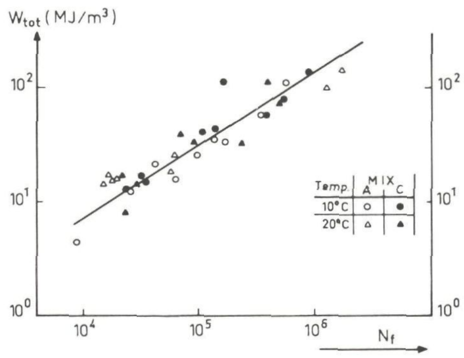


In general however the S_{mix} - S_{bit} relation can be expressed by the overall line with a band, indicating the spread.
S_{bit} is based on the nomograph of van de Poel, which gives the bitumen stiffness modulus as a function of temperature, loading time and bitumen characteristics (softening point and PI).

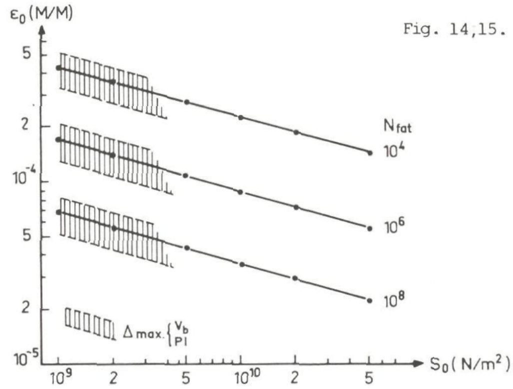
The maximum mix stiffnessmodulus is in the order of 5×10^{10} N/m² while the lowest stiffnessmodulus strongly depends on the bitumen and skeleton composition of the mix at high temperatures.

The fatigue behaviour of the mixes is tested in a 4-point bending test at constant stress amplitude. The loading frequency of 30 Hz in my opinion is too high for most of the hydraulic bituminous engineering constructions, which means that the absolute fatigue results given in this report are to pessimistic (too low).
As previously validated for road mixes, the fatigue results expressed by the fatigue lifetime can also be expressed by the total amount of dissipated energy, see Fig. 14,10.

Fig.14,10.



In general the fatigue lifetime will increase as the stiffnessmodulus decreases. The values of stiffnessmodulus and fatigue for standard road asphaltic mixes are also given by nomographs presented by Shell Francaise. The authors of paper 14 conclude that for the chosen asphaltic concretes (in view of the applying requirements) a variation for S_{mix} in the order of a factor 16 and for the fatigue of approxmately 20% can be given. That latter result is expressed by Fig. 14,15.



The average stiffnessmodulus, determined on the basis of the nomograph is approx. 30% higher than the measured stiffnessmodulus (in the region of 10^{10} N/m²). Also the predicted (with the nomograph) initial strain values for fatigue are within a variation of 10 to 30% equal to the measured strain values.
More attention should be paid to the determination of lower stiffnessmoduli because they are not yet covered by the nomographs.

Van Herpen et al (paper 17) mention that asphaltic concrete can mainly be used above water, because compaction is essential. Preferably the compaction takes place at the highest possible temperature at which the tandem vibrating roller is the most effective to prevent the execution of initial surface cracks. Also deformation of the sub-base can introduce cracks but otherwise the subbase must be compacted well (at least 95% of the maximum Proctor density), otherwise a good compaction of the asphaltic concrete is not possible.

4. Asphalt mastic

This material is mentioned by a lot of different names. Next to asphalt mastic we recognize: sandasphalt mastic, sandmastic, penetration mortar and grouting asphalt mastic. Standardisation of the name will be useful.
The asphalt mastic is a mixture of sand, filler and an excess of bitumen, see Fig. 16,1 and Table III. There is more bitumen available than

Asphalt mastic (%wt.)			
Sand	filler	bitumen (80/100)	author (report)
61 (1)	20 (2)	19	Kerstens (V,2)
60-75 (3) 63-68 (6)	10-20 (4) 12-15 (7)	15-20 (5) 20-22 (8)	Castagnetta (V,10)
68	16.5	15.5 (9)	de Groot (V,15)

(1) : rounded, instead of crushed
(2) : lime grid
(3) : natural or crushed
(4) : normally limestone
(5) : 40/50 pen above water
180/200 pen under water
(6) : riversand 0/3 mm
(7) : limestone or rockasphalt filler
(8) : cationic emulsion
(60% of 40/50 pen)
(9) : 80/100 or 160/210 pen

Table III Composition of asphalt mastics

necessary for filling the voids in the sand and filler mixture. The mix therefore is naturally dense and need not be compacted.
Poured at working temperatures it can be used for asphalt slabs above and under water for lining or as bottom and toe protection, joint filling of stone layers and for prefabricated reinforced mattresses. The material is sand- and watertight. The asphalt mastic will also be used as base material for open stone asphalt.
As flow is the main characteristic requirement under different circumstances of temperature, time and waterpressure, the viscosity behaviour is a very important parameter.
The required asphalt mastic viscosities at various temperature stages mentioned by the Dutch guidelines are presented in Table IV.

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

Table IV. Required asphalt mastic viscosities at various temperature stage (Dutch guidelines 1984)

Kerstens et al (paper 2) mention that, related to the limited flow around stones, the viscosity should be 30 to 80 Pa.s at 140°C. In Italy, Castagnetta (paper 10) mentions some other overfilled mixes with sand, filler and bitumen, called "mastic asphalt", in which a percentage of the graded sand is replaced by more fine sand or filler. In this last case also the percentage of bitumen is increased and can reach 50% (for instance 50 - 60% w limestone filler and 40-50% w bitumen 40/50 pen with 85/40 blown bitumen). Castagnetta also mentions a cold-mixed pourable asphalt mastic. This mix, see Table III, is bound by cationic emulsion (60% of 40/50 pen bitumen) and prepared in a cement mixer with 6 to 10% w of water. It is workable for 15-20 minutes and can be poured in place. The material is grouted in 25 cm thick "Reno"-mattresses, gives complete penetration hardening after 6-7 days, is completely impermeable and shows flow stability at 70°C on a slope of 1 in 2.

De Groot et al (paper 19) explain in their paper that asphalt mastic, called "sand mastic" by them, is strong enough to resist displacements by local currents and or waves. For calculation of the forces on an asphaltic layer, it is necessary to know the hydraulic pressure given by the free waterhead,the wave current loads, the permeabilities of the subsoil and the asphalt mastic. Stability calculations have to be performed and especially the edge of the asphalt mastic slab is a weak point.
Layers could be placed in 40 meter depth of water.

5. Grouting mortar

Grouting mortar is a hot-mix-type consisting of sand, filler and an excess of bitumen (asphalt mastic), with stones (gravel) added and related to the size of stones that has to be grouted, see Table V and Fig. 16,1.

	composition (%wt)				
	stone	sand	filler	bit.	author (report)
grouting mortar	30 (1)	47.5	11.5	11 (2)	de Groot (V,15)
dense stone asphalt	60 (3)	27	6.6	6.4 (2)	de Groot (V,15)

(1) : 6/16 mm, the sizes are depended on the dimensiones of the stones that has to be grouted
(2) : 80/100 or 160/210 pen bitumen
(3) : 20/40 mm

Table V. Composition of grouting mortar and dense stone asphalt.

These mortars are used for grouting stone revetments above and below waterlevel and also for slab construction (in-situ or prefabricated), like mattresses.

Below mean high sea waterlevel in general thick asphalt grouted stone revetments are applied giving greater flexibility, where settlements have to be expected, and these have a greater resistance against normal tidal action, (Schönian, paper 5).
As drainage layer crushed stone penetrated with bitumen emulsion is also used (Castagnetta, paper 10).
In the report of van Herpen et al (paper 17) grouting mortar is formed by the penetration of

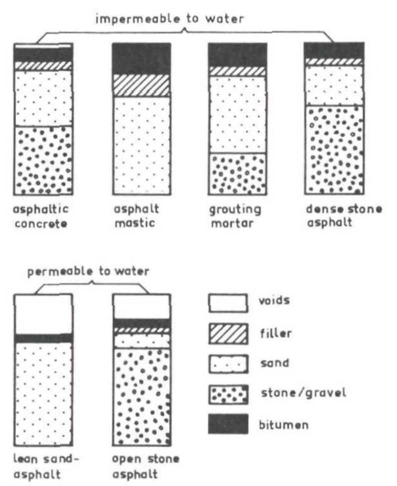


Fig. 16,1.

a layer of stone with an asphaltic mortar.
A complete, or part filling of the voids is possible, see Fig. I.

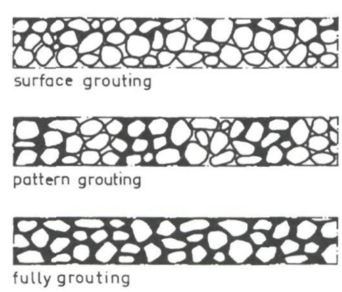


Fig. I.

Temperature of application above water is 100 to 190 °C and under water less than 150°C. The asphaltic mortar has a low viscosity at application temperature.

Penetration depth (ℓ) is given by the relation:

$$\ell = \frac{\text{const. } d^4}{n_o}$$

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

The working viscosity will in general lie between 10 and 100 Pa.s.

The largest grain size of the mineral aggregate in the mixture is related to the dimensions of the stones that has to be grouted.
Adhesion between the asphaltic grout and the stones is not necessary, because the strength is given by the skeleton. Special care during transport is required to prevent segregation of the mix.
Before the grouting mortar is placed, the voids in the stones must be free of silt or sand; especially for under water grouting this can be a problem. The applications are: toe construction, revetments of breakwater, groynes, seawalls and dikes under as well as above tidal zone.

6. Dense stone asphalt

Dense stone asphalt is a gap graded mixture of stone, sand, filler and bitumen, see Fig. 16,1. Mix compostion is given in Table V.
The amount of bitumen slightly overfills the mixture. The material is, therefore, water impermeable with a void ratio of about 3%.
It is used as revetment of the bottom and under water slope protection; also in toe construction.
The ability to form a dense stable revetment by self-compaction depends on the viscosity of the asphalt mortar (asphalt mastic or grouting mortar) and the proportion of stones to asphalt mortar.
No special information is given in the reports.

7. Lean sandasphalt

Lean sandasphalt is a mixture of sand, often locally obtained, with 3 to 5% weight of bitumen. It is a greatly "underfilled" mix with a general voids ratio of 30%, see Table VI and Fig. 16,1.

lean sandasphalt (% wt)			
sand	bitumen (80/100)	voids ratio %	author (report)
96 (1)	4	25 (2) 40 (3)	v. Damme (V, 1)
96.8 (4)	3.2	?	Schönian (V, 5)
95 - 97	3 - 5	?	Mulders (V, 18)

(1): d_{50} between 150 and 300 μm
(2): after compaction
(3): no compaction
(4): medium graded sand (0/2 mm), d_{15} = 0.22 mm, d_{50} = 0.3 mm and d_{60}/d_{10} = 1.5

Table VI Composition of lean sand asphalts

The function of the bitumen is simply to coat the sand grains and to bind them together. After some time the permeability becomes very similar to the sand from which it is made.
It is used as a core material for reclamation bunds, filter layers under open revetments and as a permanent or temporary cover layer above and below waterlevel (breakwaters).
It forms impermeable revetments in terms of waves and permeable in terms of tide.
Terms like: sandasphalt, bituminous sand, sand-bitumen and hydraulic sandasphalt are used.
No filler (such as limestone) is added, like in road sand asphalt. Therefore, to prevent misunderstandings it is preferable to use the term lean sandasphalt or bituminous sand. The mix is usually compacted by the laying equipment itself and constructed in layers from 10 to 200 cm in bulk. Production takes place in batch or drum mixers.

Van Damme et al (paper 1) mention the use of this material as a construction material for Zeebrugge's new outerport.
The use of poorly unspecified, local sands, often found close to the construction side or the plant, may lead to non-uniformity of the mix.
The permeability (10^{-4} to 10^{-5} m/s) depends on the sandcomposition, the degree of compaction and is about a factor 10 lower than the sand itself (4.5×10^{-5}m/s). The void content is 40% for not compacted and 25% for compacted mix.
Retained stability is 70% after 7 days to less than 60% after 28 days.
The stability measurement is based on Marshall tests, but in my view Marschall compaction is not representative for practical application. Probably a "kneading" compaction test is more realistic.
All results confirm that some erosion takes place and that consequently lean sandasphalt surfaces should not be left uncovered when abrasion (wind or wave) action can be foreseen.
Related to the visco-elastic behaviour of the binder lean sandasphalt is characterized by limited deformation and will not collapse like a sandbody.
The material resists waterflows to a maximum of 3 m/s.
More creep and shear test are required.

Schönian states in his paper (5) that permeable revetments of lean sandasphalt is a true three-dimensional filter layer and should be flexible to follow settlements of the subsoil. The material placed on the lean sandasphalt can have any size, as the filter laws are not valid for this case.

But van Damme (paper 1) and Mulders (paper 18) both specifically mention the use of a gravel sandasphalt mix. Composed of 40-45% w gravel (30 mm) and 60-55% w lean sandasphalt, the material is used as a filterlayer on the damcore composed of quarry stone (Zeebrugge) and as breakwater barriers (Hoek van Holland) respectively.
Lean sandasphalt which is sand-tight is not the right material to be used as a filter layer on quarry stones (10-80 cm) because it will fill up the gaps between the blocks.
The gravel in the lean sandasphalt mix, with the right size dimension, will prevent washing out

of the lean sandasphalt.

The filter capacity is measured by Schönian (paper 5) with a lean sandasphalt layer of 15 cm thickness on a silty base and covered by an asphalt mastic grouted rubble layer on a slope of 1 in 3 in a big steel tank. After 10.000 "ships passages" no signs were found of any fines having been washed out of the base or penetrated into the filter layer.
The material is also used as a pervious vertical breaking zone in the Wehebach Dam near Aachen (Germany) to improve the filter stability and the earthquake resistance, see Fig. 5,1.

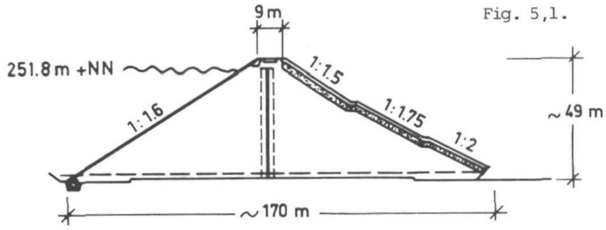


Fig. 5,1.

The material could be placed in layers of 1 m thick with a uniform void content over the full thickness.
Adding of filler and bitumen was necessary to attain the permeability of 10^{-5} m/s maximum.
The material was compacted by a vibration roller with an over thickness of 20 cm to attain a final layer of 100 cm.

The erosion resistance of lean sandasphalt was tested by Mulders et al (paper 18) in a flume, being the closest approach to reality.

No significant erosion was detected on a slope of 1 in 3 at 4 m/s water current velocity after 4 days testing.
The results were confirmed by practical experience in Belgium and South Africa.
The filter properties were nearly equal to that of the constituent sand at a bitumen content of 3-4%, as meassured by the tidal movement inside lean sandasphalt below mean sea level, see Fig. 18,6.

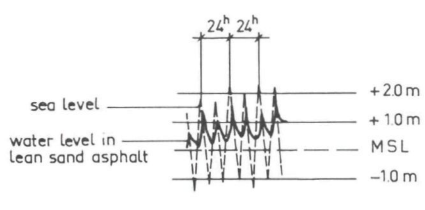


Fig. 18,6

The deformation resistance was tested in a field test as given by Fig. 18,11. The mechanical strength is partly attributable to the grain to grain contact and partly to the viscous flow between the coating joints.

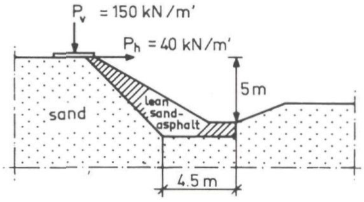


Fig. 18,11

The results are given in Fig. 18,13 which shows that the horizontal displacement is low, at least lower than the calculated values based on the material properties as found in laboratory tests.
This probably as a result of the higher shear strength of the sandasphalt related to the ageing of the binder. Since there is a slow temperature-drop after an application temperature of about 80-110°C, the angle of internal friction is 42° and the effective dynamic viscosity is 10^8k Pa.s at 40°C.
While a lot of experience has been gained with lean sandasphalt a lack of fundamental knowledge of these bonded filters still exists.

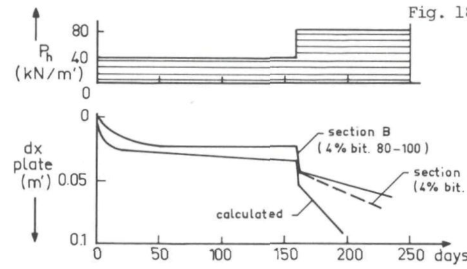


Fig. 18,13.

8. Open stone asphalt and asphaltic mattresses

Open stone asphalt is a gap-graded mixture of asphalt mastic (20% w) and stones (80% w), frequently limestone 20/40 mm. The mastic binder only coats and connects the mineral particles together, see Table VII and Fig. 16,1.
It is an "underfilled" mix with an air voids ratio of about 20 to 30% vol. By its open structure the open stone asphalt should not be placed under water, except in the form of prefabricated mattresses.

	composition (% wt)		
	crushed stone (20/40)	asphalt mastic	author (report)
open stone asphalt	81 (1)	19	Kerstens (V,2)
"Fixtone"	80	20	de Groot (V,15) v. Herpen (V,17)

(1): limestone 20/32 mm gap-graded

Table VII Composition of open stone asphalts

It is used in revetments in tidal zones and placed in-situ above water (related to the application temperature of about 130°C) and in bottom and under water slope (toe) protection as prefabricated and reinforced mattresses.

The material is characterized by good flexibility and flow resistances up to 5,2 m/s. It can be applied in- situ without joints because the application temperature (110-125°C) guarantees a good contact with the (old) layer. Special attention should be given to the de-mixing of the mix during transport and storage. The open structure affords an opportunity to cover the asphaltic structure with a layer of soil (at least 3 cm) for growing grass or plants (green dikes), see Kerstens et al. (paper 2) for the inundation dike-revetments near Antwerp.
Thanks to its high air voids volume the permeability ($k \approx 10^{-2}$ m/s) of open stone asphalt, like "Fixtone", is comparable to that of coarse gravel. Due to this high permeability the material is not sandtight, therefore a proper filter is essential between a subbase of sand and the

open stone asphalt revetment.
The mixing is done in 2 phases, first the mastic mixing and afterwards crushed aggregate is added and coated by the homogeneous mortar.

Asphaltic mattresses are prefabricated reinforced structures, filled with mastic, open stone asphalt or asphaltic concrete. Depending on the amount of asphalt mastic and stone penetration such mattresses are waterproof or porous.

One speaks of asphalt mats for thicknesses up to about 10 cm, above that size the term asphaltic mattress is used.

Castagnetta (paper 10) mentions the "Reno mattresses", which are composed of steel wire-netting, filled with crushed stone or gravel, grouted and sealed with asphalt mastic.
This type of material can be prefabricated or fabricated in-situ above and under water (to a maximum depth of 2 m). In Italy a mattress with improved flexibility, greater resistance and sufficient weight is fabricated. The so-called "Sarmac" mattress of 50 cm thick is used for protection of underwater pipelines against anchors.

De Groot et al (paper 19) show the "Fixtone" mattress, which is a prefabricated mattress of open stone asphalt in combination with a filtercloth geotextile that will be used on the seabed as a permeable sandtight protection layer. For the application of this bottom protection, using mattresses of 200 m length and 17 m wide in the Oosterschelde, a special ship was used.

9. Bitumen membranes

Reinforced bituminous membranes as an impervious systeem, are mostly used in combination with a support layer (which has both a mechanical and a drainage function) and a protection layer. They are executed in-situ or prefabricated, as reinforced thin (3-6 mm) watertight layer.

Domange et al. (paper 7) mention reinforced bituminous geomembranes, composed of a non-woven polyester geotextile, coated with fillerised blown bitumen (100/40) or elastomer bitumen (Type: Coletanche).
The geotextile provides a high tensile strength in combination with a high elongation value. The bitumen coating provides the imperviousness also after deformation and punching, see Table 7,1.

Herment et al (paper 6) successfully used this type of membrane as an impervious layer over morainic earth layer with 0/125 mm stones, whithout damage by punching effect.
The permeability coefficient is in the order of 10^{-13} to 10^{-14} m/s depending on the pressure (10 to 5 bars). The "Cemagref" test gives an indication about the resistance of the membrane to punching by measuring the permeability.
Their geomembrane [Hatelit] is tested at a maximum pressure of 0.8 MPa during 40 days.
The report gives examples of applications in irrigation canals, waterstorage pools, dams, dikes and waterproofing for pollution control.

Castagnetta (paper 10) mentions the in-situ preparation of membranes for protective revetment in canal linings and dam facings or to re-

COLETANCHE					
Type	NTP1	NTP2	NTP3	NTP4	NTPES
Thickness (mm)	3.1	3.9	4.8	5.6	4.0
Weight (kg/m^2)	3.6	4.5	5.5	6.5	4.0
Tensile strength* (da N/cm)	14	17	21	24	21
Elongation* (%)	35	35	37	47	40

*Test NF G07001 MOD: 20°C-100 mm/min-Sample 50x200 mm

Table 7,1 Characteristics of reinforced bituminous geomembranes, Coletanche (ref. Domange)

They use a supportbed of quartzite (20/40 mm) and waterpressures of more than 10 bars.

Citroni et al (paper 11) also mention such equipment, developed by their research institute (CRIS), see Fig. 11,5.

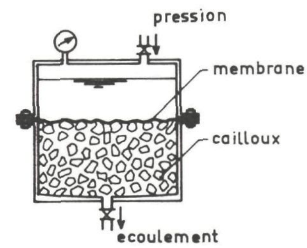
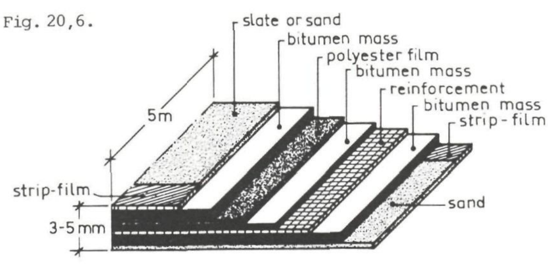


Fig. 11,5.

store the impermeability of old revetments. The membrane is composed of a tackcoat of hot 40/50 pen bitumen, non-woven geotextile, hot 40/50 pen bitumen and fine sand or limestone filler.
In place of the hot 40/50 pen bitumen also a cold-sprayed bitumen emulsion is succesfully used.
Hoekstra et al (paper 20) give the results of fabric reinforced bitumen membranes for water-storage and ground water protection.
This membrane, type Hypofors, see Fig. 20,6, is a 5 m wide fabric-reinforced bitumen membrane with an almost uniform strength per unit length (about 60 KN/m') and a high elongation at failure (~ 30%) through the use of nylon.
The bitumen is blended with polymers.
To provide sufficient flexibility for the irregularities of the subsoil together with the required waterproofness, the bitumen should be soft enough.
Protection against micro-organisms is provided by the polyester film and the resistance to the climate (UV light, rain etc.) is improved by covering the surface with crushed slate.
For the application in lined waste storage pits, its chemical resistance is explicitly tested for different saturated hydrocarbons and related compounds.

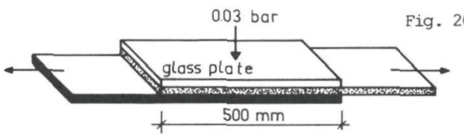
Fig. 20,6.

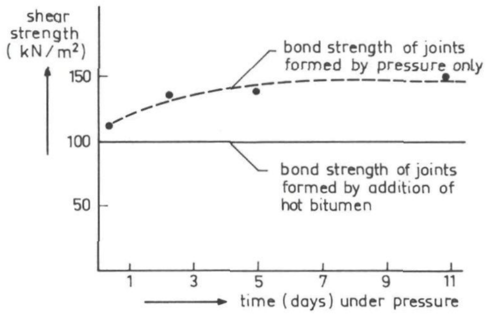


Glueing the joints they prefer hot blown bitumen (with slightly lower viscosity then that used in the coating process), because due to the lower shear strength in the joint, it reduces the risk of sliding planes inside one of the sheets rather than between adjacent sheets, see Fig. 20,7.

To measure the resistance to tear propagation the trapezoidal test method, instead the ASTM D 1004-70 method recommended for plastic films, is preferred since it strains the yarns parallel to the direction of testing.

Fig. 20,7.





The characteristic values for the force and the strain at failure and the tear propagation are given in Table 20,1.

	Hypofors		
Type	NF 1000	NF 3000	CF 7000
Force at failure (kN/m^1)	52	32	12
Strain at failure (%)	18	18	45
Tear propagation (N)	850	450	210

Table 20,1 Characteristic properties of Hypofors bitumen membranes (ref. Hoekstra)

10. Bitumen

All types of bitumen (like straight run and blown bitumen) and modified bitumens with fillers or synthetic polymers are used for binding the mineral aggregates (application temperatures from 150 to 200°C).
The effect on penetration, softening point and Fraass breaking point of the bitumen by blending with hardening- and elastomer additives is given by Castagnetta (paper 10) and presented in Fig. 10,5. In contradiction to the general idea, we recognize here an increase in the negative value of the Fraass breaking point by adding elastomer additives.

Obviously some hardening of the bitumen binder occurs during the production process. The ageing-index, as given by the ratio of the viscosities of the recovered bitumen and the original bitumen is a function of time and temperature. The decrease in penetration during the lifetime of the mix is also strongly dependent on the bitumen content (thickness of the bitumen film on the stones) and the air voids

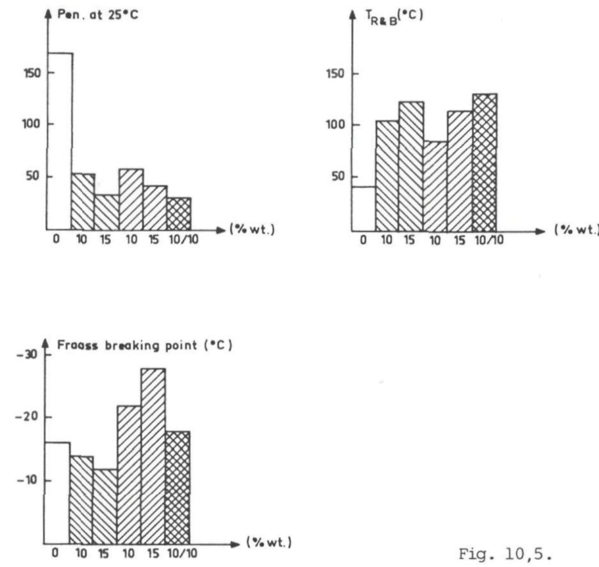


Fig. 10,5.

ratio. It is regrettable that none of the authors in this session presents some data on this aspect for bituminous mixes in hydraulic applications. More results about hardening of bitumen is also given in session I, see paper 15 of van Gooswilligen.

11. Recycling

Re-use of old bituminous mixes is widely accepted as a reliable technique in road construction.
De Groot et al (paper 15) mention that in the Netherlands two hot-mix recycling systems have been fully tested and approved in practice and by laboratory tests (based on Marshall, creep, wheeltracking, dynamic bending for stiffness-modulus and fatigue testing) for hydraulic bituminous mixes.

They present:
- a 100%-recycling proces ("Renofalt-proces"), based on steaming of lumps of broken asphalt mixes, drying, indirect heating to about 160°C, batch-feeded into a pug-mill and mixed with a "rejuvenating" oil.
- partial-recycling in a modified, conventional batch mixing-plant, using crushed, wet, and old asphaltic concrete (0/40 mm) plus overheated mineral aggregate (sand and stones at 275°C).
  After homogenizing, heat-transfer and venting of steam the filler and (soft) bitumen are added.

The authors give a detailed survey of the technological possibilities of hot-mix recycling, which is summarized by Table 15,1.
Special care must be taken to ensure that the input material is not polluted with different sorts of mixes.

Examples of partial recycling:
- "soft" asphalt bags, made of synthetic fibres, 25 ton per bag (72% old asphaltic concrete, 28% heated crushed stones 20/40 mm and 2,5% bitumen 160/210 pen) for protection of the huge concrete piles of the Eastern Scheldt flood barrage against possible damage by large stones.

	Recycled/converted into:				
original mix ↓	asphaltic concrete	grouting mortar	dense stone asphalt	lean sand asphalt	open stone asphalt
asphaltic concrete	yes	yes	yes	-	-
grouting mortar rubble	yes	yes	yes	-	-
dense stone asphalt	yes	yes	yes	-	-
lean sand asphalt	yes	yes	yes	yes	yes
open stone asphalt	yes	yes	yes	-	-

Table 15,1 Hot-mix recycling possibilities
(ref.de Groot)

- reconstruction of an asphalt dike ("Flaauwe Werk") with 25% old asphaltic concrete 0/40 mm. The old 15 pen bitumen is softened to a 80/100 pen bitumen by adding 160/210 pen bitumen. The mechanical properties (creep, strength and fatigue) for this partial recycled asphaltic concrete were comparable with virgin mixes, as given in the paper (14) of Bandsma et al. More information about recycling and modification of the binders are given in te papers of session III and IV respectively.

12. Economics and energy

Arnevik et al (paper 13) show in their contribution that the use of asphaltic concrete for the impervious core in a large rockfill dam, Storvatn dam (Norway), has economical benefits compared to moraine clay as core material, in terms of:
- transports cost of the moraine clay deposit
- asphaltic concrete is far less susceptible to bad weather conditions and
- decrease in the guantity of materials.

The Storvatn dam belongs to the largest hydroelectric power (Ulla-Førre) project in Norway (2000 MW, more than 100 km access tunnels and 50 km of roads for internal transport of materials).
Asphalt paving of the internal roads decreases the maintenance, transportation and vehicle costs.
The maintenance costs for asphalt paving are considerably lower than those for gravel roads. The figures differ from the results given by Norwegian State Highway Authorities, because the amount of heavy lorries and the total daily traffic are higher. The maintenance costs for asphalt are mainly patching, while the costs for gravel are grading and dust prevention. Due to asphalt paving, the vehicle speed and thus the transportation capacity can be increased.

Figure 13,3 shows the difference in road maintenance costs for asphalt and gravel as wearing courses in relation to the transported quantities.

Relatively new is the application of asphalt flooring in watertunnels to reduce energy loss due to lower friction and less turbine damage. Solvik et al (paper 12) describe the structural design and application of asphalt flooring. The hydraulic roughness of an asphalt lining is

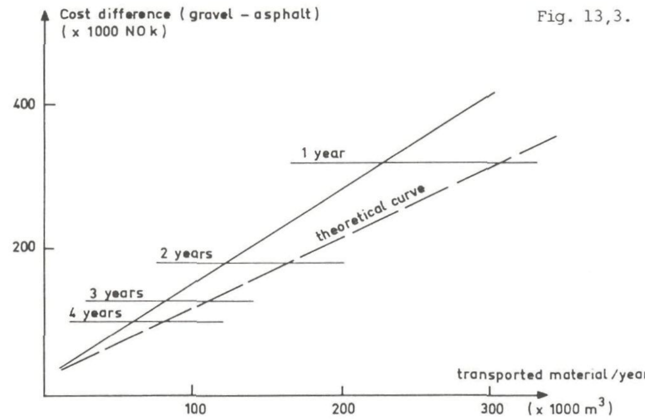


Fig. 13,3.

given by a so-called Manning-number, which is related to the sand-roughness.
The reduction in loss per km length, which is related to the discharge (volume per sec.), shows the possitive effect of asphalt lining.
In the construction special attention must be paid to evacuation openings, narrow enough to prevent indirectly erosion of the base materials. The mix is a standard asphaltic concrete with well graded sand-gravel with d_{max} = 11 mm and 6% soft bitumen (180 to 370 pen). The air void content must not exceed 5% vol. Compaction of the mix is necessary. A relation for the up-lift pressure, based on the surface roughness and the water velocity, gives the criterion for calculation of the asphalt lining thickness.

Chairman, ladies and gentlemen,

I have taken the opportumity to paint with bituminous mixes.
Finally, I would like to thank all the authors for their interesting contributions.
The material for the hydraulic engineering structures are now characterised, so it is time for the designer Prof. Agema.
Thank you.

Remark 1: due to the lack of time for presentation (30 minutes), only a part of this report is presented by the author.

Remark 2: the author has used information given by the publication "the use of asphalt in hydraulic engineering". Rykswaterstaat communications nr. 37/1985.

SESSION V

MODERATOR: Prof. Ir. J.F. Agema

As mentioned before, asphalt mixes behave as a viscoelastic material. As a consequence it deforms under slow changing loads and acts as a firm material in case the loads have a short duration. Those specific properties make that bituminous mixes are suitable for application in hydraulic engineering, in ocean as well as in inlandwater conditions.

1. INTRODUCTION

Taking into account these properties, asphalt mixes are able to fulfill the following functions:

- Protection of core material (rock, earth, sand etc.) and soils (sand, clay), against erosion and scour, caused by waves and currents. Permeable and impermeable bituminous materials are applied. (Fig.1).
- Economization of the structural design by reduction of groundwater flow (-levels). Generally impermeable asphaltic materials are applied. (Fig.2).
- Prevention of groundwater regimes and groundwater polution. Impermeable bituminous materials have to be used. (Fig.3).
- Avoidance of waterlosses (reservoirs, canals etc.) also by impermeable materials.
- Filtering to avoid losses of finer materials (sand etc.). Permeable materials fulfill this function. (Fig.4).
- Reduction of waterheadlosses in tunnels. By flooring with aspahlt mixes with a smooth surface in order to increase the power production.

Fig. 3 Prevention

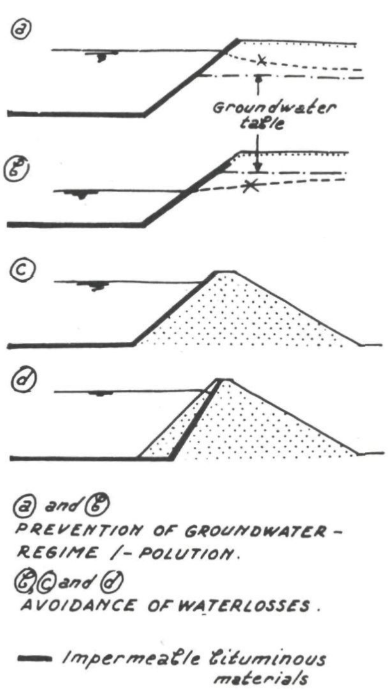


Fig. 1 Protection by bituminous mixes.

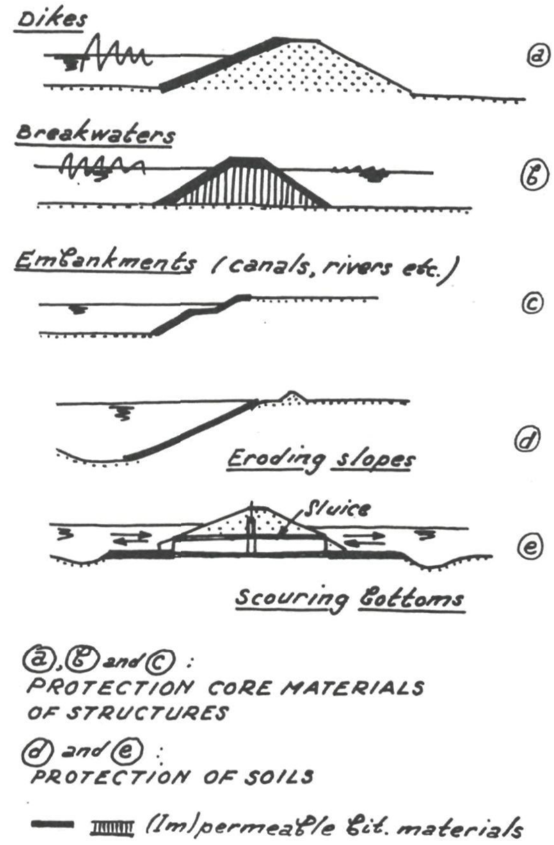


Fig. 2 Economization

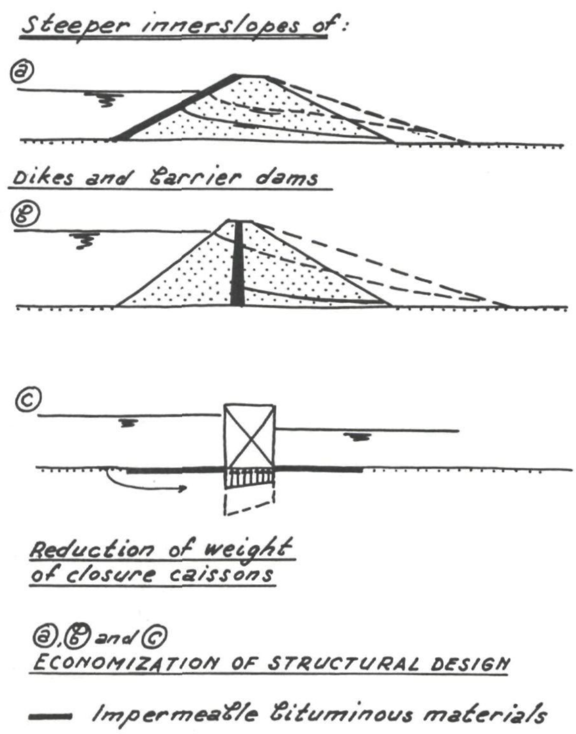
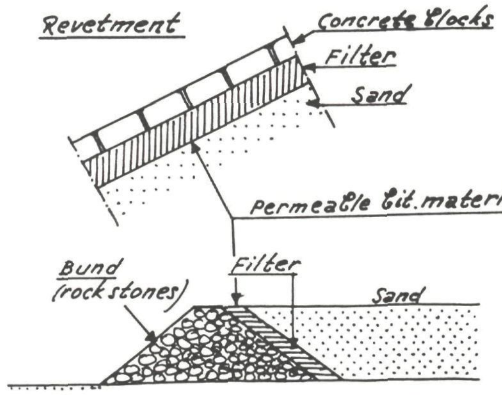


Fig. 4 Filtering

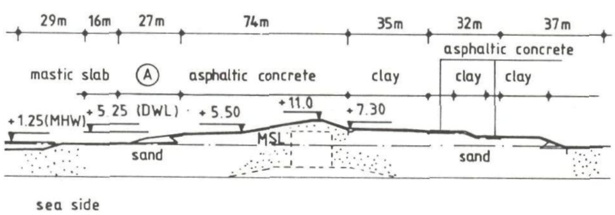


FILTERING TO AVOID LOSSES OF FINE MATERIAL (SAND ETC).

2. APPLICATIONS

Now we will consider hydraulic structures and the way how the functions play a roll in the design.
Seadikes, dams and seawalls have to withstand, special under extreme conditions, highwater levels combined with dynamic wave loading and wave run-up and also watercurrents. One of the barrierdams of the Delta scheme is the Brouwersdam (1971). (Fig.5), a structure with an outerberm, which provides safety to the south-west part of The Netherlands, against the Northsea (Agema, paper 16).

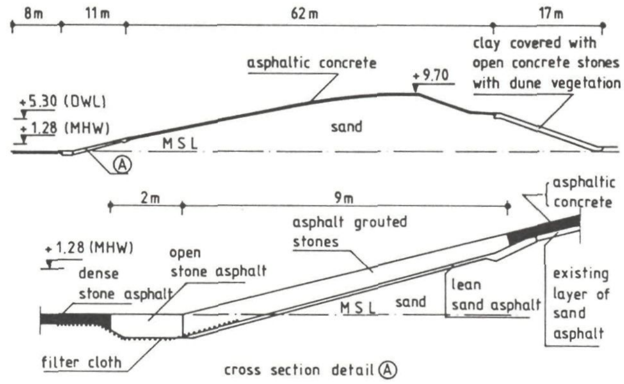
FIGURE 5 BROUWERSDAM, 1971



The core of this dam consists of sand with a bund of minestone at the toe. Those materials are protected by an asphaltic concrete revetment on the berm, upper-slopes and crest and a layer of stones grouted by asphalt mortar on the lower slope. Wave loads on the outside and water pressures under the asphaltic layer due to tidal- and wave movements, determine the dimensioning.
An insitu executed mastic slab in front of the dam prevents the dam against damage. It protects the flat bottom against erosion and scour caused by currents and wave action.
Those impermeable protection layers are also reducing groundwater movements. The wired baskets (gabions) with gravel, just in front of the toe of the dam, are acting as a filter in order to reduce water pressures under the slope and berm protection. After nearly 15 years, the bituminous materials have proved to be stable and of good quality.

A more recent designed and constructed (1984) example is the seadike " 't Flaauwe Werk" (Fig.6) along the Northsea coast on the island of Goeree (south west of The Netherlands).

FIGURE 6 SEA DIKE " 'T FLAAUWE WERK", 1984



The protection of the sandcore and sandy bottom is in principle similar to the Brouwersdam. Instead of a gabion gravel filter at the toe, an open stone asphalt on filtercloth is applied. The bottom protection consists here of dense stone asphalt.
Interesting are overflow dikes along the rivers Scheldt and Rupel in Belgium, as a part of the so-called Sigma scheme as mentioned by Kerstens in paper 2 (Fig.7).

Under normal tidal and discharge conditions those dikes are just high enough to protect the adjacent polders. During stormsurges they overflow and inundate the polders, resulting in reduction of high water levels. This is of importance to the "normal" dike-sections along the rivers.

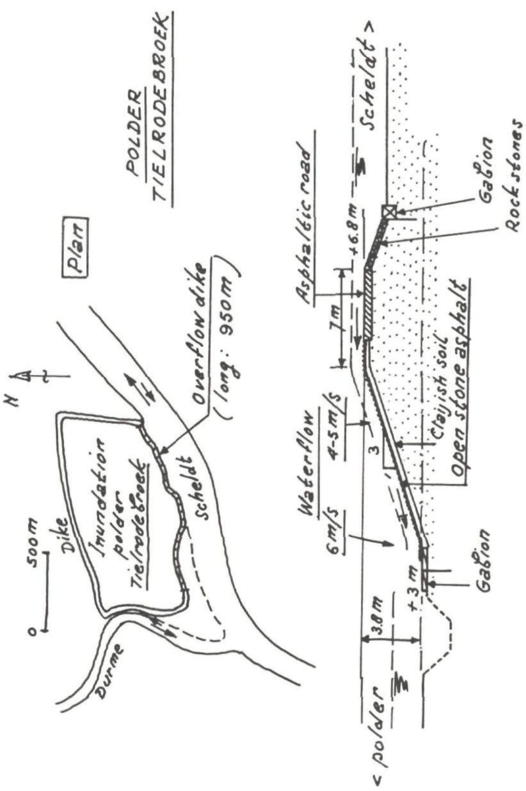
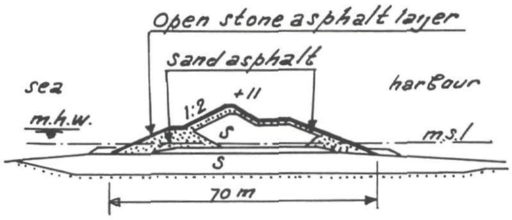


Fig. 7 Inundation and cross-section overflow dike at Tielrodebroek

During overflow, the 1 in 3 innerslope has to be stable against high water velocities of 4 to 6 m/s at the foot. Therefore, the clayish subsoil of the innerslope is protected by a continious layer of open stone asphalt. The resistance of this mix against high water currents has been investigated in a hydraulic model.
Asphalt mastic stone revetments and double layers of asphaltic concrete are applied also in Italy on slopes of seadikes (Podelta) and seawaals.

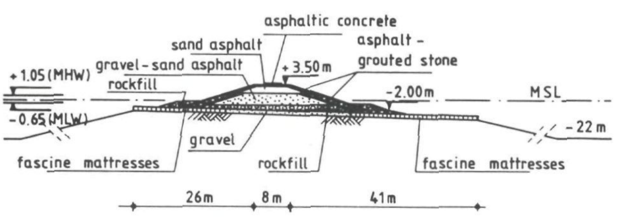
Breakwaters, groynes and simular seastructures are generally exposed to severe wave attack and have to be stable against tidal currents. Asphalt mixes are used as armourlayer and in the core. For the breakwaters of the working harbour of Zeebrugge (Belgium) as mentioned by Van Damme in his paper asphalt mixes are used (Fig.8).

Fig. 8 Breakwater (working harbour) at Zeebrugge



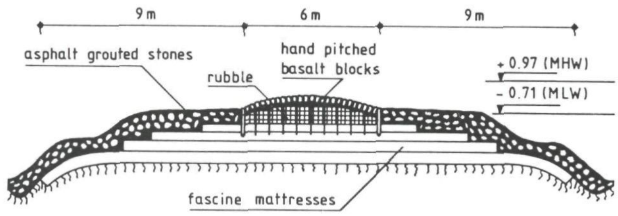
The northern breakwater of this harbour consists of a sand core, shouldered by lean sand asphalt bunds to the high water level. The upperpart is covered by a sandasphalt layer. The sandasphalt slopes (1:2) and crest are protected by an open stone asphalt lining.
The design of the western breakwater differs slightly. Due to its smaller width the whole core consists of lean sand asphalt.
The low crested deviding breakwater of the harbour Rotterdam-Europoort, the so-called Splitsingsdam (Fig.9), has also a core of lean (gravel) sand asphalt.

FIGURE 9 SPLITSINGSDAM, ROTTERDAM - EUROPOORT



The use of this material, which in this case was stable against waves and currents during construction, has proved to be successful. The slopes of the core are protected by asphalt grouted rock stones; the crest is covered by a layer of asphaltic concrete. Groynes in the tidal and wave zone of sandy beaches are important structures for coastal defences.
Stone revetments of existing groynes near Scheveningen and recent constructed stone groynes on the North Sea coast of the island of Texel (The Nehterlands) are grouted with asphalt (Fig.10).
The experience, since 1938 with this grouted groynes, is very satisfactory.
In order to assure the stability of dikes, dams and embankments, in many cases the under water slope and

FIGURE 10 NORTH SEA COAST GROYNE NEAR SCHEVENINGEN, 1938



bottom of the sandy subsoil have to be protected against erosion by currents and waves.
In Italy (Castagnetta, paper 10) Reno matresses with crushed stones, grouted with asphalt mastic underwater, to a maximum depth of about 2 m, are applied. With special grounding equipment the same matresses are used to greater depths (Fig.11).
Along the breakwaters at the entrance of the lock of the Eastern Scheldt stormsurge barrier, the underwater slope has a protection of prefabricated open stone asphalt matresses, laid by special floating equipment.

In order to prevent the bottom against erosion and scour due to increasing currents during closing of tidal channels, a protection is needed at both sides of the sill. The protection in the closure gap of the Brouwershavensche Gat consists of an in situ constructed asphalt mastic slab laid in overlapping strips (Fig.12).
In the Eastern Scheldt openstone asphalt matresses and sandmastic in situ have been applied at a large scale as bottom protection on both sides of the

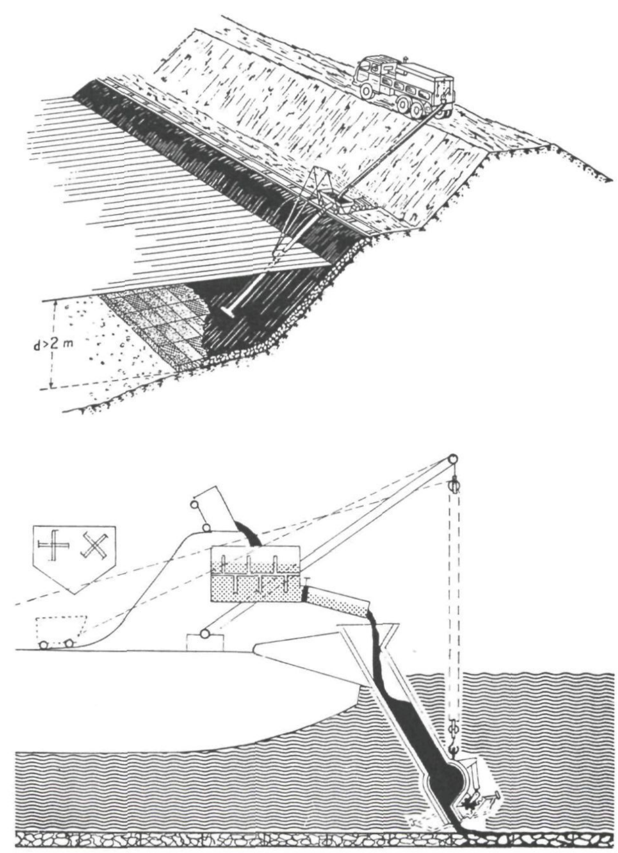
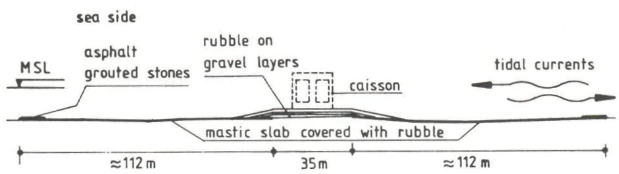


Fig.11 - Reno mattresses grouted with hot asphalt mastic under water at more than 2 m depth.

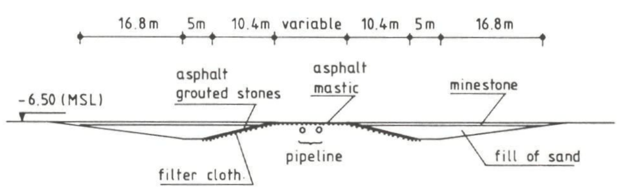
FIGURE 12 BROUWERSDAM, BOTTOM PROTECTION OF CLOSURE GAP, 1971



Stormsurge barrier. Special attention have been paid to problems related to the sandmastic bed protection. In recent years a number of loading situations have been studied intensively. Not only the hydraulic loading has been investigated, but also the response of the sand mastic. Prototype investigations have been carried out. Also calculation models have been developed to represent the behaviour of water, subsoil and sand mastic, in case the pressure difference over the impermeable layer, needs its own weight.

Castagnetta mentioned that for the protection and ballasting of pipe lines under water in Italy a particular type of prefabricated (Sarmac) matress has been developed. This matress is thicker and more flexible than the already mentioned Renomatress. Before placing, Sarmac matresses are grouted with asphalt mastic. Main applications are a gaspipeline between Algeria and Italy and an oilpipeline offshore Scotland.
A particular protection of a pipeline crossing, has been applied in the Hartel canal for inland navigation, Rotterdam-Europoort harbour area (Fig.13).

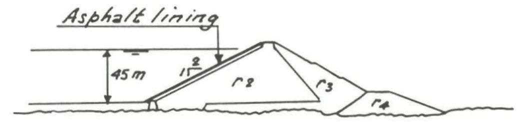
FIGURE 13 PROTECTION OF PIPELINE CROSSING IN THE HARTELKANAAL,1981



In this case the pipeline had to be protected against shipanchoring. The protection consists of asphalt-grouted stones and asphalt mastic on filtercloth.
Due to high (water) heads, the watertightness of earth and rockfill barrier dams is very important for the stability of those structures and the prevention of waterlosses. Lining, intermediate membranes and core-sealings of bituminous materials fulfill the requirements.
In France, Post in his paper 8, mentioned twenty years of experience with asphalt concrete lining on five earth/rockfill dams, to a height of 66m, has been gained (Fig. 14). Advantages of this type of lining proved to be:
- flexibility, also in relation to settlements
- stability by rapid fluctuation of the reservoir-level over a large range
- retaining of watertightness in severe climates
- short construction time
- easy to maintain and repair.

Now bituminous concrete lining in France is a common design concept (Duval, paper 9). However, it has its limitation to damheights of about 100 m.

			Asphalt-Lining			
Barrier dam	type	hight (m)	Δ'	ℓ (m)	L (m)	Surface (m2)
Sainte Cécile d'Andorge '64-'67	rock	45	1/1,7	62	155	8.000
Vallon Dol '70-'72	rock	45	1/2 1/3	87 123	270 1470	16.000 + 184.000
Kruth-Wildenstein '64+'74-'75	earth	38	1/1,5	68	250	13.000
Pla de Soulcem '81-'83	earth	66	1/1,85	142	275	22.200
Le Verney '81-'83	earth	42	1/2	88	430	35.000



Barrier dam Vallon Dol
(Cross-section)

Fig.14 Twenty years experience with asphalt.concrete lining of barrierdams

Special attention has to be paid on the connection of the lining with the rock bottom etc. (Fig.15).
The watertight sealing of the asphalt concrete facing of the Pla de Soulcem rockfill dam (France) with the rocks bottom has been established by a vertical wall of asphalt concrete with a height of 10 m, on two horizontal layers of the same material.
The joint of the facing with the rockslope has been realized by an intermediate concrete structure.
In Italy earth fill dams are lined with asphalt concrete and asphalt grouted, impervious Reno matresses.
Two types are distinguished:
- an impervious layer on one course
- an impervious layer on two courses.

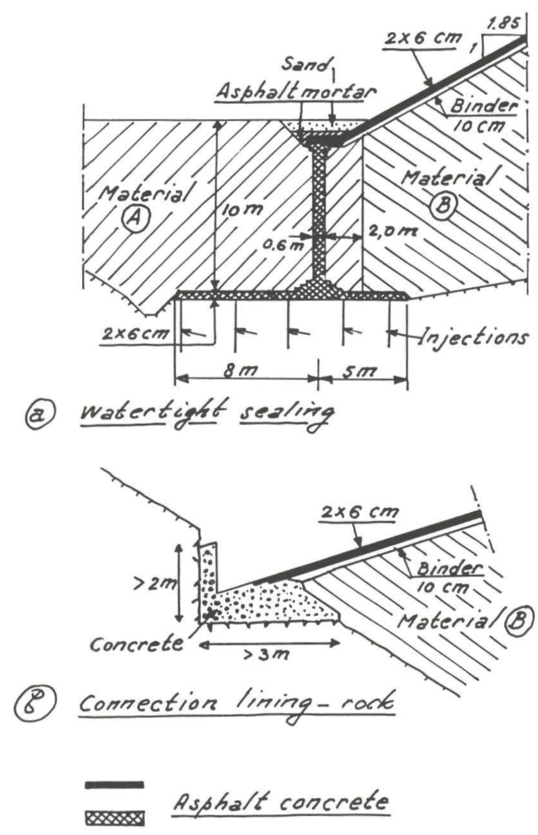
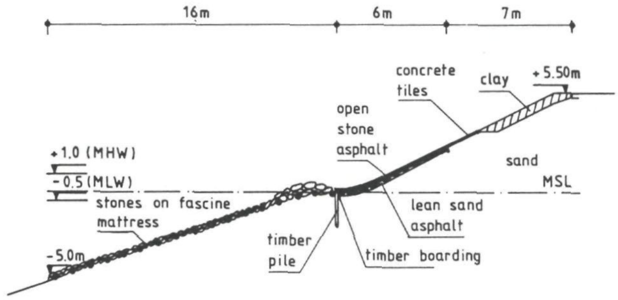


Fig. 15 BARRIER DAM PLA DE SOULCEM

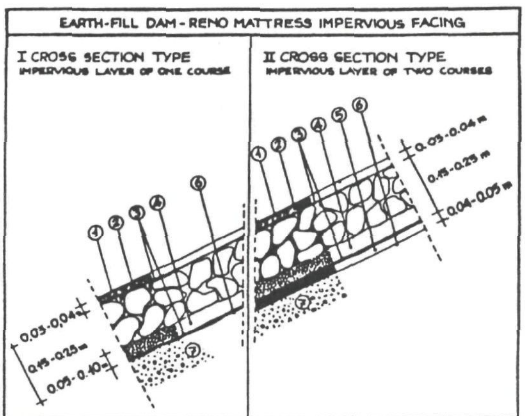
The composition of this Reno mattrasses impervious facings is presented in fig.16.

Embankmets of canals, harbour basins, rivers, etc. are exposed to waves caused by wind, ships and currents, including those generated by ships. Several types of already mentioned permeable and impermeable asphalt mixes are applied as a protection layer. An example is the revetment of a harbour basin (Dintelhaven, Europoort), which consists of open stone asphalt on a layer of lean sand asphalt (Fig.17). This permeable revetment prevents developing of excess water pressures.

FIGURE 17 REVETMENT DINTELHAVEN, 1976



In order to avoid waterlosses of reservoirs and canals through pervious soils, bottom and slopes have to be lined with impervious materials. The same type of measures have to be taken to prevent groundwater regimes in the surrounding of reservoirs and canals. This holds in situations that the reservoir water level is higher than the groundwater table as



Impervious layer:
1 - Bituminous aluminium paint reflective surface coat
2 - Sand asphalt mastic seal coat
3 - 0.15-0.25 m thick Reno mattress (mesh type 5 x 7, wire Ø 2.0 mm) filled with crushed stone or shingle (70 - 100 mm size), grouted and sealed with hot-poured sand asphalt mastic.

Drainage layer:
4 - Crushed stone or gravel (10 - 30 mm size).

Separation layer:
5 - Sand asphalt mastic.

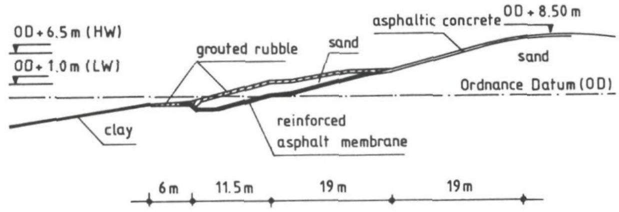
Base course:
6 - Penetration treatment with hot bitumen or cold bitumen emulsion.

Dam or embankment:
7 - Alluvial or quarry materials.

Fig.16 Waterproofing and protective revetment with asphalt grouted Reno mattresses.

well as in cases the reservoir water level is lower than the groundwater table (Fig.18).

FIGURE 18 WATER RESERVOIR, BIESBOSCH, 1973



A very important feature is the prevention of ground and groundwater by poluted water or solid and liquid waste.

Some examples in this field will be considered. In order to secure sufficient supply of drinking water, three reservoirs in the Biesbosch area (The Netherlands) have been constructed. On the sandy bottom a partly permeable silt layer and a waterproof asphaltic lining on the dikeslopes have been applied.

At the lower part the lining consists of a fabric-reinforced bitumen membrane (Hypofors). This membrane is ballasted by 2 m of sand in order to withstand water pressures under the lining. These pressures appear when the reservoir level drops quickly sothat the groundwater level in the dikecore cannot follow. The upperpart is lined with asphalt concrete.

An other recent application of waterproof lining with Hypofors concerns the construction of a pit in a polder, adjacent to the North Sea Canal (The Netherlands), to store solid waste (Fig.19). The Hypofors lining will prevent contaminiation of the groundwater. After construction and in the first stage of tipping waste, there is a need to prevent seepage of groundwater into the pit. A layer of sand prevents than uplifting of the lining.

In Italy (Castagnetta, paper 10) the Milan-Cremona-Po navigation canal is lined with a double layer of asphaltic concrete. The Piave derivation canal has an impervious lining consisting of prefabricated asphalt mattresses.

Having given different applications of bituminous materials in hydraulic engineering, design aspects of impermeable revetments on seadikes will be considered.

3. DIMENSIONING ON WAVE IMPACT.

As plate type revetments, for instance asphalt concrete, on seadikes and simular structures are loaded by waves, the design method is important (v.Herpen, paper 17). The largest forces that waves can exert, are impacts caused by plunging breakers. By schematizing the revetment as an infinitely long plate lying on a viscoelastic subgrade and a step-line loading, the deflection of the plate can be described by a differential equation. From this equation the formula for the thickness (h) of the revetment can be derived, namely:

$$h = 0.75 \times j \sqrt[5]{\frac{27}{16} \left(\frac{P_s}{\sigma_b}\right)^4 \frac{S}{(1-\nu^2)c}}$$

Fig. 19. Disposal area with HYPOFORS lining.

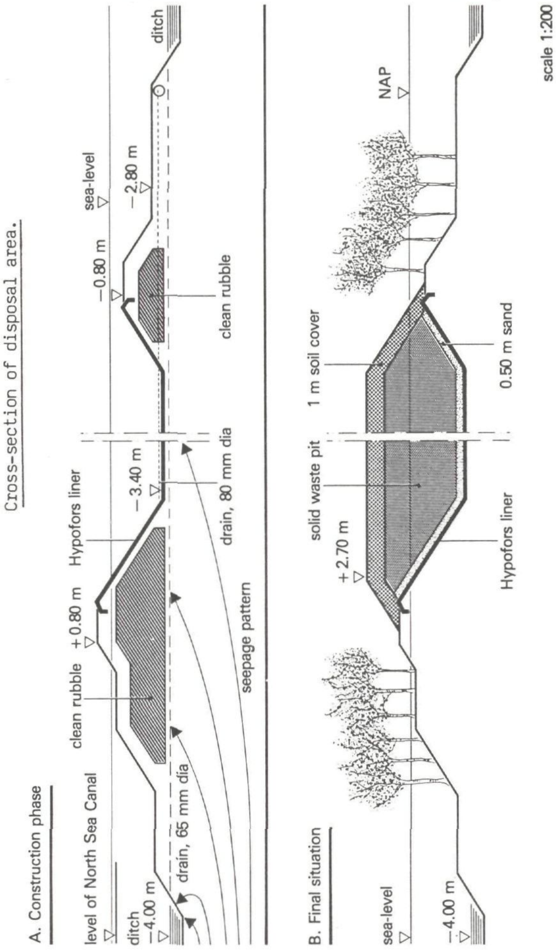
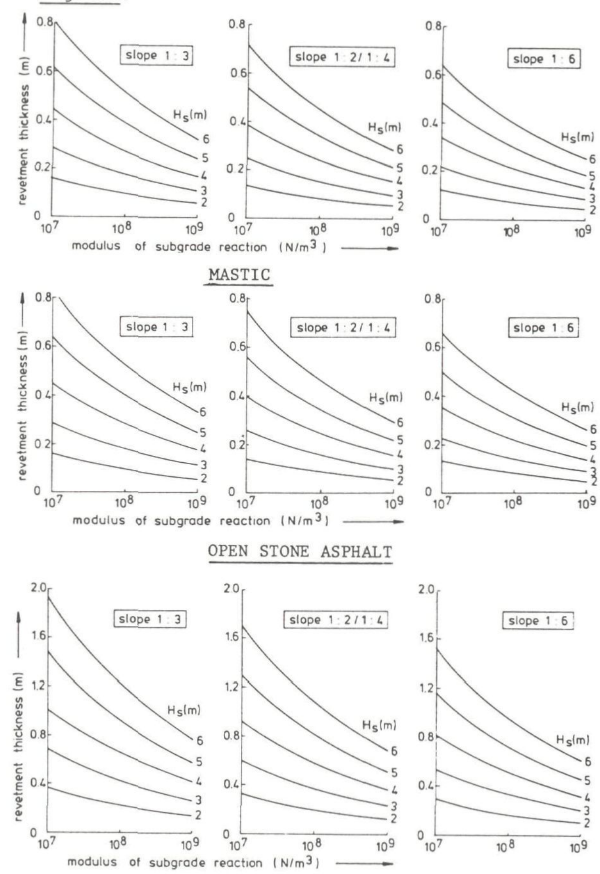


Fig. 20.

ASPHALTIC CONCRETE



Revetment thickness for bituminous mixes as function of modulus of subgrade reaction,waveheight and slope.

In which

h = revetment of thickness, (m)

P = wave impact (N/m)

σ_b = asphalt stress at failure (N/m²)

S = stiffness modulus of the asphalt (N/m²)

ν = cross contraction coefficient (= Poisson ratio) of the asphalt

c = modulus of subgrade reaction (N/m³)

The parameter P has to be determined. A problem is that stormwaves are of varying sizes. Also the duration of the storm varies. Since asphalt is sensitive to fatigue, the strength changes with the number of loadings. In the Dutch "Guidelines for the use of asphalt in Hydraulic Engineering" (1985) a method is given by which out of a storm, the design waveheight and impact can be derived. The choice of breaking strength σ_b is dependent on the number of wave impacts during the storm.

The factor j is a parameter which represents the decrease in strength of the revetment by wave attack during the period, previous to the design storm. For parts of the revetment which are only loaded by waves during the design storm, the value of j=1; for parts of the revetment which are attacked by waves under normal circumstances than $j < 1$.
For assumptions referring to Dutch circumstances,the mentioned formula has been worked out in several graphs (Fig.20). From these graphs the revetment thickness (h) for several standard mixtypes can be obtained as a function of the modulus of the subgrade reaction. Graphs are given for different slopes (1:3, 1:2/1:4 and 1:6) and some design wave heights.

The assumptions are:

- the duration of the design storm: 36 hours (3 tidal cycles).
- the number of impacts during the storm: 10% of the total numbers of waves.
- wave distrubtion in the storm: Rayleigh-distribution.
- relation between the significant wave height (Hs) and the average wave period (T) is $T=3,5\sqrt{Hs}$.

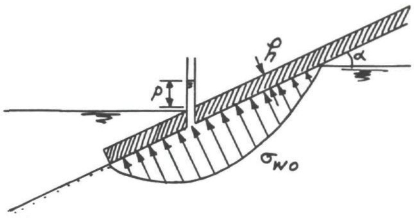
The magnitude of the wave impact P is the maximum wave pressure times the length over which this pressure is acting (0,4 Hs). It should be mentioned that the maximum wave pressure is depending on the angle of the slope. The duration of the pressure is stated at 0,3 sec. and the temperature of 5°C.
For fully asphalt grouted stones the graphs for mastic can be used. The derived thickness has to be multiplied by the factor 1,75 (v.Herpen, paper 17). It should be remarked that in openstone asphalt the wave pressures are propagating quickly through the hollows in the revetment. The resulting load on the plate is therefore smaller than in case of dense stone asphalt.

4. DIMENSIONING ON HYDRAULIC UPLIFT PRESSURES.

Depending on a number of factors, such as:
- duration and height of the outside waterlevel,
- permeability of the subsoil and dike core,

a groundwater level will establish in the dike core (Fig.21).
In case the outside waterlevel is dropping, for example the tidal high water after a storm, the

groundwater level in the dike core becomes higher than the outside level.
The water will flow out of the core but this is onstructed by the impervious revetment, so that uplift pressures are developed.



$$\sigma_{wo} = \rho_w \cdot g\,(p + h.\cos\alpha)$$

Uplift creterion :

$$h \geq \frac{\sigma_{wo}}{\rho_a \cdot g \cdot \cos\alpha}$$

Sliding creterion :

$$h \geq \frac{f.\sigma_{wo}}{\rho_a \cdot g\,(f.\cos\alpha - \sin\alpha)}$$

h = layer thickness (m')
f = friction coefficient : $f = tg\,\phi$
σ_{wo} = maximum uplift pressure (N/m^2)
ρ_a = density of revetment material (kg/m^3)
g = gravity acceleration (m/s^2)

Fig. 21. Hydraulic uplift pressures

Because of the often long duration of these pressures and the viscous character of asphalt, remaining deformations of the revetment can be the result.
There are several methods to determine hydraulic uplift pressures. The best methods are an electrical analogue (in The Netherlands under the name of the ELNAG-model) and a numerical model based on finite elements.
The maximum uplift pressure can be obtained by a simple calculation when the upper pressure at the outer surface of the revetment is known by measurement (expressed in meters waterhead):

$$\sigma_{wo} = \rho_w \cdot g\ (p_u + h.\cos\alpha)$$

in which

σ_{wo} = maximum uplift pressure (N/m^2)
ρ_w = density of water (kg/m^3)
g = acceleration due to gravity (m/s^2)
p_u = uplift pressure related to the revetment surface (in meters waterhead)
h = revetment thickness (m)
α = slope angle.

The largest uplift pressure occurs at the intersection of the outside water level and the revetment. This level together with the ground water level varies sothat uplift pressures change too, in location and extent.
In The Netherlands, hydraulic uplift is taken into account and design rules are developed. Two citeria are applied (v.Herpen, paper 17).

A. The sliding criterion.

This criterion is applied for frequently occuring circumstances; for example (spring) tides. The criterion prevents that the friction force on an element of the revetment, which is reduced by hydraulic uplift pressures, becomes less than the component of the nett weight of the element parallel to the slope. When this happens, that piece of revetment hangs onto the upper part and rests on the lower part where friction is still large enough. Permanent deformation will occur. When this situation repeats, the deformation can become so large that the revetment fractures. This is expressed in the formula:

$$h \geq \frac{f\sigma_{wo}}{\rho_a g\ (f\cos\alpha - \sin\alpha)}$$

f = friction coefficient.
ρ_a = density of the revetment material.

B. The uplift criterion.

Under extreme circumstances as superstorms, it has to be prevented that the revetment is lifted. When this happens sand movements can occur towards the cavity under the revetment and prevent it from adjusting to its original form.

In formula: $h \geq \dfrac{\sigma_{wo}}{\rho_a g\cos\alpha}$

Remark: In these criteria the revetment is schematized in discrete elements.
No account is being taken of the fact that the revetment is a plate which has to resist bending forces.

Finally, I would like to say a few words concerning the development of design methods in hydraulic engineering.
In principle three design methods can be distinguished (Fig. 22).

1. The deterministic method, this is based on conservative load and low to mean strength values, with or without applying a safety coefficient.
2. The quasi probabilistic method, in this case characteristic values are taken for the basic variables. For example a probability of exceedance of 5% for loads and a probability of non-exceedance of 5% for strengths are chosen as characteristic values. By means of partial safety coefficients a margin between both values is stated.

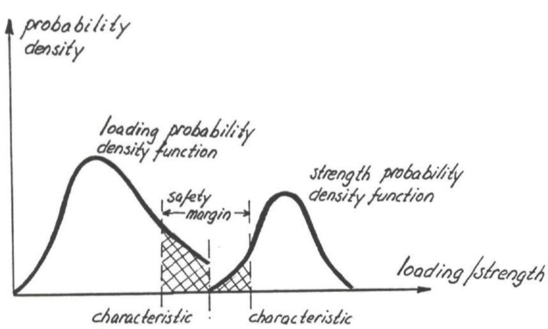


Fig. 22 Quasi probabilistic method.

3. The probabilistic method.
Now all basic variables (loads and strengths) are specified by probability density functions.
By integration the probability of failure (or a certain damage) of the structure is determined and balanced with the design criterium.

In general, the deterministic method is used. However, in The Netherlands nowadays designing tends towards the quasi probabilistic method. The introduction of the probabilistic method is in preparation in the frame work of the studies of the Technical Advisory Group for the Waterdefences (T.A.W).

Functions, a choice of applications and some important design aspects have been considered. Summarizing one can say that asphalt in hydraulic engineering plays an important role in ocean and inland water conditions.
Further developments will strengthen this statement.

ADDENDUM to papers in Volume I:

I Paper V.1 by Mr. van Damme and Mr. Brouns:

The evolution with the time of lean sand asphalt characteristics in marine environment

Prepared by :

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From the start the client, the contractor and the designers were concerned about the evolution with time of the characteristics of the lean sand asphalt, the main constructional material used for the Service port breakwaters in Zeebrugge (ref. article V.2 Proceedings). In order to study this behaviour 3 test campaigns have been executed to date.

The first campaign took place before and during the execution of the Service port breakwaters. Tests were carried out on samples of lean sand asphalt made in the laboratory in order to define the proper data for the stability analysis.
In 1981 a series of samples was taken by borehole, and an investigation of the geotechnical properties of the lean sand asphalt was carried out.
In 1985 a further test campaign was started. Not only the geotechnical aspects were to be considered this time, but also research on the quality of the components of the sand asphalt was carried out. All the tests for this campaign have not yet been completed. However the results already obtained, together with the results of the previous tests, give a good insight into the evolution of some characteristics of lean sand asphalt.

In Figure 1 various characteristics are plotted against the dry density γ_d which itself varies between 1,47 and 1,76 t/m^3.

Where the 1978 tests show clearly that the higher γ_d the lower the relative deformation $\frac{\Delta Hn}{Hi}$, the 1981 and 1985 tests do not show such a clear relation. It is, however, clear that the values of the relative deformation found in 1981 and 1985 are lower than those found in 1978.

During the tests in 1978, cohesion c' shows a little increase with increasing dry density γ_d. However the results from 1981 and 1985 give generally such low values that no clear relation can be established.

For the values of the angle of internal friction, an increase was found compared to those determined in 1978. For the latter, values varied between 18° and 20°, while results in 1981 and 1985 showed values from 26° to 33,50° except for two samples taken at depths lower than 1 m. This development of higher values occurred between 1978 and 1981, but it is not known over exactly which period. The relation itself between the angle of internal friction φ' and the dry density is similar in the three cases : the angle of friction increases with increasing dry density. For the 1978 results this increase is less pronounced than for 1981 and 1985. Compared to other sands the lean sand asphalt test results give the same general picture

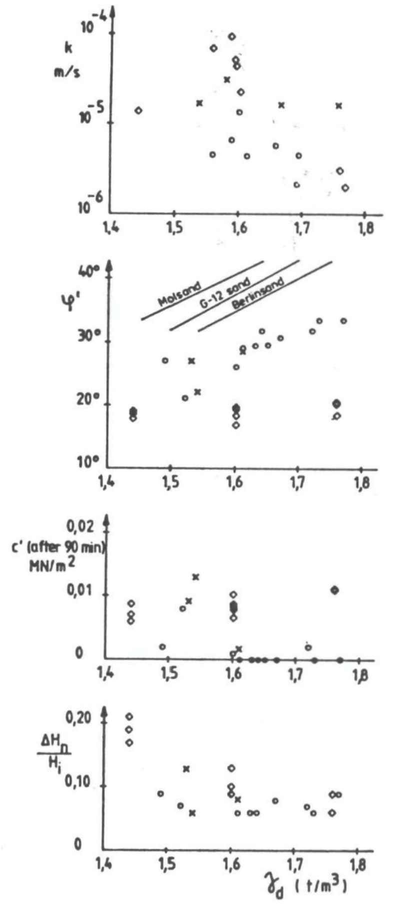

figure 1
◦ 1978 : samples formed in the laboratory
× 1981 } samples taken in boreholes
∘ 1985

for the relation between the dry density and the angle of internal friction.

For the variation of the permeability with increasing dry density, no clear relation can be seen. However the graph shows a decrease of permeability between 1981 to 1985.

Figure 2 plots various characteristics versus depth. At the left the variation of the sea-water level and the water level in the breakwater is shown. This latter water level was measured in a standpipe.

The second graph shows dry density versus depth and indicates a rather large scatter of the results. In the upper 2 meters the values are smaller than those at greater depth.

A similar phenomenon can be observed when studying the relation between the angle of internal friction φ' and the depth. Lower values - only slightly higher than the values of 1978 - are found in the upper 2 m. Higher values varying from 26° to 33,50° are found deeper in the core of the dam.

We also notice that the value of cohesion c' varies with depth, which in turn varies from about 3 m to nearly zero.

As already stated the values of the permeability of 1985 are somewhat smaller than those of 1981.

Figure 3 shows a typical result of a cone penetration test (CPT) in lean sand asphalt carried out on the Northern breakwater of the Service harbour.

3 different zones which correspond to following boundary levels can be identified :

- the top layer A, up to 3,0 m in depth, with very high values of cone resistance q_c. This layer never becomes saturated with sea-water.
- the middle layer B, from 3,0 m to 4,5 m depth with q_c values ranging from 15 to 37 MN/m². The variation of the water level inside the breakwater occurs in the upper part of layer B. The lower part remains permanently saturated.
- a lower zone C, from 4,5 to 7,0 m depth, with q_c values of about 10 MN/m². This layer lies completely below the low sea-water level.

At this very moment we are checking whether or not the higher q_c values of layer B are related to the consecutive construction phases of the dam.

Apart from the geotechnical tests, in 1985, an investigation has been carried out on the lean sand asphalt samples in respect of their bitumen content, the quality of the bitumen, the mineral fraction and the homogeneity of the mixture.

TABLE 1 : COMPOSITION OF LEAN SAND ASPHALT

	Sample nr.	Bitumen content % (m/m)	Number of samples
Zone A	1	3,8	4
	2	4,2	5
Zone B	3	3,9	5
	4	4,0	4
Zone C	5	4,4	4
	6	3,3	6

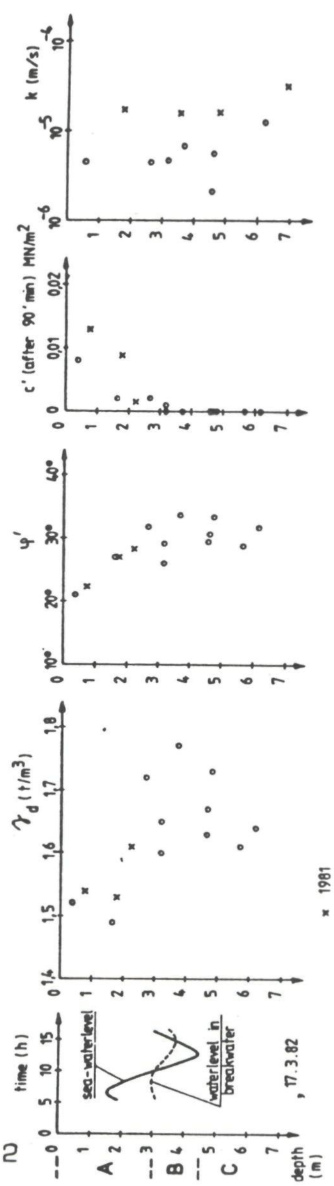

figure 2

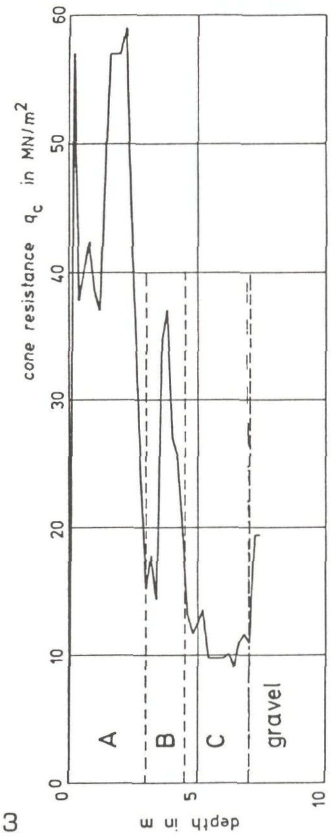

figure 3

Table 1 shows the composition of the lean sand asphalt. Given the limits accepted, the bitumen content corresponds well with the percentage bitumen that has been specified for the production, namely 4 %.

After recovery of the bitumen from the samples, two characteristics were studied : Penetration and Softening Point (Ring and Ball).

From the results listed in table 2, it is evident that the penetration value increases with depth, while the softening point decreases with it.

Compared to the initial values i.e. penetration of 80 - 100 (x 0,1 mm) and softening point ranging from 45 to 52° C, it was found that bitumen from samples which remained above the high water level was subject to more pronounced hardening than the bitumen from samples which were at all times submerged.

TABLE 2 : PENETRATION VALUE
SOFTENING POINT

	Sample nr.	Softening (R + B) °C	Penetration x 0,1 mm
Zone A	1	64	18
	2	73	10
Zone B	3	61	30
	4	60	29
Zone C	5	54	46
	6	53	47

An indication of the mixture itself is found through microscopic research. A fresh mixture shows a very thin bitumen film surrounding each sand grain, and a bitumen bridge linking two grains as shown in fig. 4A.

In several samples, and especially those taken from places where a bulk quantity of lean sand asphalt has been poured under water, it is found that the bitumen film no longer surrounds the complete grain, but that it is contracted to several spots on the grain surface; also some bitumen has contracted to the contact areas between the single sand particles (fig 4B).

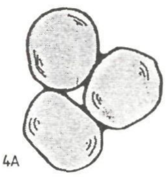

FIG. 4A

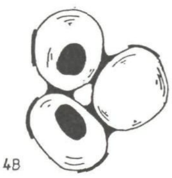

FIG. 4B

Partial stripping, due to the exposure to water of the lean sand asphalt at relatively high temperatures and for long periods may be a reasonable answer to this phenomenon.

Previous experiences on projects showed that, indeed, even under water, sand asphalt poured in bulk quantities cools down very slowly due to its poor heat conductivity.

As for the mineral aggregate, a good conformity between the sieve analyses carried out on samples taken now, and carried out on the sand before production in 1978, has been found.

Preliminary conclusions :

1° the upper layer (about 1,5 to 2 m at least) holds its initial characteristics and retains its function as an armour layer.

2° from samples taken inside the breakwater, it can be seen that some characteristics tend to those of sand : the angle of internal friction increases and the cohesion decreases.
The two potential slip surfaces calculated in a slope stability analysis and shown in fig. 5, show that this results in an increase of the safety factor.

3° the change of the various properties of sand asphalt seem to occur at a very early stage.
This can only be confirmed if on another occasions, suitable and convenient research programmes can be carried out.

REFERENCES

1) Influence of the main normal stress on the shearing strength of sand.
De Beer E.E.
Proc. of the VIth International Conference on soil mechanics and foundation engineering. Montreal 1965. Vol. 1, p. 165 - 169.

2) Geotechnical characteristics of sand-asphalt as a construction material for the breakwaters of the service port at Zeebrugge.
Carpentier R., de Saint Aubain T., Mulders G.
Preprints Symposium Engineering in Marine Environment, Brugge 1982, p. 3.105 - 3.133.

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II Paper V.7 by Mr. Domange and Mr. Herment:

Page 741: Le Barrage reservoir du Vallon Dol:
Le déformations totales maximales, au milieu de la crête du barrage, sont seulement de 26 mm en tassement et de 14 mm en planimétrie, vers l'aval.
Le débit permanent moyen du captage des 2 sources

amont, en rive droite, d'ailleurs directement influencé par les précipitations, tourne autour de 0,2 l/s (et non de 2 l/s); le débit de fuite total est du même ordre, les drains côté rive gauche étant continuellement secs.

Page 741: Le Barrage de Kruth-Wildenstein: Un seul joint, au niveau du raccordement en partie supérieure du rampant, entre matériau mis en place mécaniquement et complément mis en place à la main, a été constaté depuis les travaux de réfection et a été repris en 1983; depuis lors, plus aucun désordre n'a été observé.

Page 742: Le Barrage de PLA DE SOULCEM: Nous annoncions l'éreuve de la vidange complète. Cette dernière confirme les bonnes dispositions prises lors du projet, au vu des résultats enregistrés, notamment:

- du tassement maximal noté en crête, avec 15,5 mm au remplissage de la retenue,
- et des fuites du barrage et de sa fondation, évaluées à 2 à 3 l/s.

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III Paper V.8 by Mr. Post and Mr. Huynh:

A few photos have been printed in the wrong sequence:
Vue 8 should be placed on Vue 4
Vue 4 should be placed on Vue 5
Vue 5 should be placed on Vue 6
Vue 6 should be placed on Vue 7
Vue 7 should be placed on Vue 8

SESSION V

DISCUSSIONS

Groot,M.B. (the Netherlands)

Mr.van Herpen I have a question to your paper V,17 about the "uplift criterion", as given by you in section 3.3 on page 802,left below.

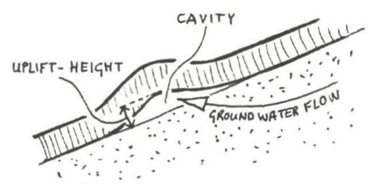
Is the "uplift criterion", even for extreme circumstances, not too conservative? Do you intend to quantify the safety between "no uplift at all" and "complete destruction"?

EXPLANATION:

As soon as any uplift occurs, two factors start to limit the uplift-height:

I Stiffness of asphalt-plate (mentioned by you)

II Gradient needed for the groundwaterflow to fill the cavity (it takes time to fill the cavity)



REMARK:

In similar circumstances (bedprotection Oosterschelde barrier), calculations made clear, that the uplift-height of the asphalt-layer during one extreme tide, remained very limited (centimeters). See V.19 page 813, left below. See also remarks of V.14, first part of the Introduction.

REMARK:

Hydraulic-gradients near the cavity are much too low to allow for any sand movement!

Herpen,J.A.van, (the Netherlands)

In the Neterlands the normal procedure to calculate the thickness of an impermeable asphaltic revetment e.g. asphaltic concrete, is by applying the sliding-criterion for frequent occurring circumstances such as tides and the uplift-criterion for rare circumstances such as heavy storms (see paper V.17). For this the revetment is divided into discrete elements and for each element the equilibrium of forces is established.
The uplift criterion means that hydraulic pressures underneath may not lift that particular element. The counterforce is the elements own weight.

The main reason why lifting is not allowed is that it is thought that sand particles can move into the cavity and thus prevent the revetment from returning into its old form. Bulges may be the result. When this situation occurs frequently or the loading has a relatively long duration these bulges may be substantial and continuining.

It is neglected that:

a. the revetment is a plate with a certain stifness and strenght and thus can resist bending forces;
b. in order to establish a cavity underneath the revetment ground water has to flow into the cavity which takes some time.

It is also stated by mr. De Groot that hydraulic gradients near the cavity are much to low to allow for any sand movement.

The following remarks can be given:

- The proposition by mr. De Groot that ground water flow takes to long and hydraulic gradients are to low to result in any sand movement is based upon investigations carried out in the last few years on the bottom protection mattresses for the Eastern Scheldt Storm Surge Barrier, which is situated on a horizontal subsoil.
  When a cavity occurs under an asphaltic concrete dike revetment which is placed mosttimes on a slope, sand movement by hydraulic forces is stimulated by gravity. Also the circumstances to which the revetment is designed e.g. extreme waterlevels, often have a relative long duration (several hours). In that interval it is thought possible that sufficient water (and sand) can flow into the cavity.

- When a wave impact acts on the revetment with underneath a cavity the resistance is reduced and yielding may occur.

However in principle their seems to be no objection to allow uplifting of an asphaltic revetment under very rare circumstances to a certain extent. Discussions about this topic take place at the moment. Investigations may be desirable.

Ishai, Ilan (Israel)

During the last five years,we can producing in Israel low-temperature energy by "Solar Pools". In these pools the water temperature at the bottom is between 80-95°C.
One of the problems in this respect (which has not been completly solved yet),is the waterproofing(sealing)of the lining membrane against the leakage of the hot salty water layers at the bottom of the pool.
Several solutions,including plastic membranes,blown asphalts etc. were failed to provide.
Some testing is being carried out now with reinforced blown asphalt with high softening point, yet with fairly reasonable values of penetration and ductility.
Do teh moderators,or any of the participants have an experience or ideas in this direction?

Nievelt,G.(Austria)

In Austria SBS modified bitumen felts without fabrics (5 mm thick and $T_{R\&B}$= 125 °C) are used successful for lining small reservoirs.

Schönian,E. (Fed.Rep.of Germany)

In Germany this problem is studied by Mr.Balzereit for the storage of water at temperatures of about 95 °C in more detail.

SESSION V

CONCLUSIONS

1. Production.

Development of equipment and execution methods in combination with the properties of bituminous mixes have lead to high productions and constant quality in the structure. This also true for severe meteorological and hydraulic conditions.

2. Mixes.

There exists a variety of compositions for the same type of bituminous mixes. This means that also the properties and qualities are different. Characterisation and measurements of the properties are therefore not consistent.

3. Characteristics.

Bituminous mixes and emulsions have been developed with such properties that they can be applied in a wide field of hydraulic engineering structures like dikes, barrier dams, reservoirs. Some characteristics are:

* good erosion resistance (protection effect)
* easy repair capabilities (safety)
* simple mechanical handling of materials
* relatively low maintenance cost, also in relation to re-use of bituminous mixes (lifetime and economy).

4. Design.

Design methods for asphalt dike revetments are developed based on the physical and mechanical properties of the asphalt mixes, the modules of the subgrade reaction on waveheight.

RECOMMENDATIONS

1. Technology of the bituminous mixes should be further developed, related to the psychical/mechanical properties and durability aspects, at dynamic and static loading and threats during lifetime. To be mentioned are for instance:

   - permeability
   - physical/mechanical properties: stiffness modules, strain at failure, fatique and creepbehaviour in relation to triaxial loading.
   - stability and flexibility behaviour
   - viscosity of bitumen binder or mortar

2. Design of bituminous hydraulic structures has to be studied in more detail especially interaction with other construction layers and components,at different loading conditions. Probabilistic design methods needs to be introduced.

3. Quality insurance of the design, execution and maintenance has to be developed. The quality of bituminous layers for instance as a function of the compaction of that layer and the bearing capacity of the subbase.

4. Monitoring.

Measurements of loads, strength, geometry and quality parameters should be further developed and introduced to establish the stability (safety aspects) and maintenance control.

5. Guidelines.

Further attention must be paid to formulate and apply guidelines for design, execution, monitoring and maintenance of bituminous mixes in hydraulic structures.