

## Abstract Preview

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Topic: Coast protection

**Title: In situ testing of the strength of asphaltic revetments on sand dikes**

Author (s): Wichman B.G.H.M.<sup>1</sup>, Huurman M.<sup>2</sup>, Van M.A.<sup>1</sup>

Institute (s): <sup>1</sup>Deltares, Dike Technology, Delft, Netherlands, <sup>2</sup>Delft University of Technology, Road- and Railway Engineering, Delft, Netherlands

### Final 1. Introduction

**paper text:** In the Netherlands 600 kilometers of the sea dikes are protected by an asphalt revetment. These asphalt revetments have to resist considerable repeated waves with a significant wave height of up to 4.5 meters. The subsoil is normally sandy, and the asphalt layer protects against erosion. The asphalt layer can fail as a result of fatigue due to repeated loading under storm conditions. In case of very high wave loads, the asphalt can fail already after a few large waves. The thickness of the asphalt layer varies between 15 and 30 cm. The asphalt material has a visco-elastic behaviour, whereas the subsoil can be described by various material models ranging from linear elastic to a more complex elasto-plastic model. The sensitivity of the asphalt revetment to weathering strongly depends on the void ratio, mainly for void ratios larger than 5. Due to weathering, the resistance against fatigue of the asphalt revetment decreases. Therefore, Dutch law prescribes to perform a monitoring every 6 years for asphalt older than 30 years (at this moment 75% of the 600 km asphalt revetments is older than 30 years). The monitoring consists of:

- falling weight deflection measurements (FWD),
- lab testing on small samples (3 point bending),
- Ground Penetrating Radar measurements,
- visual inspection and
- calculations with the so-called software program 'Golfklap' (wave attack in English) in order to calculate the strength under storm conditions.

In this analysis the asphalt and the subsoil are considered to be linear elastic. This is a reasonable first approximation.

The bearing capacity of the subsoil plays an important role in how the asphalt revetment deforms under wave loading. This is predicted with the results of the FWD measurements. For large loadings, however, it is still unsure how the soil will behave and if other failure mechanisms than fatigue may occur. In case the subsoil becomes saturated, the bearing capacity will reduce and also failure due to liquefaction might occur. Due to weathering, the revetment becomes more heterogeneous, because void ratios are locally increasing and the process of so-called stripping is accelerated. For safety reasons, 5% characteristic values from laboratory testing are used for the analysis with the software program 'Golfklap'. This results in a safe but pessimistic estimate of the resistance of the revetment to wave loading, as the fatigue properties have a large effect on the overall uncertainty. Therefore, the current approach has to be validated in order to reduce the uncertainties.

### 2. A new test set-up

In order to obtain an adequate method for determining the strength of the revetment on a scale similar to that of the wave impacts, a new test set-up has been investigated. The aim is to simulate typical storm conditions on a weathered revetment on a dike body, and thus validate the software program Golfklap. At first, laboratory tests will be performed, in which two dimensional wave-loading conditions are simulated by means of a mechanical system. In a later stage, such a mechanical system can be applied in the field. The laboratory test is considered to be necessary, as the conditions are more controlled than in the field. These tests have been investigated briefly with some Finite Element (FEM) calculations.

Some simplifications have been made: on the basis of earlier Deltaflume experiments the wave attacks during storm conditions can be described by means of a statistical model consisting of triangular wave loadings (see ref. 1). As the sideways width of the waves is larger than 20 meter, the pressure distribution of the wave attacks is assumed to be two dimensional. Furthermore, it is assumed that all waves are running perpendicularly to the dike surface. The statistical wave model was implemented in the software program Golfklap.

The pressure distribution of one wave attack in this statistical model is assumed to be triangular with  $p_{max}$  as the maximum value of the triangle. The impact of a wave is then described by equation 1.

$$p_{max} = 0.25/\tan(\alpha) * \rho_w * g * q * H_s \text{ [eq. 1]}$$

with:

$p_{max}$  = maximum pressure on dike surface (Pa)

$\alpha$  = slope angle;

$\rho_w$  = density of water (kg/m<sup>3</sup>);

$g$  = gravitational acceleration (m/sec<sup>2</sup>);

$q$  = factor of impact (-);

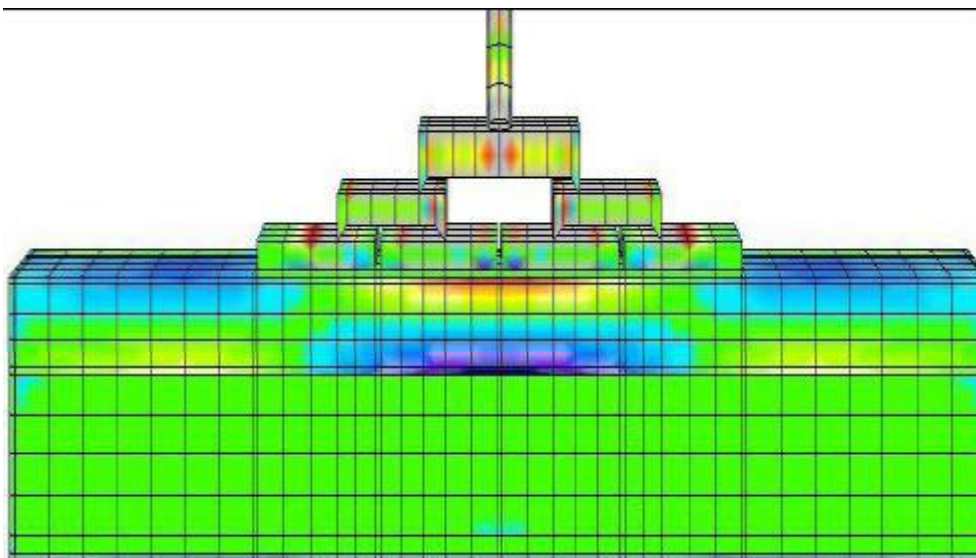
$H_s$  = significant wave height (m).

In Golfklap the bending stress in the asphalt is determined analytically, using a so-called Winkler spring foundation with a modulus of subgrade reaction  $c$ . The maximum bending stress for this case is used as a

reference for calculations with a more advanced soil model.

At first, FEM element calculations were made for a rough design of the laboratory test, consisting of a steel right-angled open container ( $l \times h \times w = 4000 \text{ mm} \times 700 \text{ mm} \times 300 \text{ mm}$ ) which is filled with, starting from the bottom to the top: a rubber mat (150 mm thickness), sand (300 mm thickness), asphalt layer (250 mm thickness).

The wave attack is simulated by means of a mechanical system, see figure 1. The load system of the test aims to introduce a triangular load with a width of 1 to 2 m and a maximum pressure in the centre. To introduce this load use is made of a statically determinate load system.



[figure 1]

The stiffnesses of the sand and the rubber mat were determined by the requirement that the calculated maximum bending stress in the asphalt (almost) equals the calculated bending stress in the asphalt on a typical sandy dike body. Also the effect of the friction (coefficient  $\mu_w$ ) between the steel and the asphalt, sandy layer and mat, respectively, has been investigated.

### 3. Expected behavior of a revetment construction during repeated loading

In order to determine the allowable amount of fatigue of the asphalt the so-called Miners rule is used, see equation (2):

$$\text{Sum } (n_i/N_{f,i}) = M \leq 1 \text{ [eq. 2]}$$

with:

$i$  indicates the respective bending stress intervals

Failure occurs when  $M > 1$ . This means that for a certain applied bending stress level  $i$  there is a maximum amount of allowable loadings  $N_{f,i}$ , which is described by means of the so-called fatigue curve. The fatigue curve has the following form:

$$\log(N_f) = \beta(\log(\sigma_b) - \log(\sigma_0))^\alpha \text{ [eq. 3]}$$

with:

$\sigma_0$  = applied bending stress in fatigue test (N/mm<sup>2</sup>),

$\sigma_b$  = the flexural strength of one load repetition and

$\alpha$  and  $\beta$  are regression coefficients (see ref. 2).

The stiffness  $E_a$  of the asphalt and stiffness  $E_s$  of the subgrade (sand), in combination with the asphalt layer thickness contribute to the bending stress level. Also  $p_{\max}$  and the geometry of the triangular wave loading (with base  $\Delta z$ ) is of influence (see ref. 1). For lower bending stress levels at the underside of the asphalt revetment, more repeated loadings are allowed until failure.

It is well known from laboratory tests that the stiffness of the asphalt decreases after repeated loading, which implies that, according to Hooke's Law, the bending stress becomes less. In this case the load is carried more and more by the subgrade, in case this subgrade is strong enough.

It is expected that the proposed large scale laboratory test will show a typical decrease in stiffness of the asphalt layer, which implies that the subsoil has to support the revetment more and more during the period of repetitive loading (period of storm surge). This implies that the bending stresses gradually will become less, and thus failure occurs in a later stage. A typical failure criterion is that the stiffness of the asphalt has become half of the initial value.

#### 4. Results of the Finite Element Analysis for optimizing the design of the laboratory test

Optimizing of the design properties of the proposed laboratory test was done by means of a Finite Element Analysis. The deformations and stresses in the revetment and the dike body under two-dimensional loading conditions conform the software program 'Golfklap' (for a Winkler foundation and a solid foundation) were compared with these numerical results of the laboratory set-up. A single triangular load is applied for two cases with different magnitude and width. The analysis has been performed in comprehensive steps.

The finite element analysis is performed with the following assumptions:

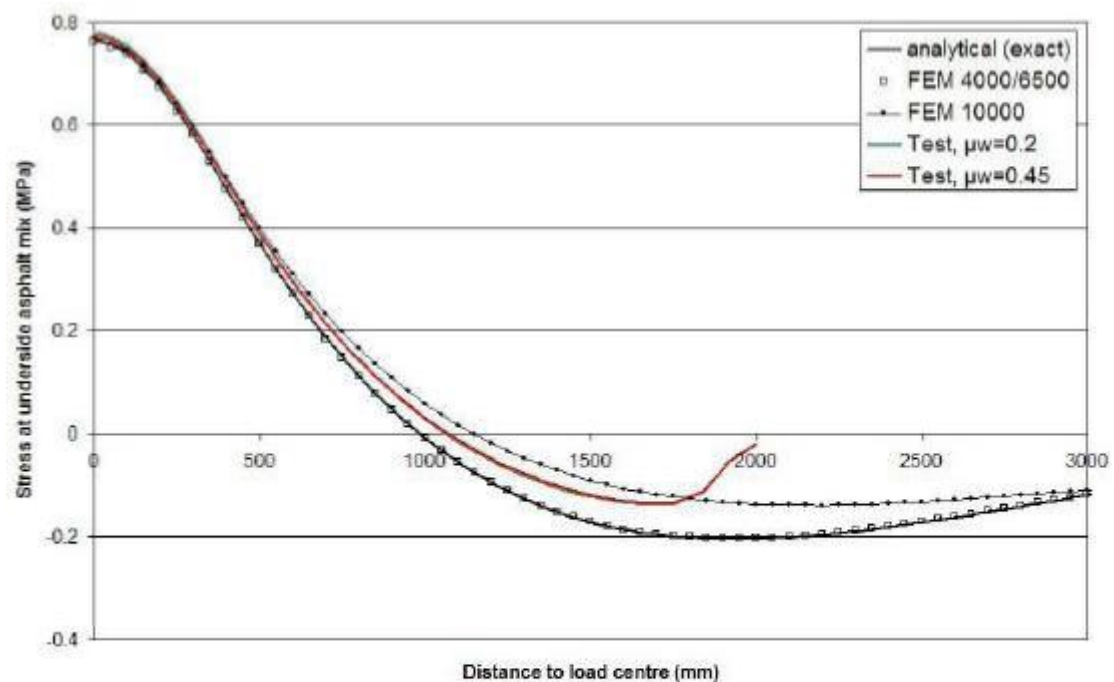
thickness of asphalt layer d	0.25 m
asphalt stiffness $E_a$	13500 MPa
poisson's ratio of asphalt $\nu_a$	0.35
maximum pressure $p_{max}$	case 1: 0.07 MPa case 2: 0.164 MPa
width of triangular wave load $\Delta z$	case 1: 1 m case 2: 2 m
Winkler foundation:	
modulus of subgrade reaction c	37.5 MPa/m
Solid foundation:	
stiffness of sandy foundation $E_s$	58 MPa
poisson-ratio sandy foundation $\nu_s$	0.35

[table 1]

$p_{max}$  follows from eq. 1 with  $\tan(\alpha) = 0.25$ ;  $g = 10 \text{ m/sec}^2$ ;  $q = 3.5$ ; case 1:  $H_s = 1.85$  and case 2:  $H_s = 4.5$  m.

The stiffness  $E_s$  of 58 MPa was determined by equalling the maximum bending stress.

The calculated bending stresses for case 1 at the bottom of the asphalt layer are given in figure 2.

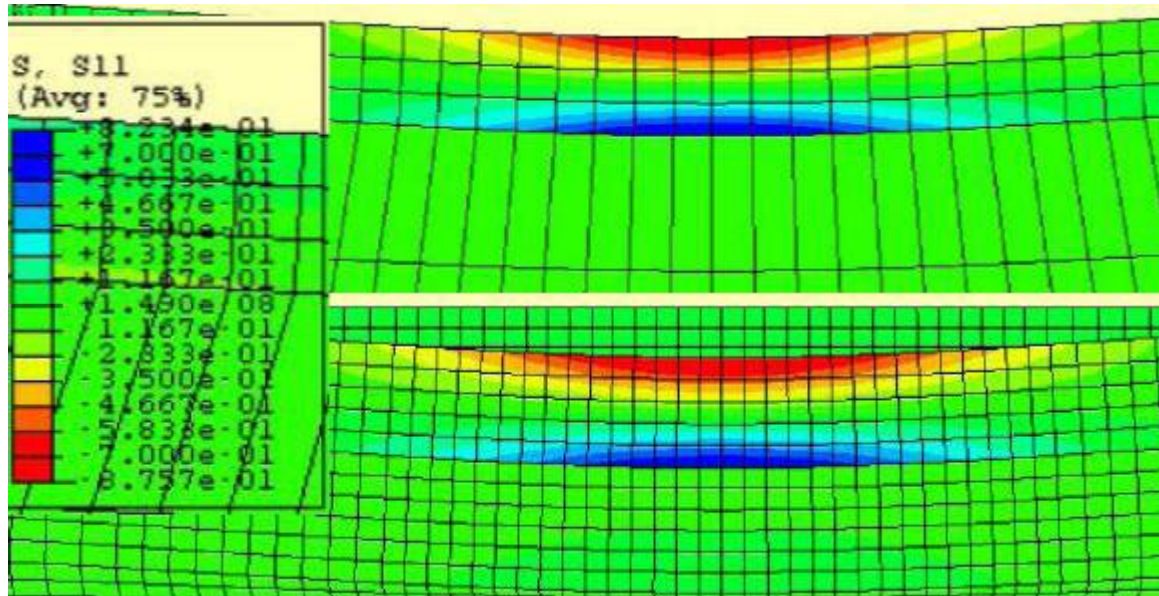


[figure 2]

In figure 2 is shown:

- the analytical solution using Golfklap (analytical —)
- a 2 dimensional analysis of the asphalt layer on a Winkler foundation for a limited length scale, i.e. 4000 mm and 6500 mm (FEM 4000/6500 □)
- a Model Solid 10000 with a modeled solid foundation with thickness 10000 mm. (FEM10000), see the top part in figure 3.
- results for the laboratory set-up with a range of friction coefficients for sand on steel of  $\mu_w = 0.2$  and  $0.45$ , respectively (Test,  $\mu_w = 0.2$  and Test,  $\mu_w = 0.45$ ). As the steel deforms sideways during loading, the

asphalt is not in direct contact with the steel and a proper sliding material can be used. The stiffness  $E_m$  and thickness  $h_m$  of the rubber mat have been determined by equaling the maximum bending stresses in the asphalt layer. It follows that  $h_m = 150$  mm,  $E_m = 3.5$  Mpa and Poissons constant  $\nu_m = 0.3$ , see the lower part in figure 3.



[figure 3]

A comparison of the bending stresses and deformations (times 250) between FEM10000 (top) and the laboratory test (bottom) is given in figure 3.

From Figure 2 it follows that the effect of the friction between the sand and the steel is limited, as the difference between Test,  $\mu_w = 0.2$  and Test,  $\mu_w = 0.45$  is very small.

In case a solid foundation with a thickness of 5000 mm is modeled, the bending stresses are similar to that for a 10000 mm thick foundation, whereas the deformations are 15% less. In reality, it might be expected that the stiffness of the subsoil increases as a function of depth. This can be taken into account in the ultimate design of the laboratory test.

For case 2, i.e. the broader and higher wave load, the differences in maximum bending stress between the Winkler foundation, the solid foundation (FEM 10000) and the laboratory set-up (Test,  $\mu_w = 0.2$ ) are less than 4%.

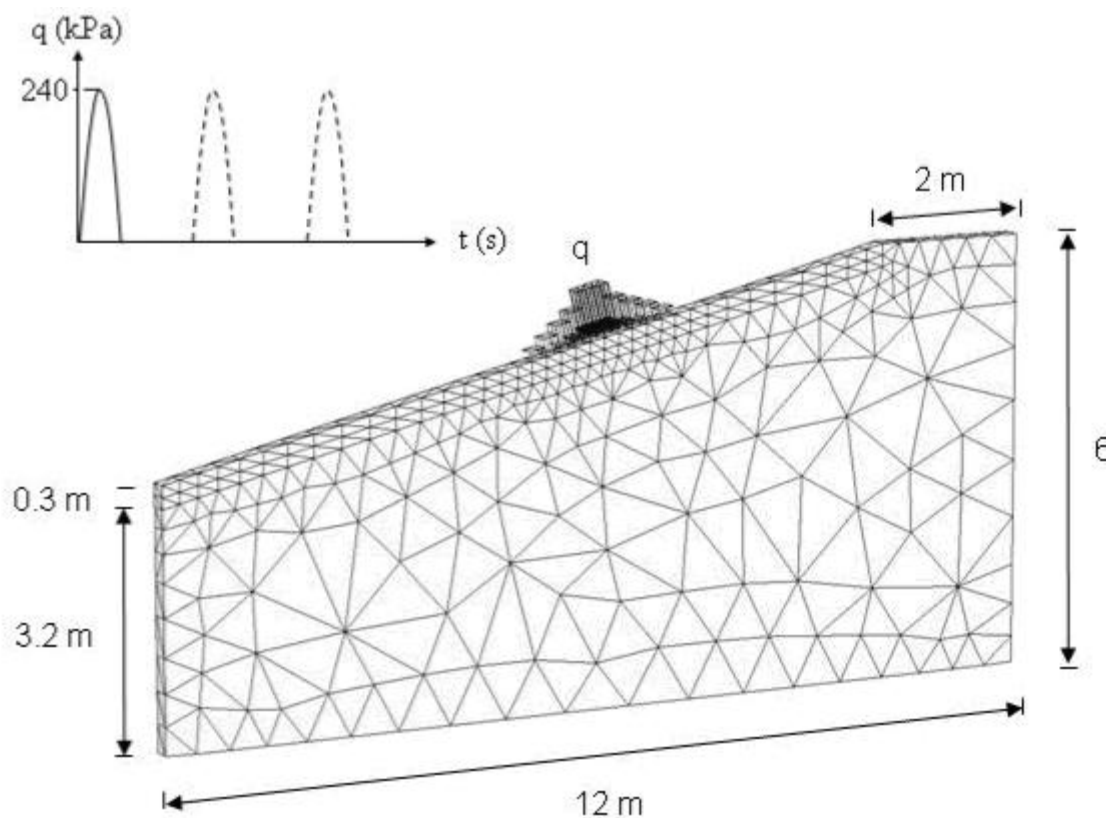
From this result and figure 2 and 3 it can be concluded that the stresses and strains in the laboratory set-up are expected to be realistic, i.e. resemble a two dimensional wave loading on a dike body covered with asphalt, within a realistic range of magnitude and width of the load.

##### 5. Finite Element Analysis of a practical field test

A practical test set-up can be obtained by bringing the mechanical loading system from the laboratory to the dike. The asphalt will be sawn perpendicular to the dike length, obtaining a section with a length of 4000 mm and a width of 300 mm. This gives a mechanically isolated asphalt section to be loaded at the center. The subsoil, however, is not sawn and therefore is three dimensional. Therefore, the bending stresses were found to be approximately 55% of that in the 2-dimensional support in the laboratory test. By increasing the load by a factor 100/55 in the field test the bending stresses can be comparable to the lab test stresses.

##### 6. Full dynamic analysis

The degree of saturation of the subsoil has a large influence on its bearing capacity. There is a potential risk of liquefaction in case the wave loading is strong, the layer thickness is small (10-15 cm) and the sand is loosely packed. The proposed field test has its practical difficulties as to saturate the sand in the dike body, and this makes a numerical more advanced analysis for partially saturated sand desirable. In order to investigate the effect of repeated loading on a partly saturated dike body, preliminary calculations have been performed by Pieter Vermeer et al. (see ref.3). The type of calculation is a non-standard finite element method (FEM) with a coupling between groundwater flow and deformations under dynamic loading. An example of the model is given in figure 4.

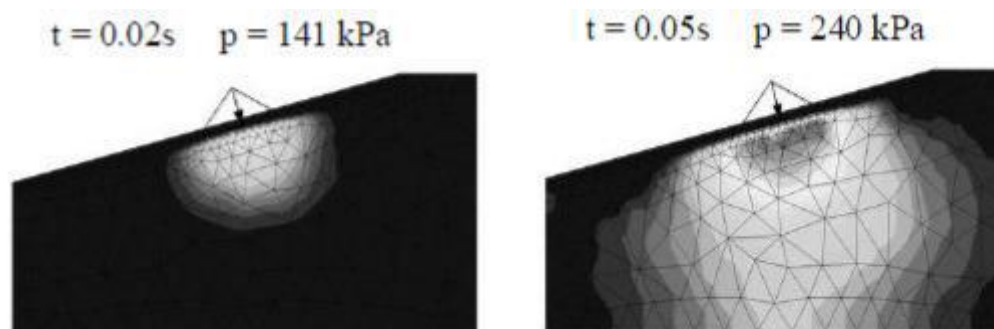


[figure 4]

The FEM calculations consisted of:

- The application of 10 severe wave attacks ( $p_{\max} = 0.24$  Mpa, a pulse duration of 0.1 sec, and a repetition interval of 10 seconds)
- on a 30 cm thick asphalt revetment, slope 1:4, considered as linear elastic (with  $E_a = 20, 1000$  and  $10000$  Mpa,  $\nu_a = 0.3$ )
- on a partially saturated sandy layer (schematised as a Mohr Coulomb soil, with  $E_s = 10$  Mpa,  $\nu_s = 0.33$ ,  $c = 1$  kPa,  $\phi = 35^\circ$ , Darcy permeability =  $10^{-4}$  m/sec, degree of saturation = 0.99).

From the calculation with  $E_a = 1000$  Mpa it follows that the maximum excess pore pressure reaches only 20% of the maximum load intensity (see figure 5). In this case the maximum permanent deformation of the asphalt after 10 loadings is 4 cm.



[figure 5]

In order to simulate larger deformations and a more accurate response, the applicability of the Material Point Method is studied in further research.

## 7 Conclusions

By means of the proposed laboratory set-up it is possible to simulate repeated wave attack on a dike body covered with an asphalt revetment on a realistic scale. In a later stage the test can be performed in the field, using the loading part of the laboratory set-up. Preliminary calculations with a non-standard finite element method show that it is possible to simulate repeated wave attack on asphalt on partial saturated sand.

**References:**

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2. de Looff, Arjan K., Martin F.C. van de Ven, Robert 't Hart, 2011, Resistance of aged asphalt concrete to wave attack, Proceedings of the Coastal Structures 2011 conference, Yokohama.
3. Vermeer, Pieter, Issam Jassim, Shuhong Tan, 2011, New Horizons in Numerical Analysis for Geotechnical Engineering, proceedings of the Int. Conf. on Geotechnics for sustainable development, Hanoi 2011.

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