

CHAPTER 161

FULL-SCALE WAVE ATTACK OF UNIFORMLY SLOPING SEA DYKES

Alfred Führböter¹⁾ and Uwe Sparboom²⁾

ABSTRACT

Shock pressure phenomena due to breaking waves acting on sloping faces of sea dykes are focussed in this paper. The probability approach is pointed out and maximum shock pressure estimations are given for smooth and impermeable dyke slopes 1:4 and 1:6. An extension of the results to steeper and flatter slopes is proposed.

Results of full-scale stability tests on concrete block slope revetments are also reported in this paper. For various structural solutions with granular and geotextile filter layers stability numbers are recommended. Initial block lifting is explained physically by pressure measurements.

The full-scale experiments were carried out in the new research facility LARGE WAVE CHANNEL of the universities in Hannover and Braunschweig (Federal Republic of Germany).

1. INTRODUCTION

In the past dykes and revetments at the coastal zone of Germany were frequently destroyed by heavy wave attack during storm surge tides. Therefore, the most important objective of the new large scale facility LARGE WAVE CHANNEL is to investigate dykes and revetments with special reference to the German North Sea Coast (Führböter, 1982). The main reasons for dyke failures can be summarized as follows:

- shock pressures due to wave breaking,
- wave run-up and overtopping,
- up- and down-rush velocities,
- dyke construction (slope, cover layer, subsoil, etc) and
- local sea state characteristics.

First investigations of the fundamental research program dealt with shock pressure phenomena on dykes with uniformly sloping faces. Shock pressures due to plunging breaker types may cause very first damage of the cover layer and the subsoil (Führböter, 1966; Stephan, 1981). Comprehensive studies on this topic were firstly published by Bagnold (1939). Contributions by Skladnev and Popov (1969) dealt with impact forces and scale effects. Nearly full-scale test results on wave

1) Prof. Dr.-Ing., Full Professor, Leichtweiß-Institute, Division of Hydrodynamics and Coastal Engineering, Technical University of Braunschweig, Beethovenstraße 51a, 3300 Braunschweig, Fed. Rep. of Germany

2) Dr.-Ing., Senior Research Engineer, Large Wave Channel, Coastal Engineering Research Group SFB 205, University of Hannover, Merkurstraße 11, 3000 Hannover 21, Fed. Rep. of Germany

impacts acting on dyke slopes were reported by Stive (1984). Führböter (1986a) compared results of model and full-scale tests of wave impacts with reference to a 1:4 dyke slope.

Further full-scale investigations were focussed on the stability of concrete block slope revetments. Such permeable revetments must be designed in combination with filter layers if the dyke core exists of sand. The filter layer can be realized by a 3-dimensional granular layer, by a 2-dimensional geotextile mat (Heerten, 1984) or by a combination of both feasibilities. Stability criteria were often found by the extension of results from small-scale models to the prototype structure (Führböter et al., 1976; Whillock, 1980; Kostense and den Boer, 1984). This method is always affected by scale effects. A review on this topic was reported by Powell et al. (1985). From Dutch large-scale model tests stability numbers were published (den Boer et al., 1983; TAW/CUR-VB, 1984; Pilarczyk, 1987). Results of pressure measurements in the filter layer as well as results of numerical and analytical calculations were published by Bezuijen et al. (1986) and (1987). A theoretical treatment of the behaviour of loose blocks on slopes under wave action was given by Townson (1987). With reference to the bottom protection structure with loose blocks placed on granular sublayers at the EIDERDAMM storm surge barrier unexpected failure mechanisms were considered theoretically by Führböter (1986b).

The new research facility LARGE WAVE CHANNEL makes it possible to carry out full-scale investigations on dykes and revetments at German coastal regions. The main dimensions of the channel are: depth 7.0 m, width 5.0 m and length 324 m. Regular waves and random seas are produced mechanically by a wave generator. The maximum wave height is 2.5 m. Details about the LARGE WAVE CHANNEL were published by Grüne and Führböter (1975). Design criteria and technical works were reported by Grüne and Sparboom (1982).

2. PROTOTYPE DYKE STRUCTURES

The core profiles of the prototype dykes were constructed by sand. The compact cover layers were built by asphalt-concrete in a thickness of nearly 0.20 m. During the wave tests the core was drained to avoid dyke failures caused by positive water pressure below the impermeable cover layer.

With regard to modern dyke protection works the faces of the prototype dykes were constructed with uniform slopes 1:4 and 1:6 (Sparboom, 1987). The investigated asphalt dykes (Fig. 1) were supplied with measuring devices to obtain calibrated electrical signals for

- wave parameters at the toe of the dyke,
- wave impact pressures on the slope surface,
- wave run-ups on the slope and
- wave impact pressures in the subsoil.

The waves were generated regularly under absorption control. For each test nearly 200 waves were generated. Additionally, test series with narrow banded wave spectra were carried out.

A cross-section of the prototype revetment is given in Fig. 2. The area of the highest expected wave attack was subdivided into two testfields each of 10 m in length and 2.5 m in width. Out of the testfields the blocks were directly placed on a geotextile mat with a stabilization layer (needle punched fabric TERRAFIX 904 RS). The permeability of this filter was about $k_f \approx 3 \cdot 10^{-3}$ m/s⁻⁴ and ten times greater than the estimated sand permeability ($k_s \approx 3 \cdot 10^{-4}$ m/s).

There were tested four structural variants in the testfields (Fig. 3). In all cases the blocks were placed in stretcher-bond equivalently to practice.

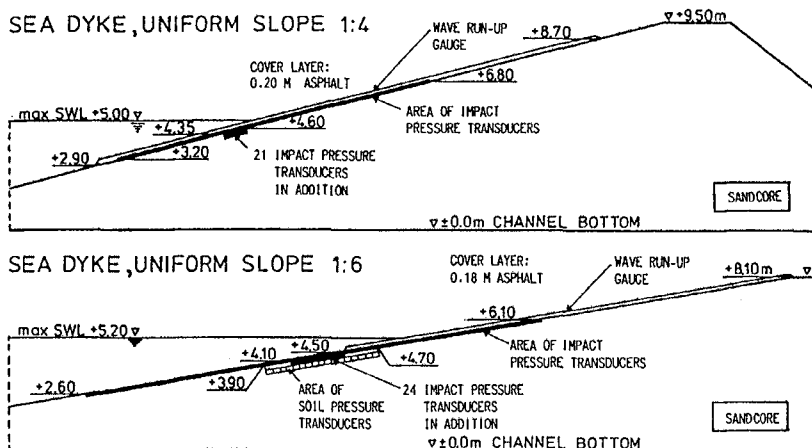


Figure 1. Investigated prototype sea dykes with impermeable top layers in the LARGE WAVE CHANNEL

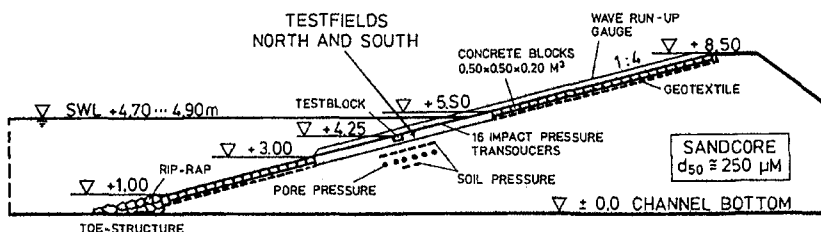


Figure 2. Prototype revetment in the LARGE WAVE CHANNEL

The measuring equipment (Fig. 2) was designed to registrate synchronous electrical signals for:

- wave parameters at the toe of the dyke revetment,
- wave impact pressures acting on the testblocks (top, bottom, each gap side),
- soil pressures beneath the top layer,
- pore pressures in the subsoil,
- block motions,
- wave run-ups on the slope and
- wave impact pressures on the slope between the testfields.

The waves were generated regularly applying an integrated absorption control system. The minimum wave number for each test was 200. After the first block failure the test was interrupted.

3. SHOCK PRESSURE INVESTIGATIONS

A typical record of a shock pressure (wave impact) due to wave breaking (plunging breaker type) on the slope can be seen in Fig. 4.

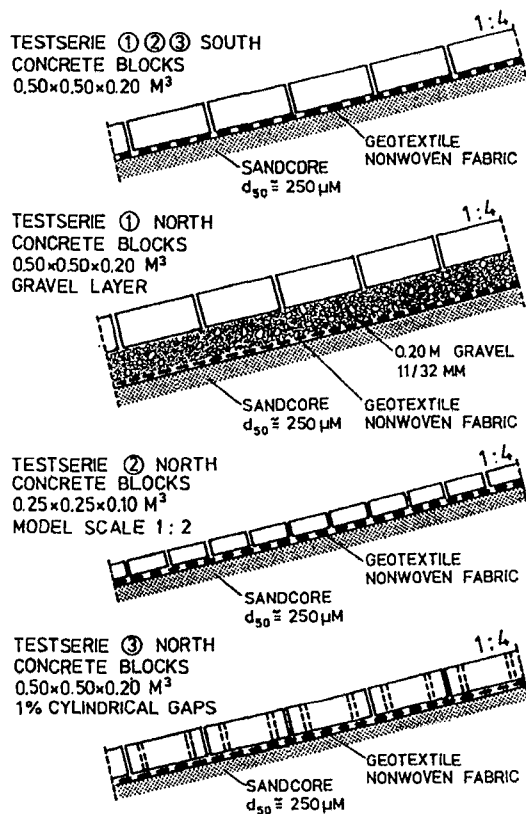


Figure 3. Cross-sections of investigated placed block revetments

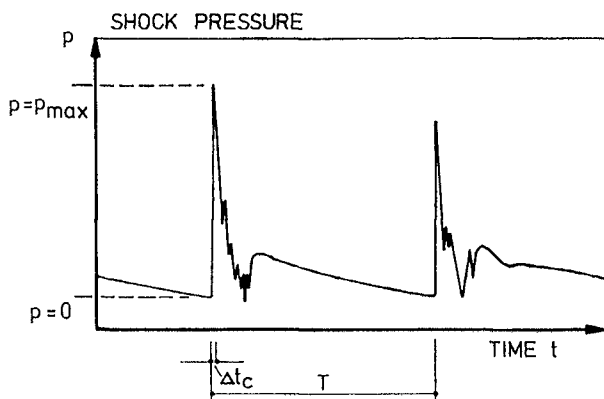


Figure 4. Typical shock pressure caused by breaking waves (plunging breaker type)

The peak pressure which is much higher than the wave or breaker height is highly effective during a relative short time. Due to the included airvolume by the plunging breaker the shock pressure occurrence can be explained by a compression phase. The time of compression Δt_c of all evaluated impacts lies between 10 and 60 milliseconds. The amplitude as well as the time history of shock pressures are strongly dependent on

- breaker process and aeration,
- angle of the sloping face,
- thickness of the backrush-water and
- wave characteristics (regular and random).

3.1 Probability theory

In the analysis of jet impacts (Führbötter, 1966 and 1969) the maximum pressure $p_{\max}$ is described by

$$p_{\max} = \delta \cdot \rho \cdot v \cdot c \cdot (c/v)^{1/3} \quad (1)$$

with $\delta = (E_a/E_w \cdot R/D)^{2/3}$

- E_a and E_w = elasticity of air and water
 R = hydraulic radius of impact area
 D = representative thickness of included air content
 δ = dimensionless impact number
 ρ = density of water
 v = impact velocity vertical to the wall
 c = sound velocity in water.

In this equation the air content term expressed by D (see Bagnold, 1939) is strongly stochastic and has an important influence on the impact as well as the hydraulic radius R of the impact area and the relation E_a/E_w . The following transformation of Eq. (1) yields:

$$p_{\max} = (E_a/E_w \cdot R/D)^{2/3} \cdot \rho \cdot v \cdot c \cdot (c/v)^{1/3} \quad (2)$$

stochastic $\downarrow$ deterministic

$$\log p_{\max} = 2/3 \log E_a/E_w + 2/3 \log R - 2/3 \log D$$

(stochastic) (3)

+ $\log (\rho \cdot v \cdot c \cdot (c/v)^{1/3})$ (deterministic).

In this way a linear function is obtained for $\log p_{\max}$ and $\log D$. If the stochastic term is distributed after GAUSS, $p_{\max}$ must consequently follow a Log-Normal distribution (Führbötter, 1966; see also Weggel, 1971). Comparing results on jet impacts, wave impacts from 1:10 scaled model and from prototype (scale 1:1) in the LARGE WAVE CHANNEL Führbötter (1986a) concluded that the Log-Normal distribution is valid for impact pressures in general as predicted by theory. Impact pressures due to breaking waves can therefore never be described deterministically but only probabilistically by the Log-Normal distribution.

With special reference to wave impacts on slopes Führbötter (1986a) extended Eq. (1) to

$$p_{\max} = \epsilon \cdot \rho \cdot g \cdot H_B \cdot (y_B/H_B)^{1/3} \cdot ((1+2 \cdot n^2)/(1+n^2))^{1/3} \quad (4)$$

with $\epsilon = (c^2/2g \cdot E_a/E_w \cdot d_1/H_B \cdot 1/D)^{2/3}$

- ϵ = dimensionless impact number for breaking waves on slopes
 H_B = breaker height
 y_B = breaker crest above the bottom
 d_1 = length of the impact area on the slope

g = acceleration due to gravity

n = front slope 1:n .

Considering practical design methods of sea dykes the relation between wave height and impact pressure seems to be most convenient (Führbötter, 1986a):

$$p_{\max} = p_i = \text{const.} \cdot \zeta \cdot g \cdot H \quad (5)$$

with $i = 50, 90, 99, 99.9 \%$.

3.2 Shock pressure experiments

Maximum shock pressures were evaluated from each individual breaking wave. As already shown by Führbötter (1986a) the maximum shock pressure values follow a Log-Normal distribution. In Fig. 5 and 6 the results for $i = 50, 90, 99, 99.9 \%$ are plotted as dimensionless relative impact pressures. Each line represents the results of a test serie with nearly 200 waves. For this wave number sufficient estimates were reached up to a level of 99.9 % probability.

In order to get informations about the spatial distribution of shock pressure occurrence the measured pressures (p_i) of each transducer at the slope were determined for each test cycle. Fig. 7 represents results of two test cycles. The length of the shock pressure area is strongly dependent on the wave parameters. The mean level of the highest maximum shock pressure, evaluated relatively to the wave height is drawn in Fig. 8 versus the relative shock pressure. The mean occurrence point for the slope 1:6 can be characterized by 0.5 H below SWL. The same value was found by Stive (1984) for steeper slopes 1:3 and 1:4.

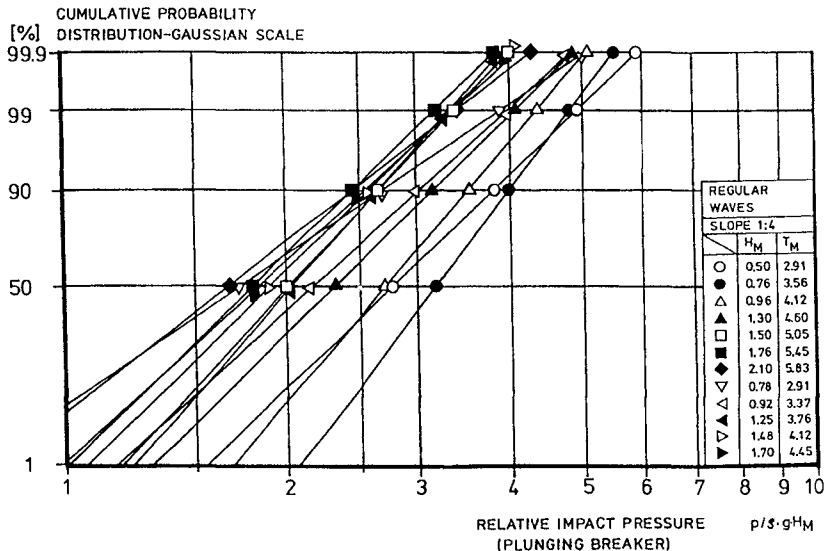


Figure 5. Shock pressure results, slope 1:4

Especially at the slope 1:6 shock pressures induced by simulated sea states were investigated. Representatively, in Fig. 9 relative shock pressures of a test with a significant wave height of 1.5 m and a peak period of 6.0 s are drawn against the well-known breaker number

(IRIBARREN-number). Maximum pressures occurred at breaker numbers from 0.6 to 1.8. The highest pressures were found at a breaker number of nearly 1.2.

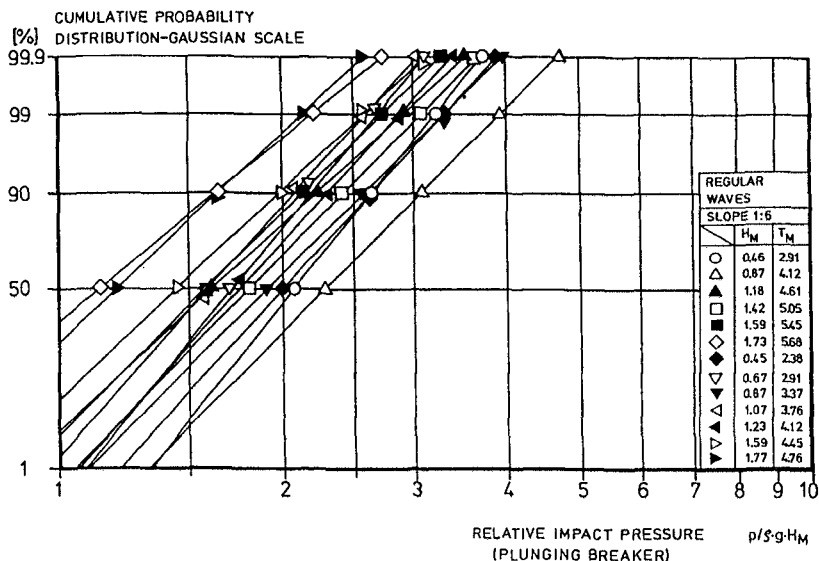


Figure 6. Shock pressure results, slope 1:6

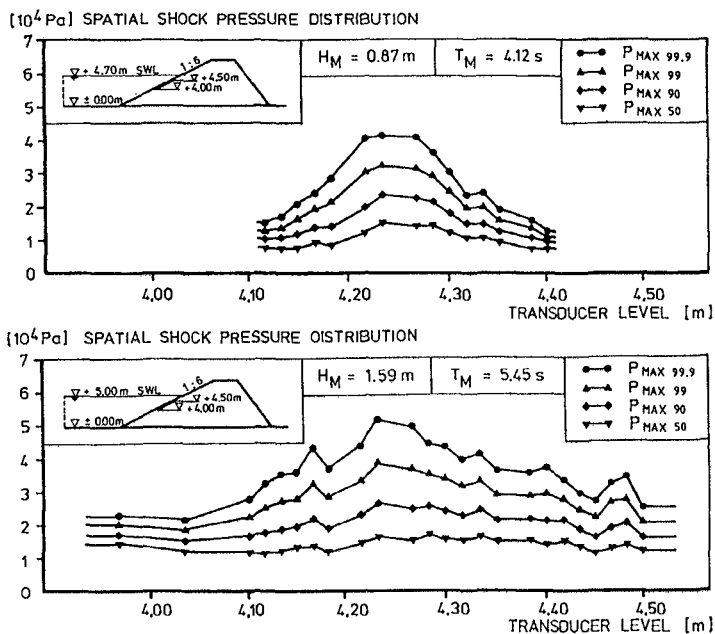


Figure 7. Spatial distributions of shock pressures on the slope 1:6

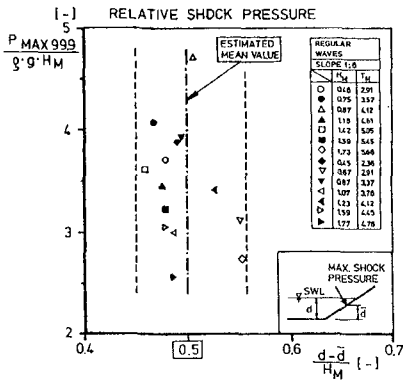


Figure 8. Area of maximum shock pressure occurrence for the slope 1:6

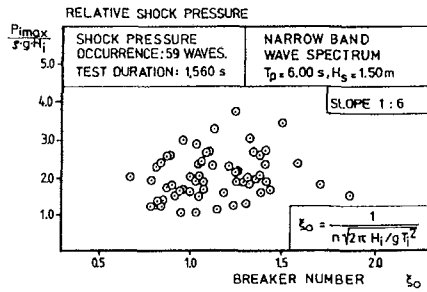


Figure 9. Random wave impact vs. breaker number, slope 1:6

3.3 Maximum shock pressure estimation

In Fig. 10 there are compared full-scale shock pressure results obtained by test cycles with regularly generated waves with those obtained by field measurements at the EIDERDAMM storm surge barrier and by test cycles with narrow banded wave spectra in the channel. Looking towards a practical approach of sea dyke design the maximum shock pressure distribution of the worst case is estimated with a tendency to the safe side (line with circled points).

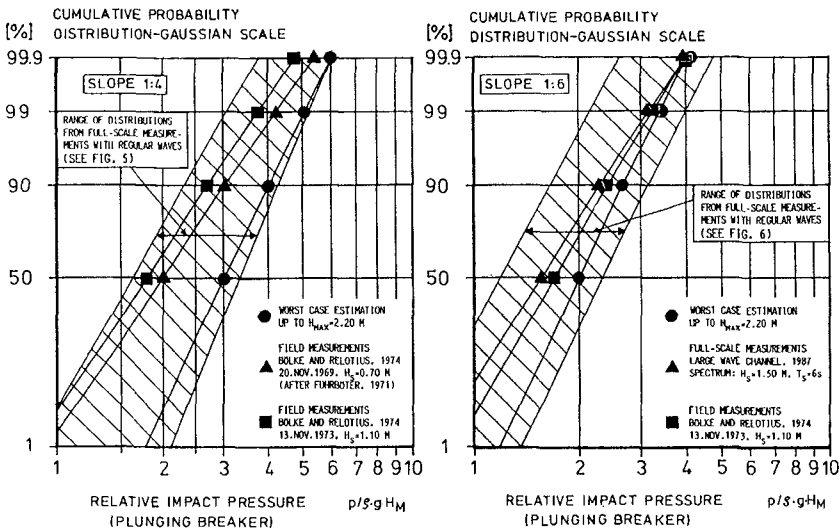


Figure 10. Full-scale shock pressure distributions

Based upon the worst case distributions (Fig. 10) for both slopes 1:4 and 1:6 it seems to be possible to establish a formula which contains the influence of the slope angle:

$$p_{\max} = p_i = \text{const} \cdot 1/n \cdot f \cdot g \cdot H \quad (6)$$

with $i = 50, 90, 99, 99.9 \%$ and
 $n = \text{front slope } 1:n.$

The empirical Eq. (6) may be justified by the important damping influence of the backrush-water on the impact amplitude. The thickness of the backrush-water increases with flatter slopes and the impact pressure decreases proportionally. The application of this formula to slopes in general is proposed in Fig. 11. With respect to the real experiments with 1:4 and 1:6 slopes it should be considered that only for the range 1:3 to 1:8 a high reliability can be achieved. For the range 1:1 to 1:3 the values are extrapolated and should therefore be considered very cautiously. Based upon jet impact investigations Führbötter (1966) also determined relative impact pressures for the vertical wall. Expected values for breaker heights of 1.0 m are added in Fig. 11. Future full-scale investigations on very steep slopes up to the vertical wall could be useful to proof the proposed empirical formula.

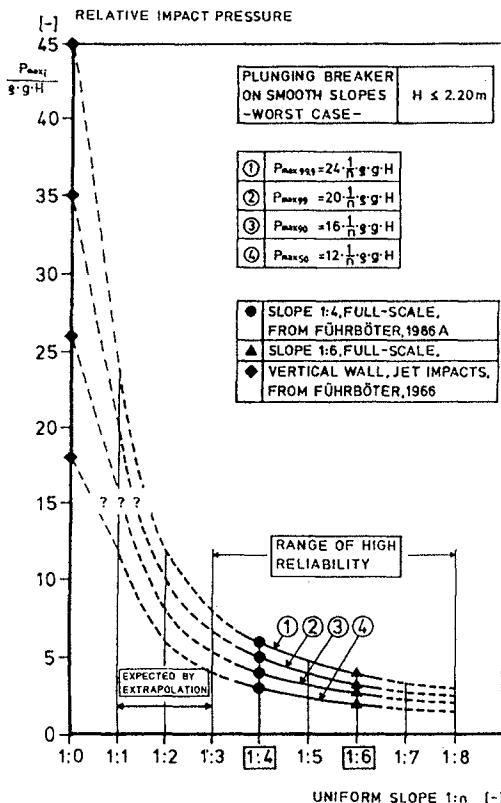


Figure 11. Generalized proposal for breaker-induced maximum shock pressures on smooth slopes

4. INVESTIGATIONS ON CONCRETE BLOCK SLOPE REVETMENTS

First experiments with permeable prototype revetments in the LARGE WAVE CHANNEL were used to explain statements of the following problems:

- Which real wave parameters (height and period) are responsible for the instability of natural revetments ?
- At which wave phase and in which location do very first failures occur ?
- How does a gravel layer (filter base) influence the stability of a revetment ?
- Which scale effects can be observed using a 1:2 model in revetment design ?
- How does an increase of the top layer permeability influence the stability of a revetment ?

4.1 Stability results

Usually, blocks or stones of seawalls are designed applying the HUDSON formula. The wave period is not considered in this approach. Recent investigations have shown clearly that the block stability depends on the breaker number ξ (Bruun, 1985). This parameter contains the influence of the wave period (or better wave steepness) and the slope angle.

Pilarczyk (1987) proposed a stability formula for concrete block revetments:

$$H_S / \Delta \cdot d_B = \gamma \cdot \cos \alpha / \xi_z^{1/2} \quad (7)$$

with H_S = significant wave height

Δ = relative block density

d_B = block thickness

α = slope angle

ξ_z = breaker number $\tan \alpha / (2\pi H_S / g T_z^2)^{1/2}$

T_z = mean wave period

γ = Stability coefficient defined for $\xi_z = 1$ in categories I to V.

Placed blocks which are treated in this paper should therefore be classified in categorie II ($3 < \gamma < 4$). The permeability of the top layer and of the filter layer as well as the horizontal dimensions of the single blocks are not taken into account. Special problems occur if the results of laboratory tests with regular waves are to be transferred to natural structures under real sea state conditions (wave spectra).

With regard to recent results on the block stability the parameters $H / \Delta \cdot d_B$ and ξ_o are considered here.

H = wave height (regularly generated)

Δ = relative block density

$\xi_b = \xi_w / \xi_o$

d_B = block thickness

ξ_o = breaker number in deep water

$\xi_o = 1 / (n \cdot (H / L_o)^{1/2})$

with $L_o = g \cdot T^2 / 2\pi$

n = front slope 1:n

L_o = wave length in deep water

T_o = wave period (regularly generated)

g = acceleration due to gravity.

Fig. 12 represents results of testseries ①, ② and ③. Safety is guaranteed up to $H/\Delta \cdot d_B = 4$. Comparing these results with those in Fig. 13 which refer to a 1:2 scaled model the same upper bound can be stated. But, failures seems to occur at higher ξ_0 - values. Probably only the unit of length should be scaled up and not the unit of time simultaneously. This problem due to scaling may necessitate more systematic tests in the future.

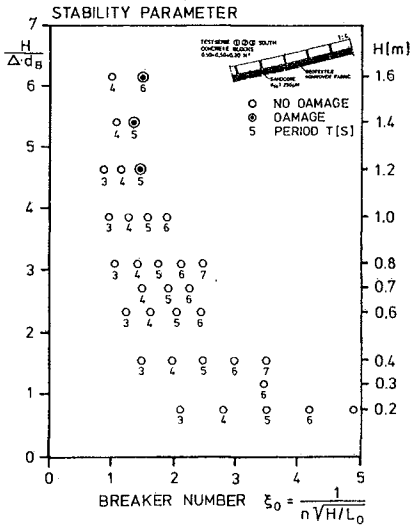


Figure 12. Stability numbers
Testseries ①②③
South; Full-scale

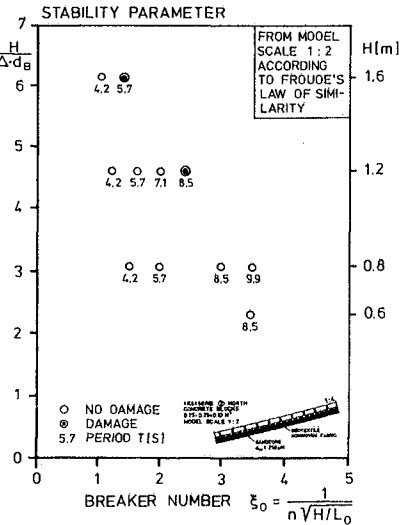


Figure 13. Stability numbers
Testserie ② North;
Model scale 1:2

In Fig. 14 there are plotted stability results for the structure with an added gravel layer. $H/\Delta \cdot d_B \approx 2.5$ is estimated as the maximum safe number. The influence on the stability by increasing the permeability of the top layer is seen in Fig. 15. Safe conditions can be expected for $H/\Delta \cdot d_B \approx 5.5$. In the last test vertical block motions in the range of 2 to 3 cm were observed. It was not possible to simulate the real failure because the revetment in the other testfield was totally damaged.

All observed failures occurred after about 65 to 110 waves. The location of the first block liftings can roughly be given with 0.5 H beneath SWL. Failures of the geotextile filter mats did not happen. Sandtightness was guaranteed over all test series (about 10,000 waves).

4.2 Initial block lifting

Placed block stability depends clearly on wave breaking characteristics. In the range $1 \leq \xi_0 \leq 1.5$ block failure was mainly observed. This is the same range in which Führböter (1986a) found the maximum wave impacts on an impermeable 1:4 sloping asphalt dyke. Wave impacts are of short duration (milliseconds) and therefore they can not cause block liftings out of the top layer. But wave impacts probably cause a strong reduction of friction contact (or friction stress) between the single blocks. Test observations have shown that first block liftings occurred just in front of the progressive wave before breaking. In order to study pressure variations, transducers were installed in special test-

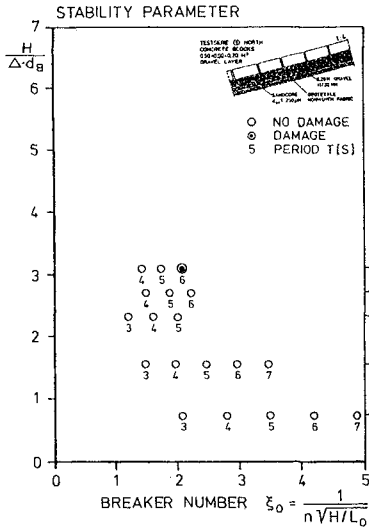


Figure 14. Stability numbers
Testserie ① North;
Full-scale

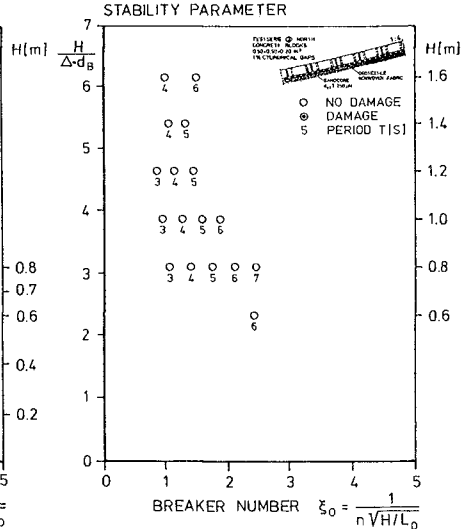


Figure 15. Stability numbers
Testserie ③ North;
Full-scale

blocks.

In Fig. 16 and in Fig. 17 synchronously measured pressures at top and bottom of a testblock are plotted. Additionally, the pressure difference is calculated. The positive sign indicates uplift pressures. The reaction time T_{up} is defined from the beginning of increase of Δp to the maximum of Δp before wave breaking. Dynamic impact pressures were not yet evaluated in detail. But nearly all impacts reacted in very short time and opposite to the uplift direction before wave breaking.

For about 35 waves of testserie ① ($H_0 = 0.70$ to 0.80 m, measured at the toe of the dyke; $T = 4$ to 6 s) the reaction time T_{up} was evaluated (Fig. 18). The wave conditions correspond to the beginning of block liftings above the granular layer. Although a large scatter is obvious it can be seen that for both filter layers the uplift pressure amplitudes are of nearly equal amount but differ definitely in the reaction time. Increasing duration of the uplift pressure seems to be a most important phenomenon causing block failures.

In testserie ③ long reaction times were also found in the geotextile filter just before failure occurrence, but the waves were much higher. The reaction time can obviously be reduced by an increase of the top layer permeability. This was also demonstrated in testserie ③ (Fig. 15).

5. CONCLUDING REMARKS

As already stated by Führeböter (1986a) shock pressure phenomena due to breaking waves acting on dyke slopes can only be described probabilistically by the LOG-normal distribution.

The thickness of the backrush-water has an important influence on the shock pressure occurrence. Prototype and field investigations (slope 1:4 and 1:6) show that a proportionality exists for shock pressures and slope angles (Eq. 6). For mostly used sloping faces of sea

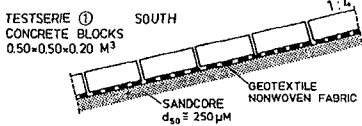
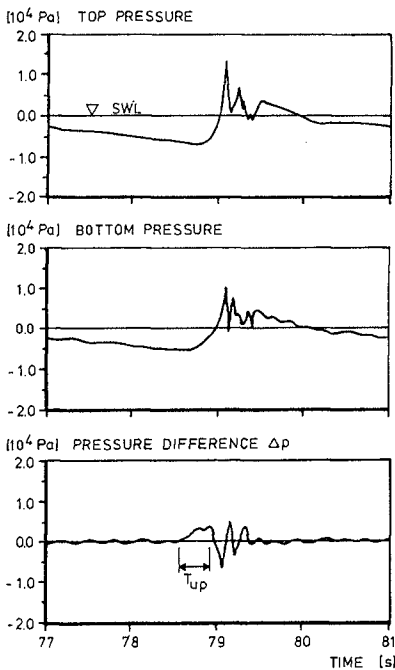


Figure 16. Block pressures
Testserie ① South

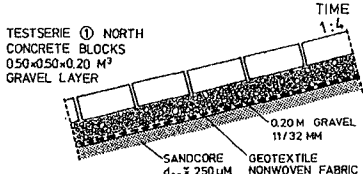
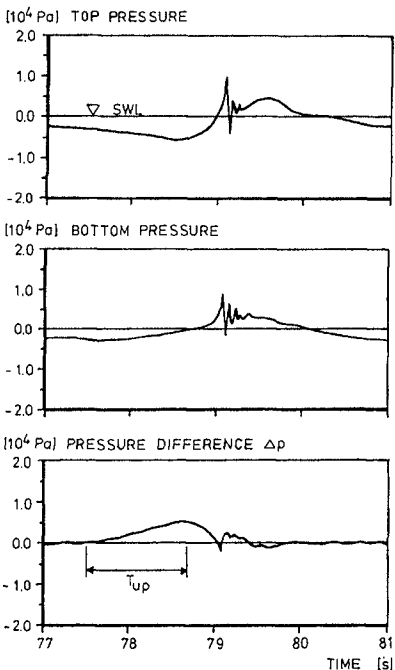


Figure 17. Block pressures
Testserie ① North

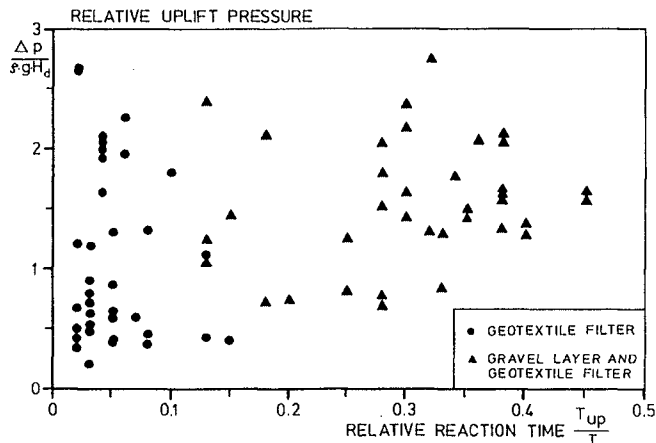


Figure 18. Relative uplift pressures versus relative
reaction time (Testserie ①)

dykes in shallow water regions the following maximum shock pressures, acting nearly 0.5 H below SWL on the slope surface are to be expected:

$$\begin{array}{ll} \text{SLOPE 1:4} & p_{\max} \approx 6 \cdot \zeta \cdot g \cdot H, \\ \text{SLOPE 1:6} & p_{\max} \approx 4 \cdot \zeta \cdot g \cdot H. \end{array}$$

Full-scale experiments on wave breaking phenomena are indispensable to avoid scale effects in small-scale modelling. Together with field investigations under real sea state conditions (Grüne, 1988) full-scale measurements guarantee highly reliable results for future design on sea dykes.

From the first analysis of the comprehensive full-scale measurements dealing with placed block revetments stability criteria are found. If the blocks are normally placed in stretcher-bond on a 2-dimensional geotextile filter, the stability parameter $H / \Delta \cdot d_b$ should not exceed 4. 1:2 scaled models can be used but variations in breaker characteristics should be considered carefully.

A 3-dimensional granular filter layer in a thickness equal to the block thickness reduces the stability considerably ($H / \Delta \cdot d_b \leq 2.5$).

Block failure occurs if the reaction time of the uplift pressure rises up to at least 1/3 of the wave period. Uplift pressure values roughly amount 2 to 3 times of the wave height measured at the toe of the dyke.

Better block stabilities can be reached by increasing the top layer permeability. For placed blocks on a geotextile filter the stability number $H / \Delta \cdot d_b \leq 5.5$ is obtained by uniformly distributed cylindrical gaps (1 % of the block surface).

Reported results can be applied to loose concrete blocks placed in stretcher-bond. Favourable stability effects due to block interlocking or connecting are not yet considered. It is proposed to carry out large-scale experiments for such block systems to find out most economical solutions.

ACKNOWLEDGEMENTS

The authors would like to acknowledge gratefully the German Research Foundation (DEUTSCHE FORSCHUNGSGEMEINSCHAFT) sponsoring the research program on safety and reliability of dykes and revetments for coastal protection (SFB 205 / A1).

REFERENCES

- Bagnold, R.A.: Interim Report on Wave Pressure Research. Journal of the Institution of Civil Engineers, Vol. 12, London, 1939
- Bezuijen, A.; Klein Breteler, M.; Pilarczyk, K.W.: Large-Scale Tests on a Block Revetment Placed on Sand with a Geotextile as Separation Layer. Proc. 3rd Int. Conf. on Geotextiles, Vienna, 1986
- Bezuijen, A.; Klein Breteler, M.; Bakker, K.J.: Design Criteria for Placed Block Revetments and Granular Filters. Proc. 2nd Int. Conf. on Coastal and Port Engineering in Developing Countries, Beijing, 1987
- Bölke, S. and Relotius, P.C.: Über die wellenerzeugten Druckschlagbelastungen von Seedeichen im Böschungsbereich zwischen 1:4 und 1:6. Mitt. Leichtweiß-Institut, Tech. Univ. Braunschweig, Heft 42, 1974 (in German)
- den Boer, K.; Kenter, C.J.; Pilarczyk, K.W.: Large Scale Model Tests on Placed Block Revetments. Proc. Coastal Structures '83, Arlington, 1983
- Brun, P.: Design and Construction of Mounds for Breakwaters and Coastal Protection. Elsevier, Amsterdam, 1985

- Führbötter, A.: Der Druckschlag durch Brecher auf Deichböschungen. Mitt. Franzius-Institut, Tech. Univ. Hannover, Heft 28, 1966 (in German)
- Führbötter, A.: Laboratory Investigations of Impact Forces. Proc. Symp. Research on Wave Action, Tech. Univ. Delft, Paper No. 6, Delft, 1969
- Führbötter, A.: Zufallsprozesse bei der Belastung durch brechende Wellen. Jahresbericht 1970, Sonderforschungsbereich 79, Tech. Univ. Hannover, 1971 (in German)
- Führbötter, A.: Die Forschungseinrichtung "GROSSER WELLENKANAL" (Hannover) und ihre Aufgaben. Berichtswerk Intermaritec '82, Hamburg, 1982 (in German)
- Führbötter, A.: Model and Prototype Tests for Wave Impact and Run-Up on an Uniform 1:4 Slope. Coastal Engineering, Elsevier Science Publishers, 10, 1986a
- Führbötter, A.: Hydrodynamische Belastungen der Sohlsicherung des Eidersperrwerkes. Bauingenieur 61, Springer-Verlag, 1986b (in German)
- Führbötter, A.; Dette, H.H.; Grüne, J.: On the Response of Seadykes due to Wave Impacts. Proc. 15th Int. Conf. on Coastal Engineering, Honolulu, 1976
- Grüne, J.: Wave Induced Shock Pressure under Real Sea State Conditions. Proc. 21th Int. Conf. on Coastal Engineering, Malaga, Spain, 1988
- Grüne, J.; Führbötter, A.: Large Wave Channel for "Full Scale Modelling" of Wave Dynamics in Surf Zones. Proc. Symp. on Modelling Techniques, San Francisco, 1975
- Grüne, J.; Sparboom, U.: Großer Wellenkanal-Problemmstellung und Lösung aus versuchstechnischer und konstruktiver Sicht. Technischer Bericht, Sonderforschungsbereich 79, Universität Hannover, 1982 (in German)
- Heerten, G.: Geotextiles in Coastal Engineering—25 Years Experience. Geotextiles and Geomembranes, Vol. 1, Elsevier Applied Science, Amsterdam, 1984
- Kostense, J.K.; den Boer, K.: Effect of Model Scale on the Stability on Concrete-Block Slope Revetments. Proc. Symp. on Scale Effects in Modelling Hydraulic Structures, Esslingen, 1984
- Pilarczyk, K.W.: Dutch Guidelines on Dyke Protection. Report WB-NO-87110, Rijkswaterstaat, Delft, 1987
- Powell, K.A.; Allsop, N.W.H.; Owen, M.W.: Design of Concrete Block Revetments Subject to Wave Action—A Literature Review. Rep. No SR 54, Hydraulics Research Limited, Wallingford, 1985
- Skladnev, M.F. and Popov, J.Ya.: Studies of Wave Loads on Concrete Slope Protections of Earth Dams. Proc. Symp. Research on Wave Action, Tech. Univ. Delft, Paper No. 7, Delft 1969
- Sparboom, U.: Konzeption von Deicheinbauten im GROSSEN WELLENKANAL. Technischer Bericht, Sonderforschungsbereich 205, Universität Hannover, 1987 (in German)
- Stephan, H.J.: Über Schäden an Seedeichen durch Wellen- und Druckschlagbelastungen. Mitt. Leichtweiß-Institut, Tech. Univ. Braunschweig, Heft 70, 1981 (in German)
- Stive, R.J.H.: Wave Impact on Uniform Steep Slopes at Approximately Prototype Scale. Proc. Symp. on Scale Effects in Modelling Hydraulic Structures, Esslingen, 1984
- TAW/CUR-VB-Technische Adviescommissie voor de Waterkeringen/Stichting voor onderzoek, voorschriften en kwaliteitseisen op het gebied van beton: Leidraad cementbetonnen dijkbekledingen. Rapport 119, Betonvereniging, Zoetermeer, 1984 (in Dutch)
- Townson, J.M.: A Lower Bound for the Thickness of Revetment Blocks. Proc. Institution of Civil Engineers, Part 2, Vol. 83, 1987
- Weggel, R.L.: Discussion on "Shock Pressures on Coastal Structures (by Kamel)". ASCE, J. of the Waterways and Harbour Division, WW 3, 1971
- Whillock, A.F.: The Stability of Revetment Blocks under Wave Attack. Rep. No. IT 195, Hydraulics Research Station, Wallingford, 1980