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Experimental research into the internal stability of a base layer of recycled asphalt granulate mixed with sand under wave loading.

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ABSTRACT: The Netherlands has 600 kilometres of asphalt revetments on sandy dikes to resist severe wave attack. More recently, base layers have been used consisting of granular material, that gives additional protection. As the phreatic line in the dike can be high, the internal stability of these layers needs attention, as part of a failure mechanism that leads to disintegration of the granular material due to wave action and as a result unallowable S-shaped deformation. The asphalt granulate sand mixture was sampled in the field for further laboratory research into the permeability and internal stability of this material. 6%, 14%, 21% of added sand covers the variability in the field. An upward water flow was induced in the laboratory, and increased step by step. The head difference was cycled at the end of the test series, thus simulating the effect of wave action. Across the sample the hydraulic head was measured with small standpipes at 5 locations. Upon placement of the sample, hydraulic heads were applied across the sample and the head differences, flow rates, and permeant water temperature were monitored periodically. In addition, camara images were taken. It is concluded that the internal stability of the samples is insufficient, the apparent cohesion vanishes due to cyclic loading and the permeability increases drastically. This implies that the material will form an unwanted S-shape profile that finally causes failure of the asphalt layer.

1 INTRODUCTION

The Netherlands has 17,691 kilometres of dikes that protect against flooding. 600 kilometres of these have an asphalt revetment that needs to resist severe wave attack and protects the vulnerable dike body against erosion. These asphalt revetments consist mainly of asphaltic concrete, have a slope that is often close to 1:4 and have a typical thickness of 20 cm. The lower end of the asphaltic concrete is a few meters above sea level and this asphalt is impermeable. However, water can enter the dike through the lower part of the slope and through the sandy subsoil. More recently, base layers have been used consisting of granular material, that gives additional protection. The typical thickness of these base layers is 30 cm.

As the phreatic line in the dike can be high, the internal stability of these layers needs attention, as part of a failure mechanism that leads to disintegration of the granular material and as a result unallowable S-shaped deformation (Wichman et al., 2018). Figure 1 shows the response of the structure to wave action. Due to the water flow towards the moving asphalt layer, large cyclic gradients occur under wave attack with a significant wave height of a 1.6 to 2 meters, see (Wichman et al., 2018), that may cause liquefaction and disintegration. The uplift of the asphalt layer is limited by de permeability of the underlying granular material, since only during wave rundown for a few seconds groundwater will flow in the space beneath the asphalt layer (see Figure 1). There are about 100 kilometres of dikes that suffer from this potentially dangerous mechanism, and there will be more dikes in future, as, due to climate change, the phreatic line tends to rise, and can become higher than the lower end of the asphalt revetment.

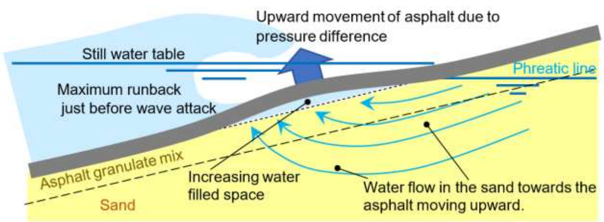

Figure 1. A sketch of the response of the asphalt – on – the asphalt granulate mix – on - sand under wave action.

The research presented here focuses on the potential benefit of a typical granular base layer, consisting of recycled asphalt granulate and a variable fraction of mixed dike sand. The aim during construction was to add 20% of dike sand, as this gives the best densification (de Bock, 2004). This material was expected to have a substantial cohesion and therefore some resistance against liquefaction and disintegration. In (Mousa et al., 2021) and (Wu et al., 2012) it is shown that the blending of asphalt granulate (RAP) with crushed aggregate increases the apparent cohesion and the resilient modulus, while this reduces permeability. This apparent cohesion can disappear due to cyclic loading, as fatigue of the bituminous material occurs.

2. RESEARCH AIM

An extensive field research was done to determine the layering, the composition and density of the asphalt granulate-sand base layer. The material was sampled for further laboratory research into the internal stability and permeability. In the laboratory upward flow gradients based on (Wichman et al., 2018) in the material just beneath the asphalt layer were applied, at first in increments and finally cyclic. According to cyclic consolidation theory, there is a limited penetration depth of the cyclic pressure changes beneath the moving asphalt plate. These pressure changes are typically 20 kPa for the upper part of the wave attack. A degree of saturation in the sand of about 90% leads to a penetration depth in the sand of the order of decimetres (with wave period of 6 seconds and permeability range 1,5e-4 – 1,1e-3 m/s). The same holds for the asphalt granulate-sand mixture, where the penetration depth is expected to depend on the sand fraction as well. Due to this, gradients in pressure occur that are larger than 1 during a time span of seconds. These cyclic upward gradients can cause local liquefaction if the cohesion in the granulate material is insufficient.

The questions to answer were: does the granulate asphalt-sand mixture disintegrate, and is there any transport of fines? These processes also affect the permeability of the granulate material and thus affect the uplift and potential failure of the asphalt layer.

In the laboratory research, permeability was measured for three different mixtures of asphalt granulate and sand that cover the variability in the field (the lower limit of the sand fraction is 6%, the higher limit is 21% and 14% represents the major part in the middle, see Table 1). Permeability was determined by means of Darcy's law from the pressure difference across the sample (pore pressure ports S1 and S5, see Figure 3) and the measured outflow, under variable hydraulic conditions, including cyclic loading (50 times from 60 cm head to 10 cm to 60 cm head again).

The laboratory samples were built artificially, layer by layer and with 100% proctor density, see (*) in Table 1. The grain size distributions from the field were slightly adapted, in a sense that the large grain size fraction > 31.5 mm was left out. The grain size distributions of the three tested mixtures are given in Figure 2.

Table 1. Material properties of the soil mixes.

Property	**6% Sand Mix**	**14% Sand Mix**	**21% Sand Mix**
Maximum Dry Density (Mg/cm^3) (*)	1.569	1.651	1.716
Optimum Moisture Content (%) (*)	3.9	3.9	4.0
Angle of Repose (^{o})	30	32	30
ρ_s (-)	2	2	2.1
Coefficient of uniformity, c_u (-)	4.31	10.66	36.75
Coefficient of curvature, c_c (-)	1.35	2.31	5.68
Classification Acc. USCS	GP(**)	GP	GP

(*) following the Dutch standard RAW 2015 proef 10;
(**) GP = gravel, poorly graded.

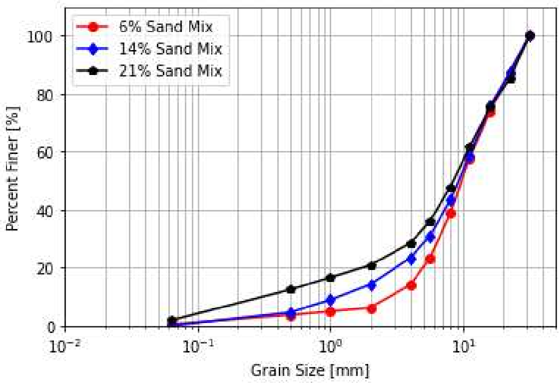

Figure 2. Grain size distributions of the three asphalt granulate mixtures tested.

The grading of the curves in Figure 2 has been checked by means of the criterion for internal stability as developed by (Kenney and Lau, 1985). For the 21% sandy mixture, the smaller grain sizes D (below 2 mm) tend to be unstable, as the fraction of grain sizes between D and 4D is not large enough. The curves have lack of detail for grain sizes below 1 mm, that implies that these fines could not be checked thoroughly and are possibly unstable. In addition, the flow in our experiments is in the reverse, i.e. upward, direction of that of Kenney and Lau, and no surcharge was applied. This implies that the fines are expected to be unstable in all three mixtures.

3. EXPERIMENTAL SET-UP

The experimental set-up is shown in Figure 3. An upward water flow was induced in the asphalt granulate-sand mixture, and increased step by step. The piezometric head was measured with small standpipes, see Figure 3. Figure 3 shows a sketch of the setup together with the standpipe port vertical spacings. Two sets of standpipes were used to detect local variations in permeability during the experiments.

The sample container has a diameter of 300 mm, and the sample height was 300 mm, which coincides to an elevation just below the topmost standpipe port. This 300 mm sample height was found in the field and the 300 mm diameter was large enough in comparison to the largest grains of 30 mm. On top of the sample an adjustable perforated plate was placed that limits the movement of the larger asphalt grains in the water filled spacing alike on the dike (see Figure 1), and that is transmitting the finer fraction. Sample preparation commenced with installation of the soils in layers of 50 mm, which were pluviated under water and by constant tamping.

Upon placement of the sample, hydraulic heads were applied across the sample and the head differences, flow rates, and permeant water temperature were monitored periodically.

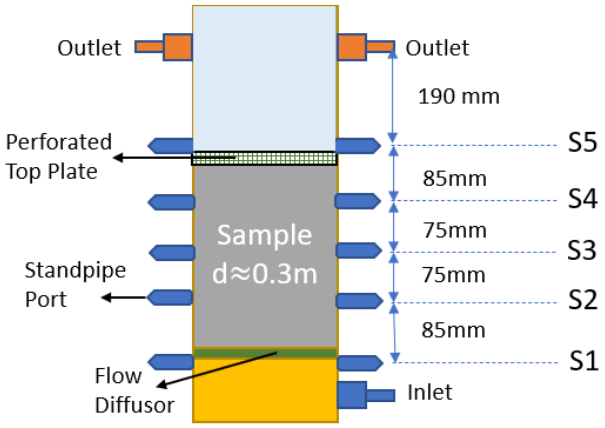

Figure 3. Experimental set-up of permeability and disintegration test. Top: front view in the laboratory. Bottom: schematic drawing with d =diameter. The filters of the standpipes S1 to S5 are in the container wall.

As local inhomogeneities are expected to arise, two arrays of standpipes across the sample height were used and camara images were taken.

4. RESULTS

Figure 4 shows the hydraulic head with reference to the top standpipe S5 during a loading phase in the experiment on the 14% sandy sample. During this loading stage the sample was prevented from any expansion by the perforated top plate. It is visible that the pressure differences S1 to S4 are not equidistant, which implies that the hydraulic resistance is not the same throughout the sample and the composition of this sample is not uniform. The higher hydraulic resistance in the bottom part is due to segregation of sand during sample preparation, as the pores in the asphalt granulate allow sand to move downward. For the 21% sandy sample this hardly occurred, as the granulate pores are filled quite well with the sand fraction and the pressure differences turned out to be almost equidistant during the first loading steps. This also follows from the permeability values that were close to that of the dike sand.

It visible from Figure 4 that S2 (i.e. the difference between the head in S2 and S5) on the left has a higher value than on the right. S3 and S4 on the left and on the right are significantly lower. This implies that the hydraulic resistance in the sample material on the left, directly 75 mm above S2, is higher with respect to the right, and this difference increases in time. Blocking of the pores by the sand transported from the bottom part of the sample gives an increase in hydraulic resistance (clogging). In this bottom part the hydraulic resistance on the left has become lower than on the right. During the cyclic loading phase, a fissure opened just beneath this clogging close to standpipe port S2, at the left side of the sample, see Figure 5.

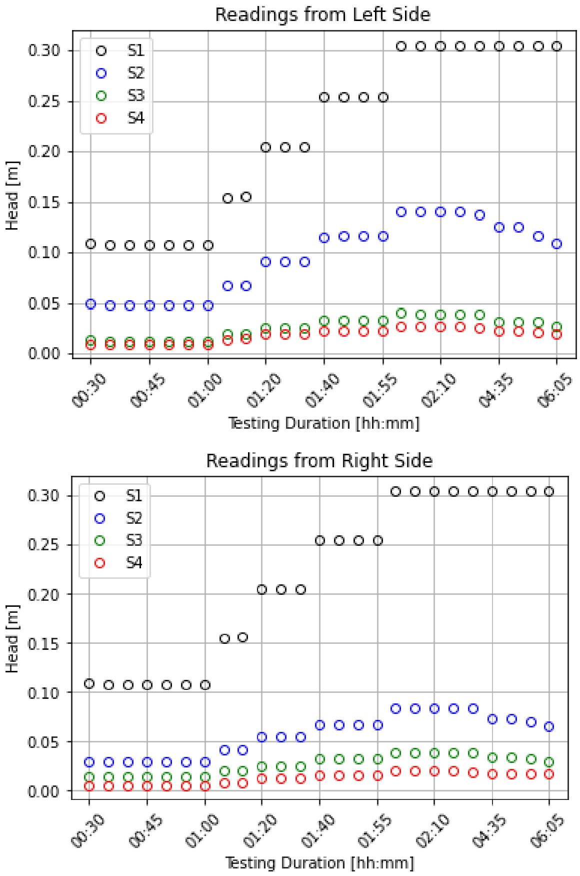

Figure 4. 14% sandy sample experiment. The hydraulic head at the standpipes with reference to S5. Left and right are as shown in Figure 3.

Figure 5. The 14% sand sample after the cyclic loading phase with fissure at the left part of the sample. Visible are the left side standpipe ports.

To investigate if this is due to an accumulation of fines, batches of material were taken at the end of the experiment for determining the grain size distribution. Close to the bottom, i.e. up to 10 cm, the fines are overrepresented. Figure 6 shows a drop in permeability of a factor of two with the loading scheme as shown in Figure 4.

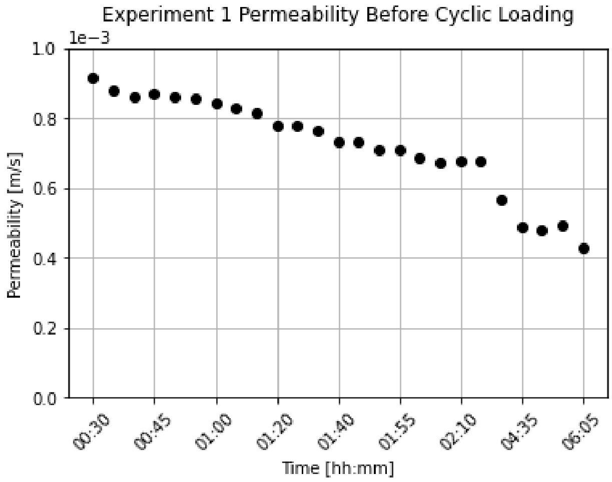

Figure 6. The 14% sand sample experiment. The overall permeability in time is shown with the loading scheme in Figure 4.

This drop in permeability is due to the transport of fines that are blocking the pores (clogging). Figure 7 gives the permeability for the three samples that was measured at the subsequent incremental loading stages in the experiments. For the 14% sand sample, the range in permeability in Figure 7 coincides with that in Figure 6. In the later stages of the 14% sandy sample experiment the permeability only slightly fluctuates, until the cyclic loading phase. After the cyclic loading (50 cycles of 60 cm to 10 cm to 60 cm head) the permeability finally doubled to a value of 0,9 e-3 m/sec.

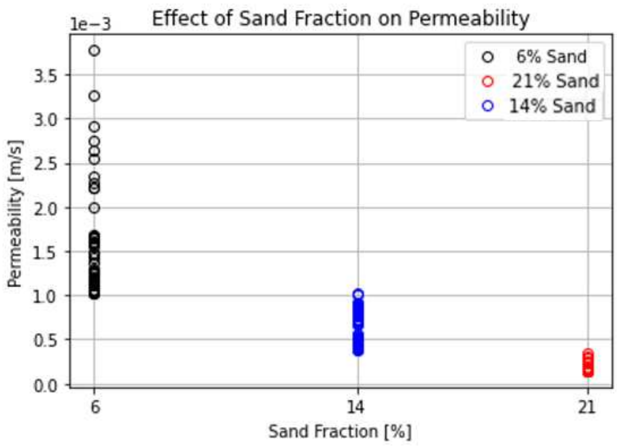

Figure 7. The permeability values for the subsequent incremental loading stages for the three samples.

The permeability values for the 21% sandy sample were close to that of the underlying dike sand.
In the other experiments, the fines tend to migrate, too, and fissures arise just beneath the clogging zones. During cyclic loading, the overall permeability increases drastically with at least a factor of two, and the hydraulic resistance between the standpipe ports indicates that the sand has moved to the upper part of the sample. Even sand was seen to leave the top part of the sample.
In the 21% sandy sample experiment, this effect was clearly visible from the standpipe readings, where the hydraulic resistance of the top part of the sample increases during to the loading cycles (60 cm to 10 cm to 60 cm head).
A head difference across the sample of about 30 cm corresponds to an average gradient of 1. If there is no cohesion, the sample would desintegrate and swell.
For the 14% and 21% sandy samples, swell of the sample occured for head differences larger than 45 cm. This leads to transport of fines and changes in pore pressure distribution at the ports S2 to S4. For the 6% sandy sample the sample did not swell until gradual loading to 60 cm head difference. During cyclic loading, the samples become fully in contact to the top perforated plate, i.e. the samples swell irreversibely with about 5 mm and the apparent cohesion vanishes.
Figure 8 shows the disintegration of the 6% sand sample and the release of fines at the end of the testing at a head difference of 80 cm.

Figure 8. The disintegrated 6% sand sample at the end of testing at a head difference of 80 cm.

5. CONCLUSIONS

It is concluded that: (1) the internal stability of the samples is insufficient, i.e. transport of fines inside and out of the sample occurs; (2) the apparent cohesion between the asphalt grains vanishes due to cyclic loading and (3) permeability increases drastically during this cyclic loading.
This implies that this base layer material disintegrates and releases fines. It is expected that, under wave loading, this causes an unwanted S-shaped slope profile. Finally, this results in failure of the asphalt layer.

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