8 - Waves, erosion and stability

ct4310 – Bed, Bank and Shoreline Protection

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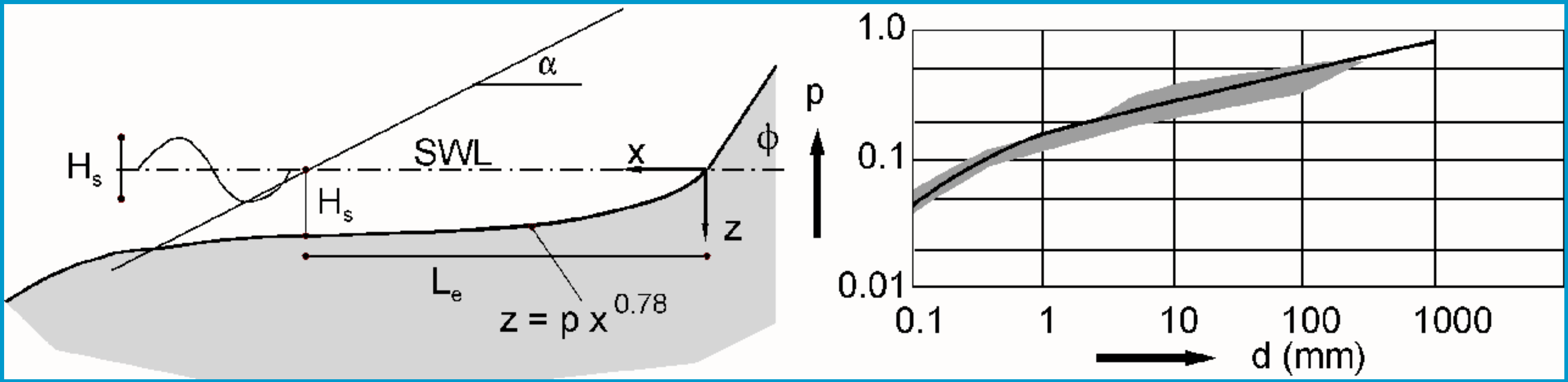
Introduction

- there is a relation between the slope angle and the grain size
- for sand, standard profile can be used
- For stability the value $H/\Delta d$ is important
  - $H/\Delta d < 1$ caissons or seawalls
  - $H/\Delta d = 1...4$ stable breakwaters
  - $H/\Delta d = 3...6$ S-shaped and berm breakwaters
  - $H/\Delta d = 6...20$ rock slopes
  - $H/\Delta d = 15...500$ gravel beaches
  - $H/\Delta d > 500$ sand beaches (during storm surge)

erosion of slope by waves

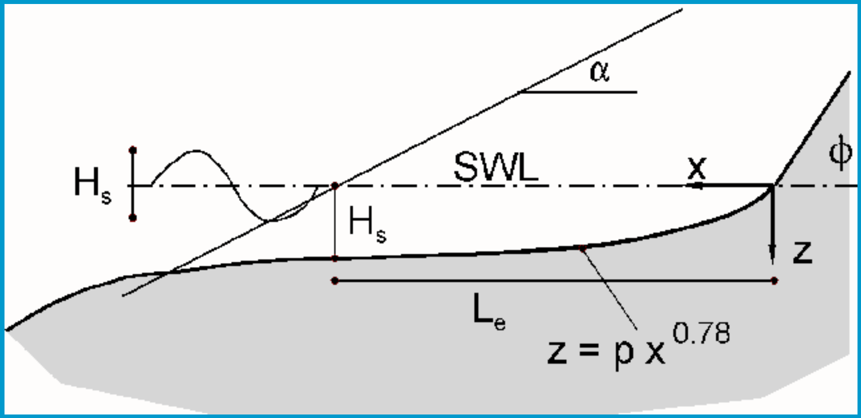


The Vellinga Profile

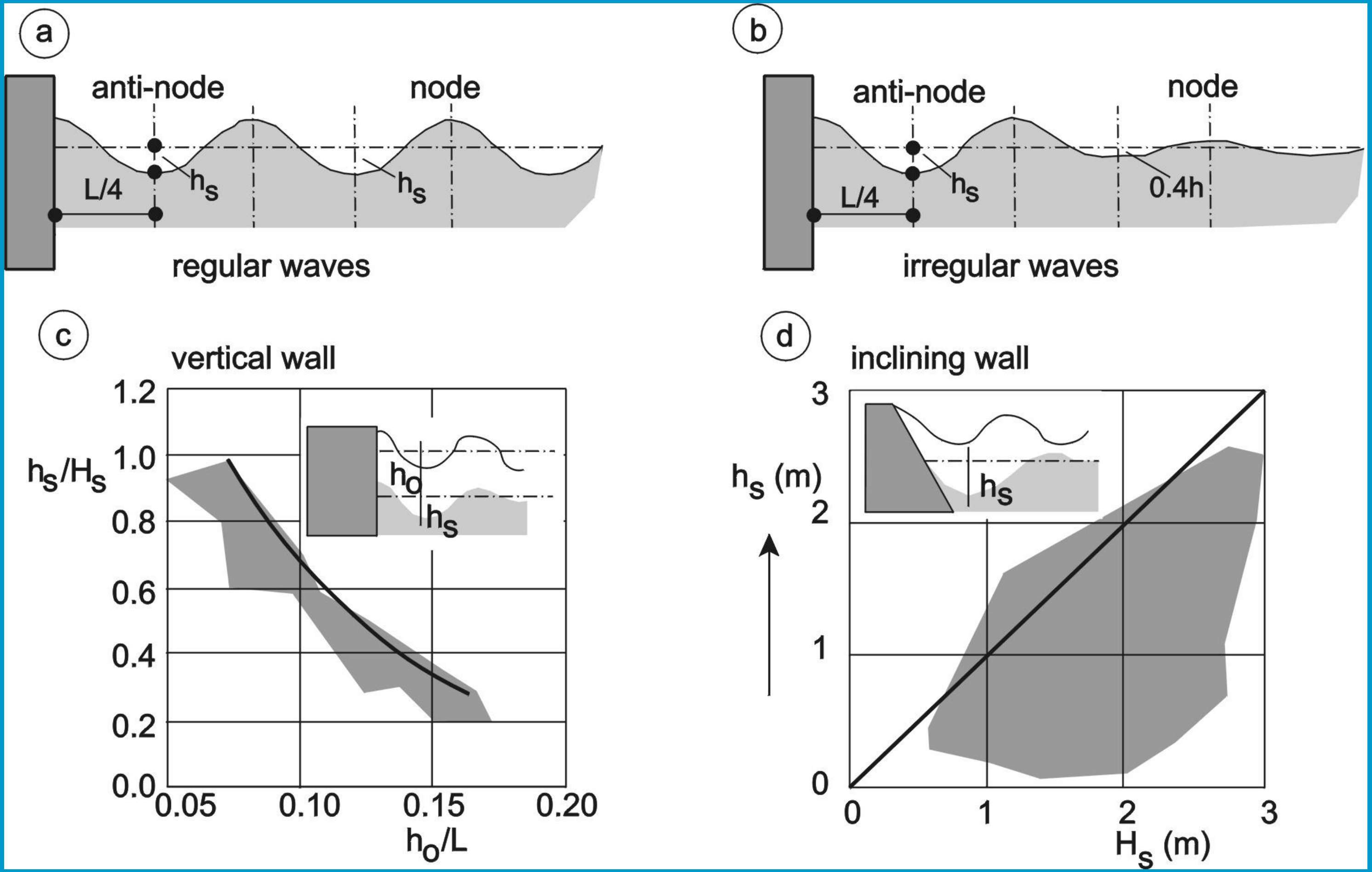
$$\left(\frac{7.6}{H_{0s}}\right) y = 0.4714\left[\left(\frac{7.6}{H_{0s}}\right)^{1.28}\left(\frac{w}{0.0268}\right)^{0.56} x + 18\right]^{0.5} - 2.00$$

$$z = 0.39\, w^{0.44}\, x^{0.78} = p\, x^{0.78}$$

$$L_e = p^{-1.28}\, H_s^{1.28}$$



bed erosion in front of a wall



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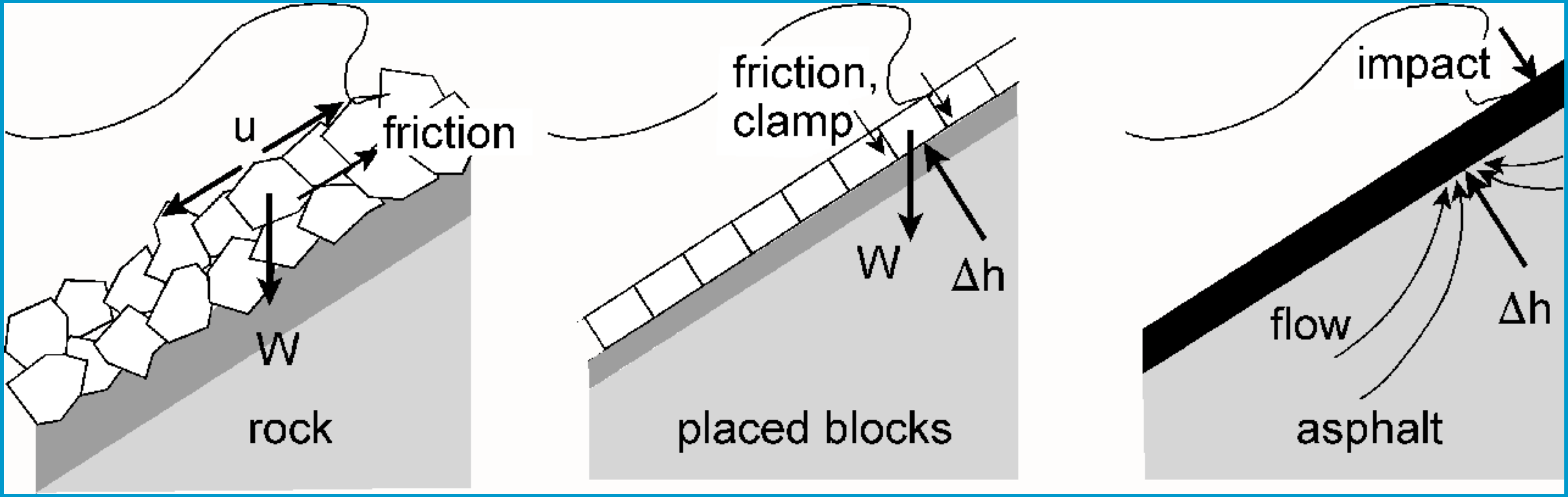
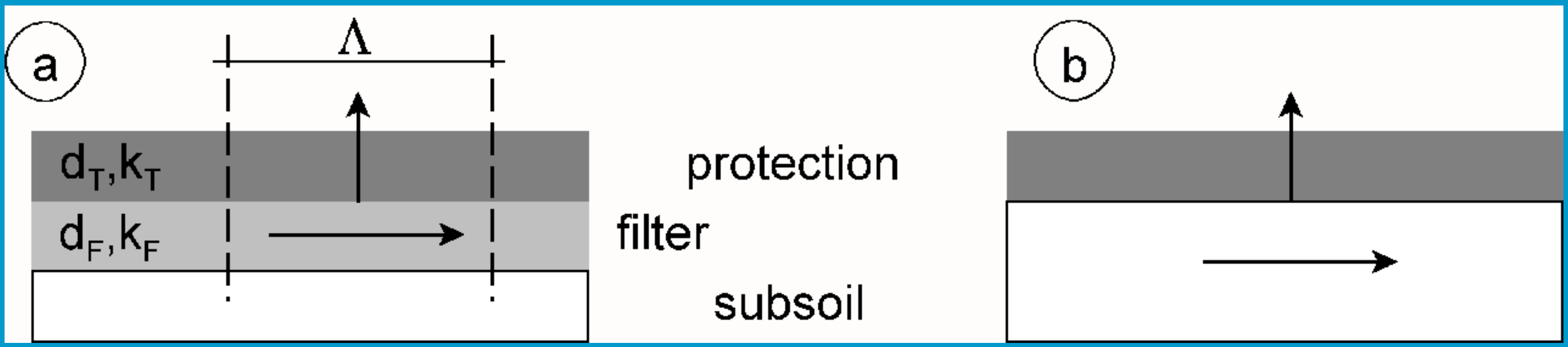
three main types of protection against waves



definition of leakage length



$$\frac{d_T}{k_T \Lambda} = \frac{\Lambda}{k_F d_F} \quad \rightarrow \quad \Lambda = \sqrt{\frac{k_F d_F d_T}{k_T}}$$

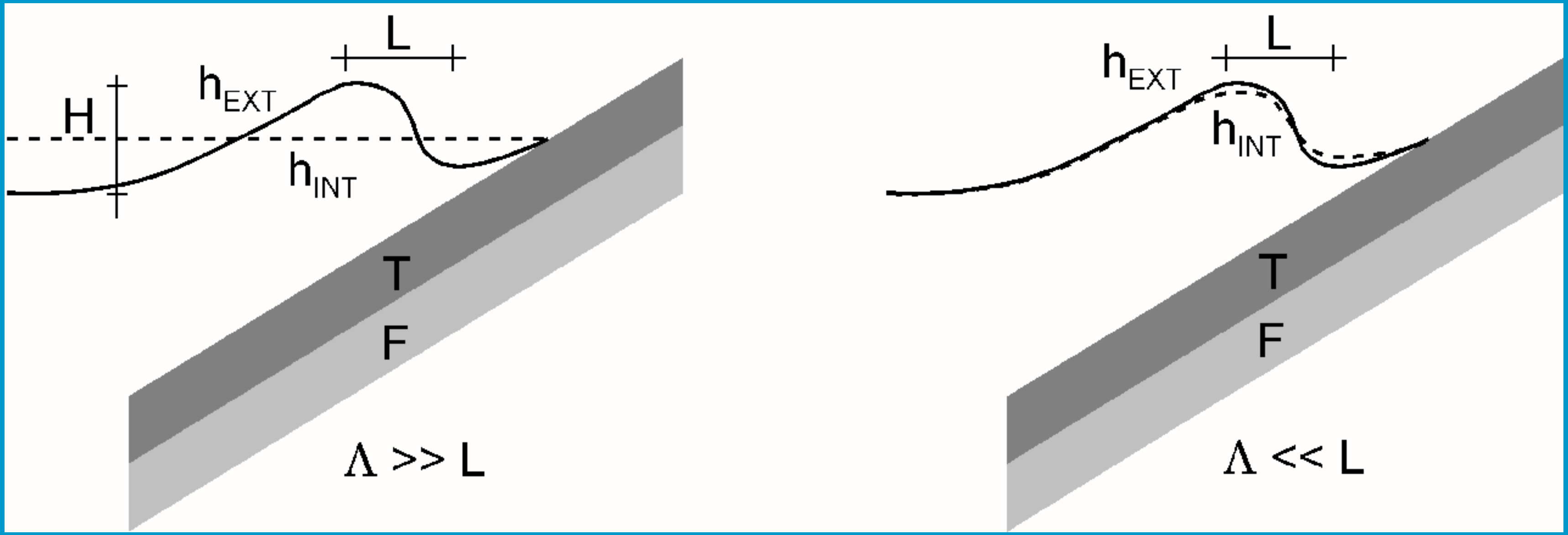
leakage length

$$\frac{d_T}{k_T \Lambda} = \frac{\Lambda}{k_F d_F} \quad \rightarrow \quad \Lambda = \sqrt{\frac{k_F d_F d_T}{k_T}}$$

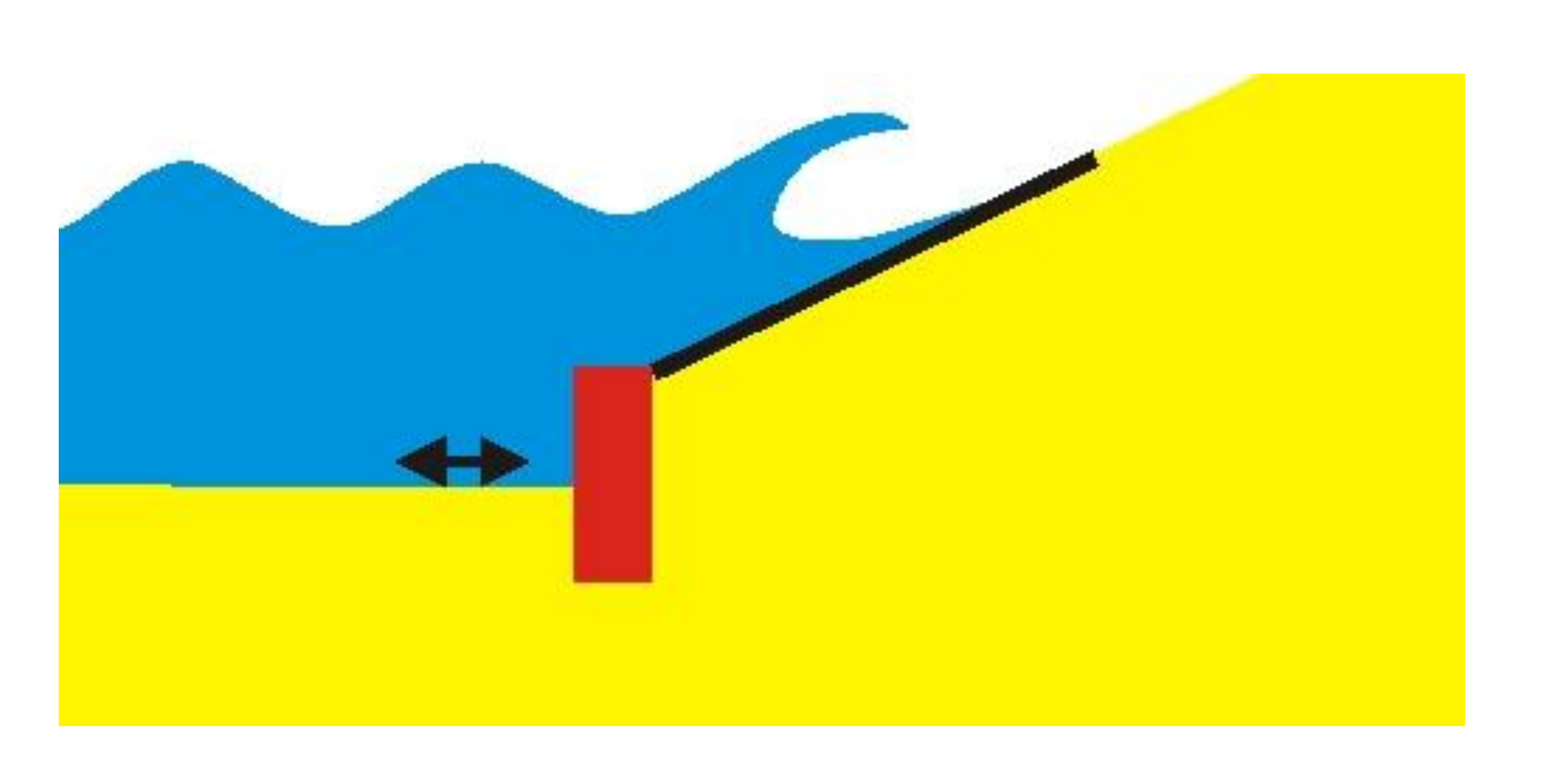
Parameter	"Rock"	"Blocks"	"Asphalt"
d_T (m)	0.5	0.25	0.25
d_F (m)	0.25	0.2	2
k_T (m/s)	0.5	0.001	"0"
K_F (m/s)	0.1	0.05	0.0001
Λ (m)	0.15	1.5	"∞"
L (m)	1-2	1-2	1-2

influence of leakage length

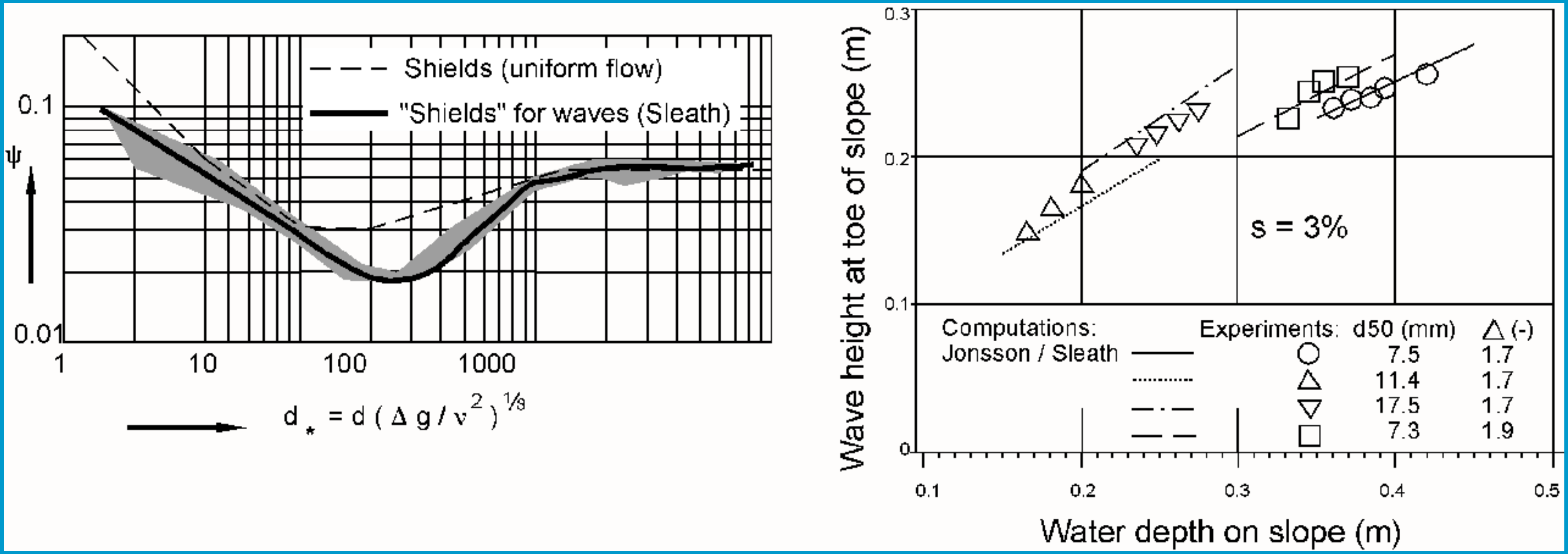
$$\frac{d_T}{k_T \Lambda} = \frac{\Lambda}{k_F d_F} \quad \rightarrow \quad \Lambda = \sqrt{\frac{k_F d_F d_T}{k_T}}$$



Erosion at the toe



modified Shields diagram for waves and stability in non-breaking waves

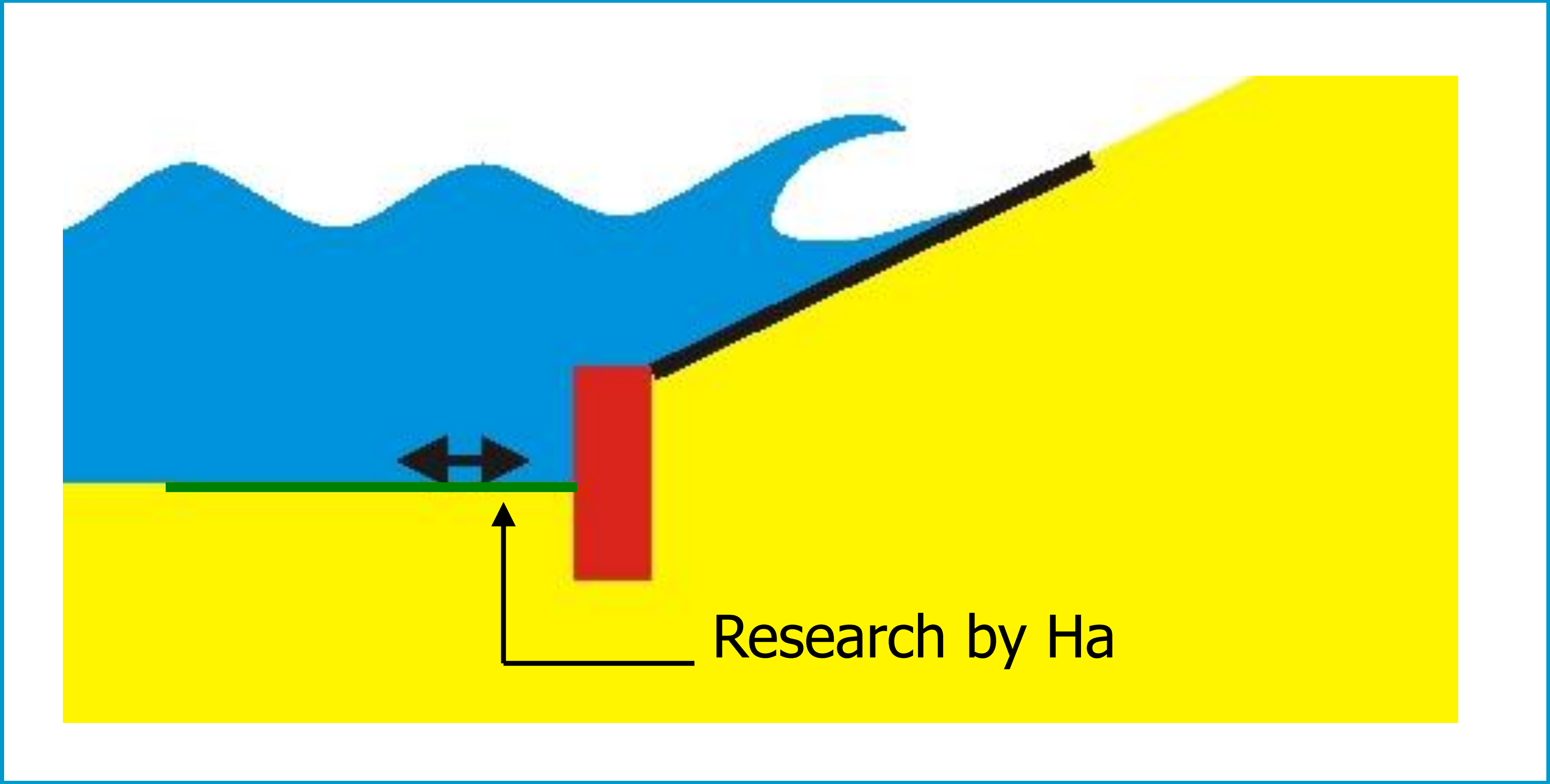


direct equation for bed stability

$$\frac{a_b}{T^2 \Delta g} = 0.025 \left(\frac{a_b}{d_{50}} \right) \quad \rightarrow \quad d_{n50} = 2.15 \frac{\hat{u}_b^{\,2.5}}{\sqrt{T} \left(\Delta g \right)^{1.5}}$$

a_b	orbital stroke at the bottom
$\hat{u}_b$	maximum orbital velocity
d_{n50}	assumed equal to 0.85 d_{50}
T	wave period

Erosion at the toe



stability on a slope

$$\rho_w\, g\, H\, d^2 \quad \propto \quad (\rho_s - \rho_w)\, g\, d^3 \quad (\tan\phi\cos\alpha \;\pm\; \sin\alpha)$$

"drag" force — resisting force — slope correction

$$M \propto \frac{\rho_s H^3}{\Delta^3 (\tan\phi\cos\alpha \;\pm\; \sin\alpha)^3}$$

Iribarren

$$M = \frac{\rho_s H_{sc}^{\;3}}{K_D \Delta^3 \cot\alpha} \qquad \left(\text{or: } \frac{H_{sc}}{\Delta d} = \sqrt[3]{K_D \cot\alpha}\right)$$

Hudson

limitations of Hudson

Not included in the equation:

- **wave period**
- **permeability**
- **storm duration**
- **damage level**

Van der Meer

$$\frac{H_{sc}}{\Delta d_{n50}} = 6.2\, P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \xi^{-0.5} \qquad \text{(plunging breakers)}$$

$$\frac{H_{sc}}{\Delta d_{n50}} = 1.0\, P^{-0.13} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \xi^{P} \sqrt{\cot \alpha} \qquad \text{(surging breakers)}$$

$$\xi_{\text{transition}} = \left[6.2\, P^{0.31} \sqrt{\tan \alpha} \right]^{\left(\frac{1}{P+0.5} \right)}$$

$\xi > \xi_{\text{transition}}$ → surging breakers

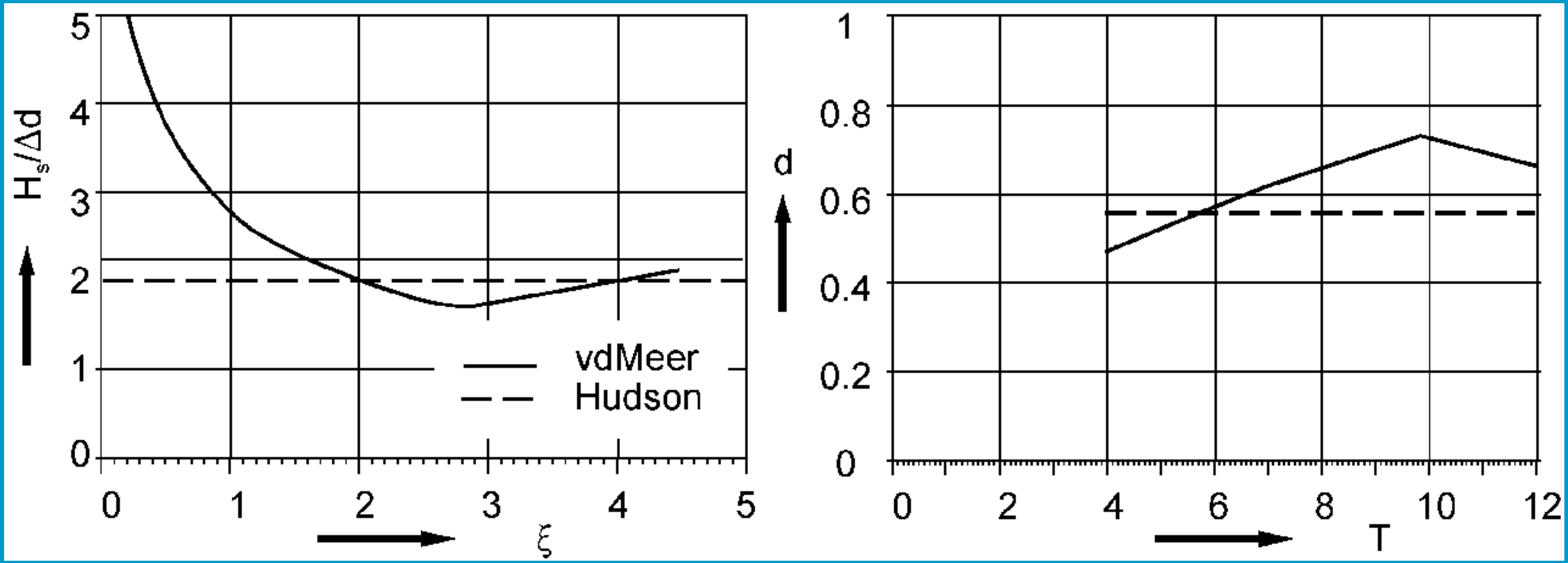
$\xi < \xi_{\text{transition}}$ → plunging breakers

Video: Rock slopes on gravel beaches
wbk 049 - 14 min

reference case

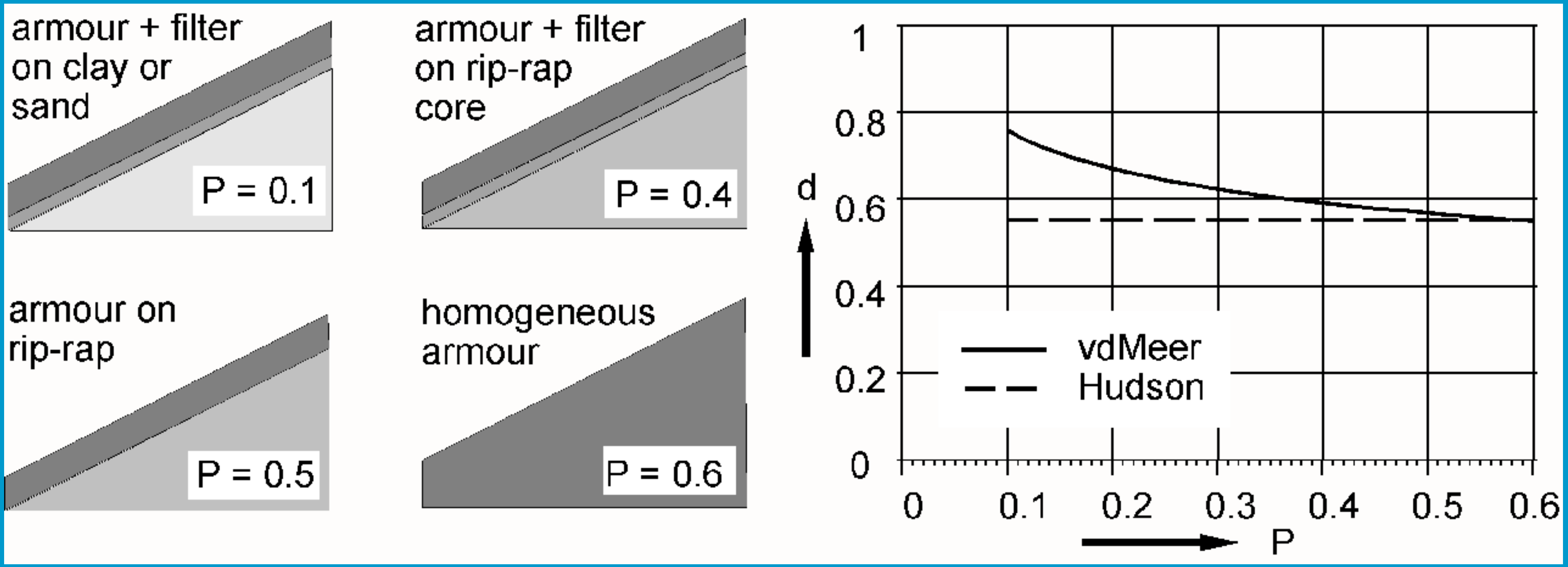
sign. wave height	H_s	2 m
slope of revetment	$\cot\alpha$	3
"Permeability"	P	0.5
mean period	T_m	6 s
number of waves	N	3000
rock size	d_{n50}	0.6 m (300-1000 kg)
relative density	Δ	1.65
damage level	S	2
Hudson coefficient	K_D	2

Wave period



5
4
3
2
1
0
$H_s/\Delta d$
0
1
2
3
4
5
ξ
vdMeer
Hudson
1
0.8
0.6
0.4
0.2
0
d
0
2
4
6
8
10
12
T

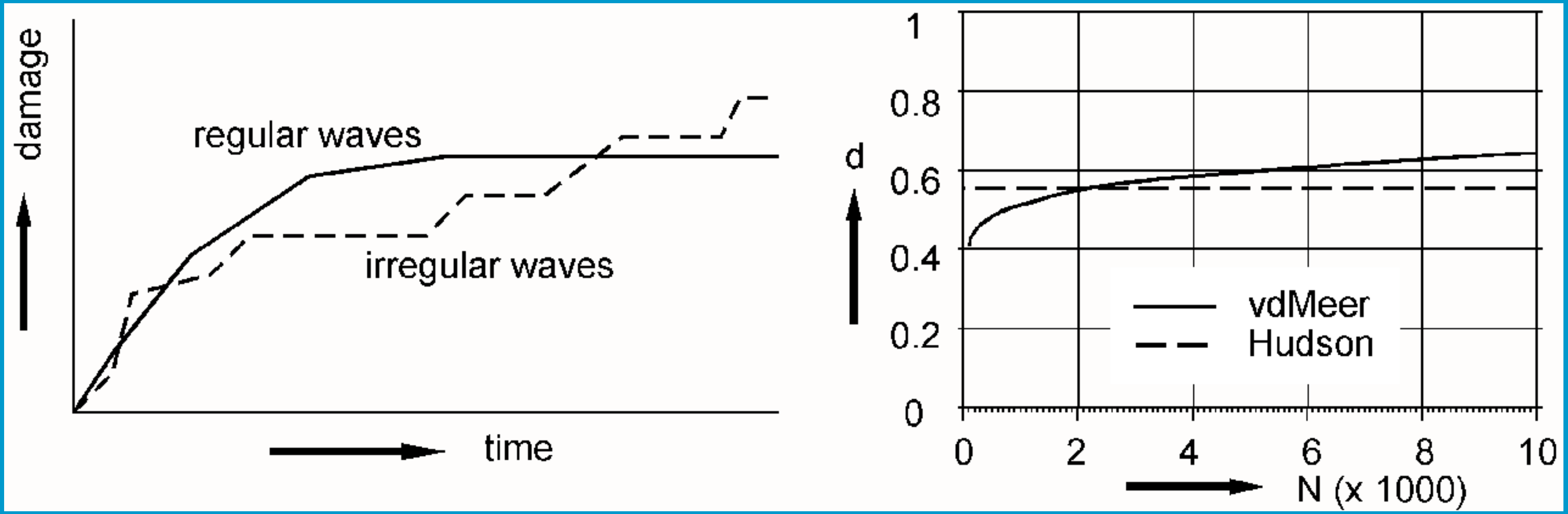
permeability



P = notional permeability factor

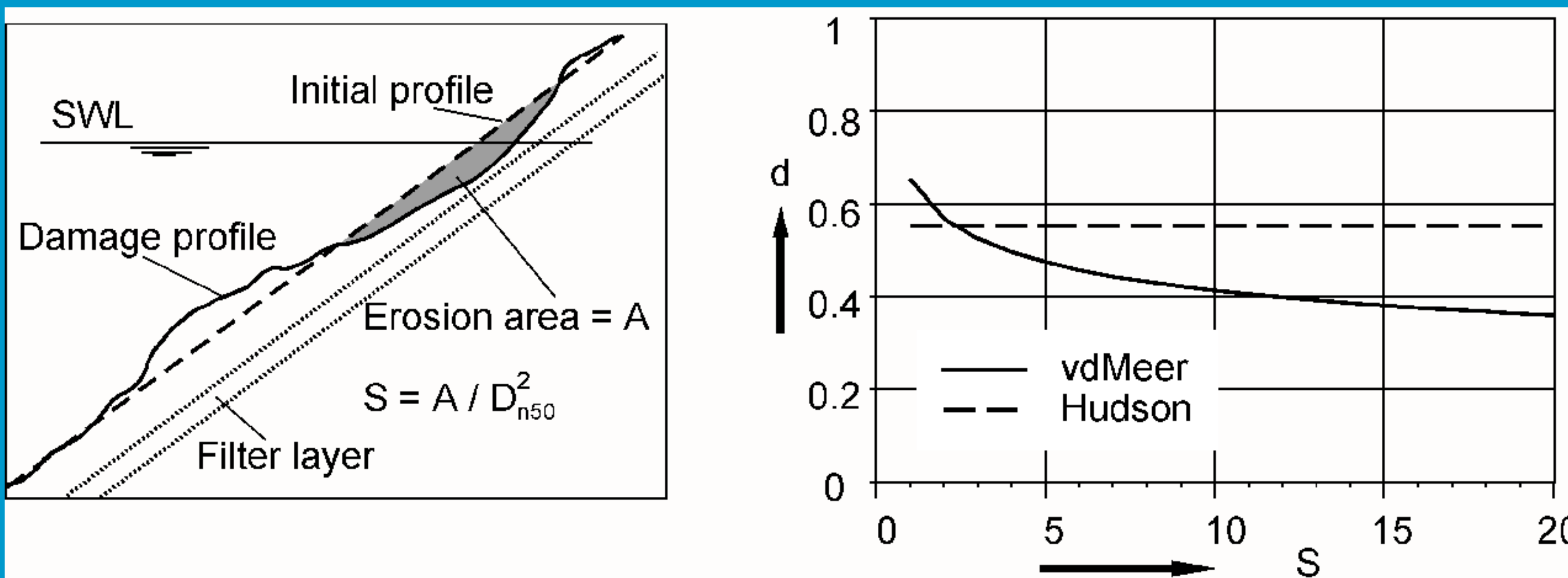
notional:
belonging to the realm of ideas, not of experience; existing only in the mind

number of waves

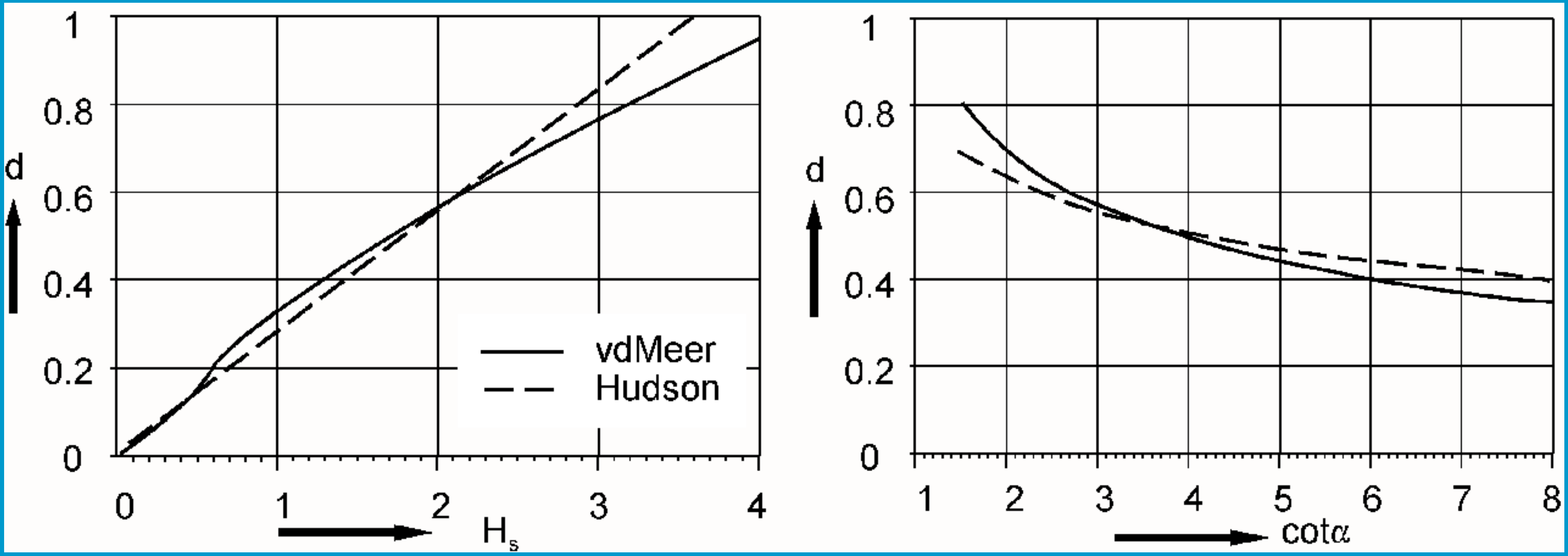


maximum number of waves: 7500
3000 waves of 6 s is 5 hours

damage level



slope angle


1
0.8
d
0.6
0.4
0.2
0
0
1
2
3
4
H
s
vdMeer
Hudson
1
0.8
d
0.6
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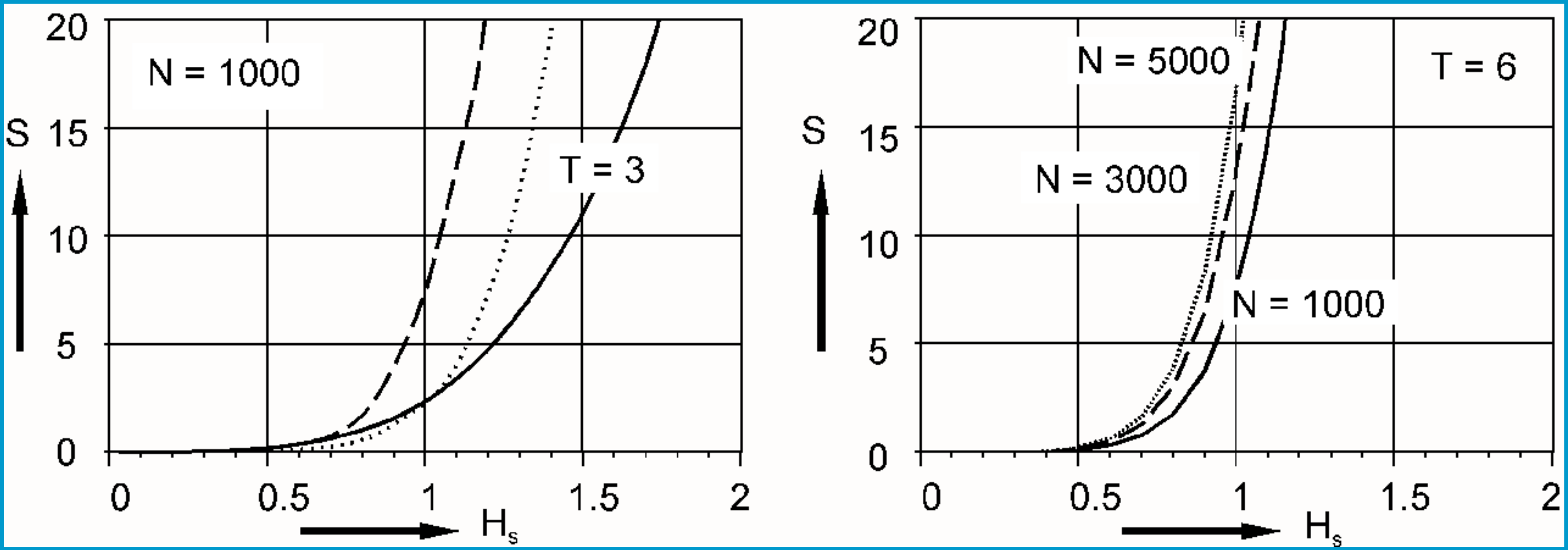
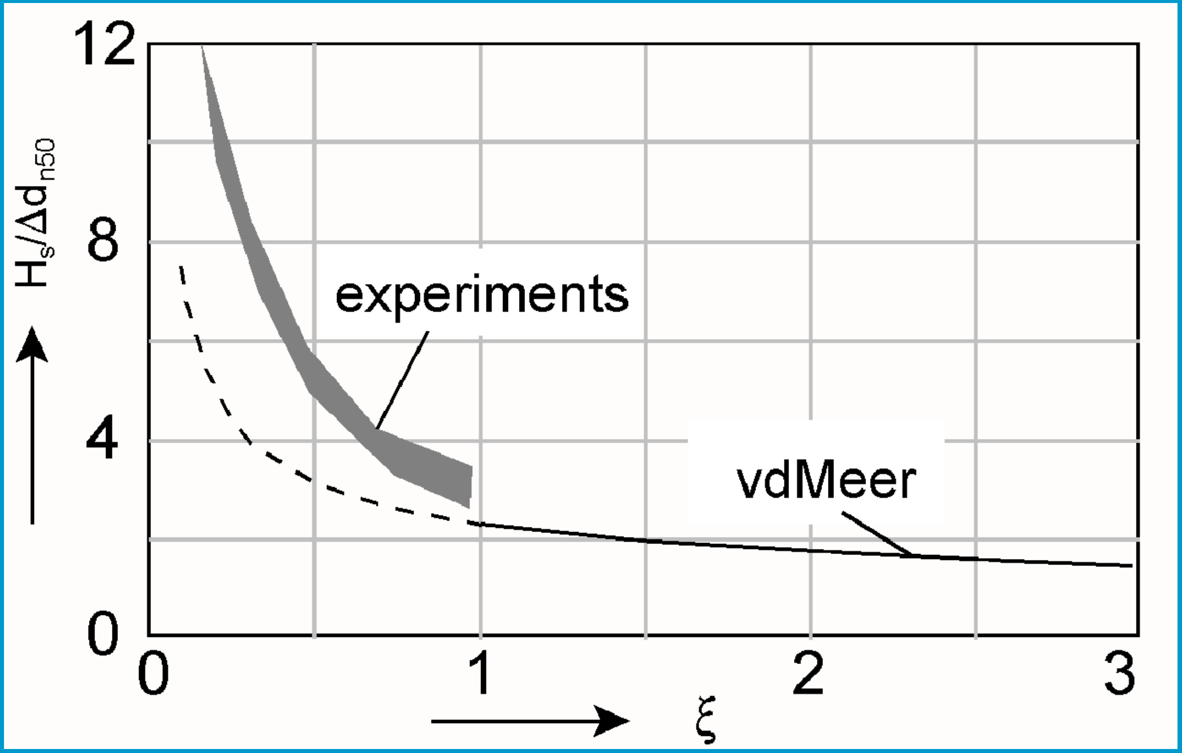
0
1
2
3
4
5
6
7
8
cotα

damage development



mild slopes



low crested dams (1)

crest above water level

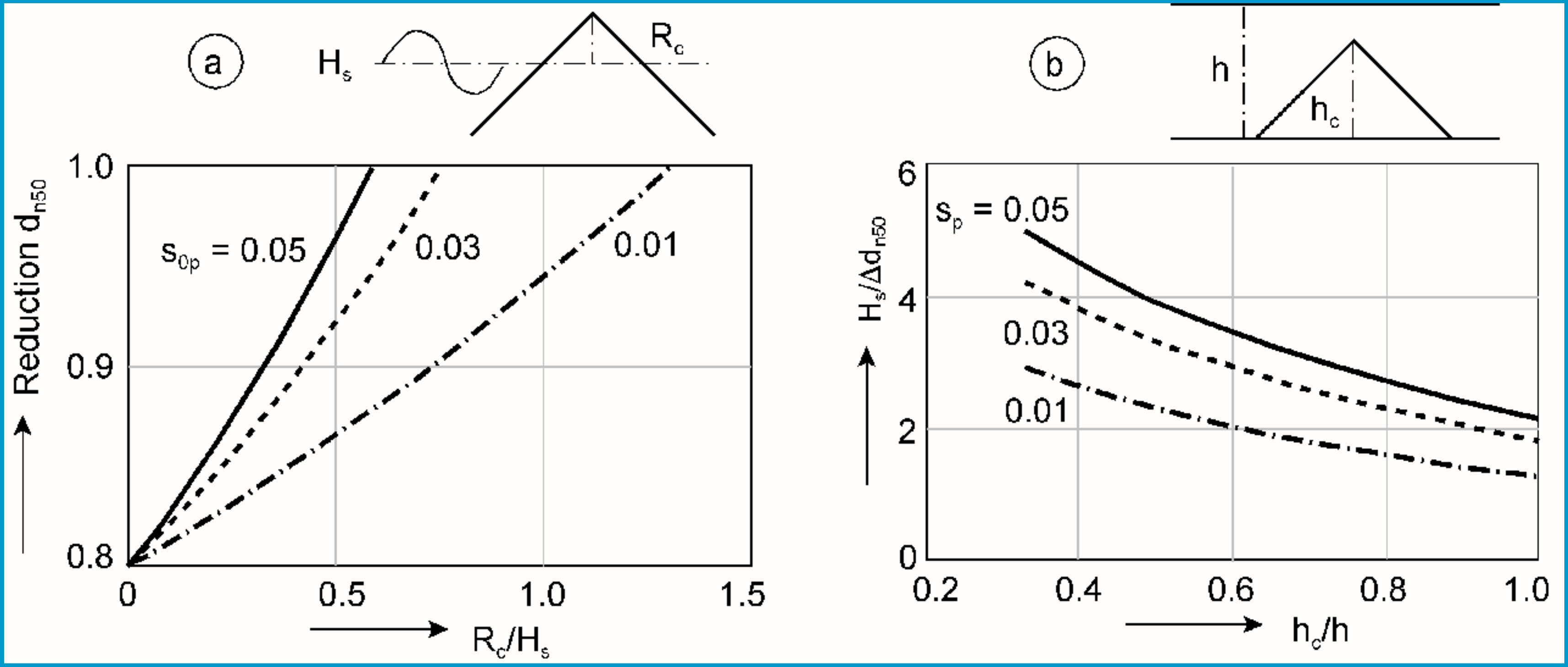
$$\text{Reduction } d_{n50} = \frac{1}{1.25 - 4.8\dfrac{R_c}{H_s}\sqrt{\dfrac{s_{0p}}{2\pi}}}$$

crest below water level

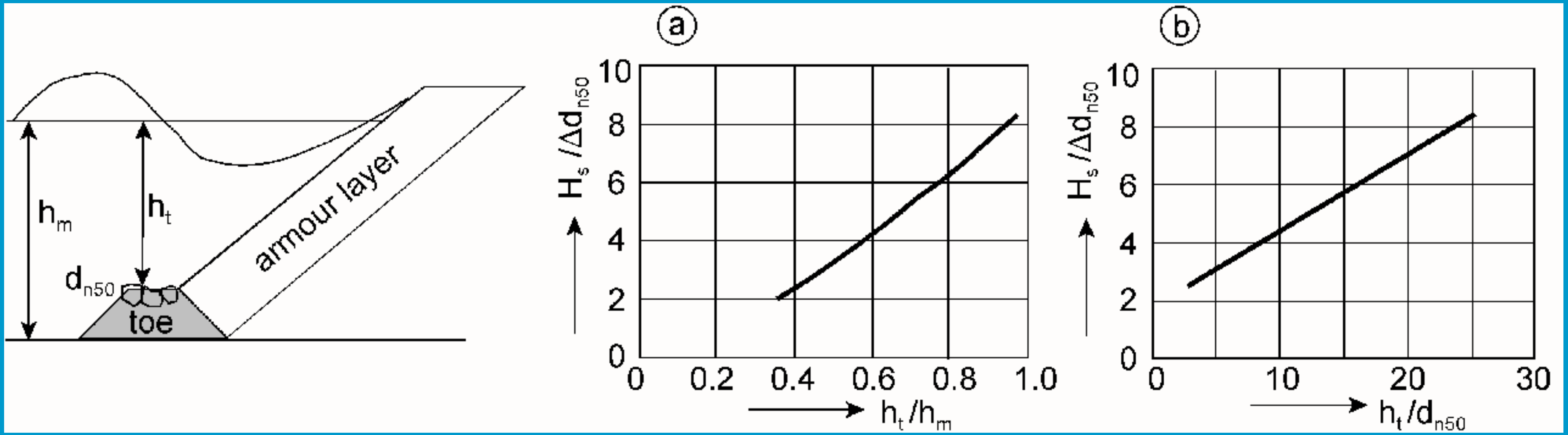
$$\frac{H_s}{\Delta d_{n50}} = -7\ln\left(\frac{1}{2.1+0.1S}\frac{h_c}{h}\right)\sqrt[3]{s_p}$$

R_c	crest height with respect to SWL
s_{0p}	(deep water) wave steepness (from T_p)
h	waterdepth
h_c	height of dam

low crested dams (2)



stability of toes



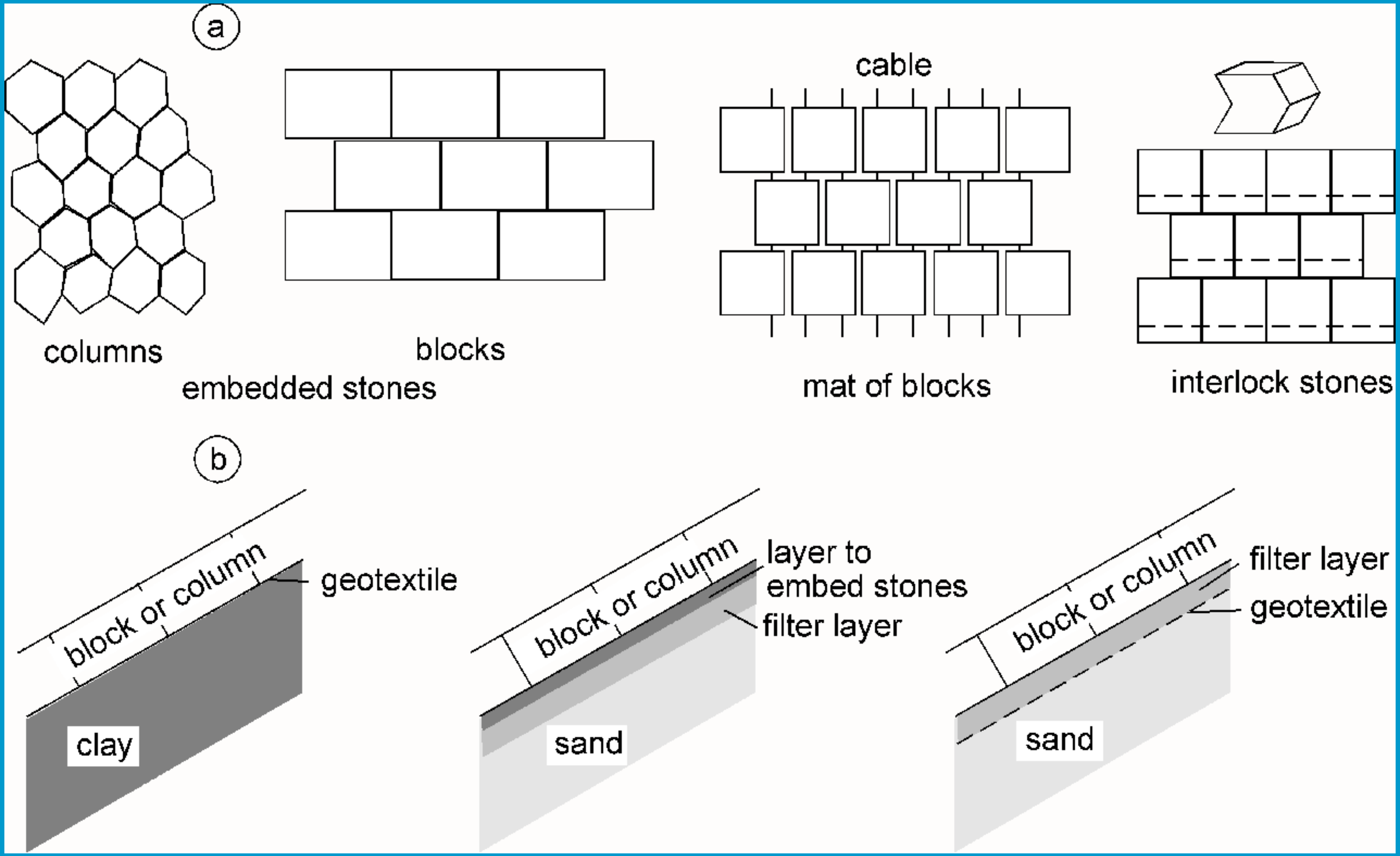
a: deep toes with small damage

$$\frac{H_s}{\Delta d_{n50}} = 8.7 \left(\frac{h_t}{h_m} \right)^{1.4}$$

b: shallow toes

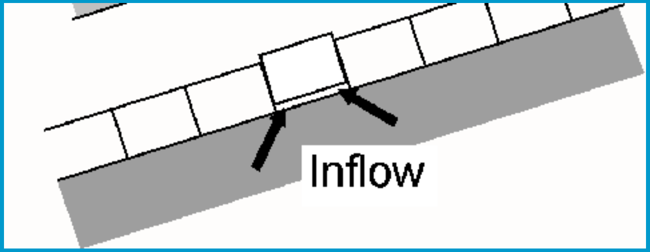
$$\frac{H_s}{\Delta d_{n50}} = 1.1 \left(0.24 \frac{h_t}{d_{n50}} + 1.6 \right)$$

block types and filters in revetments

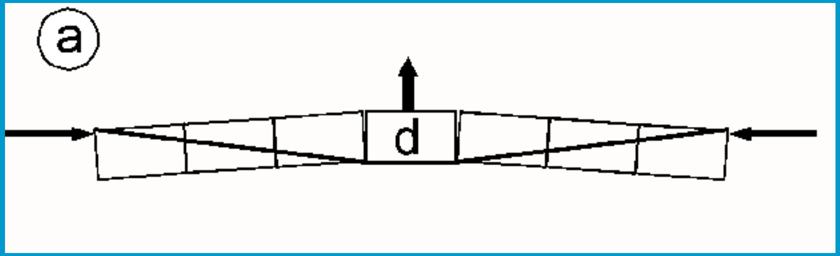


Two failure mechanisms for blocks

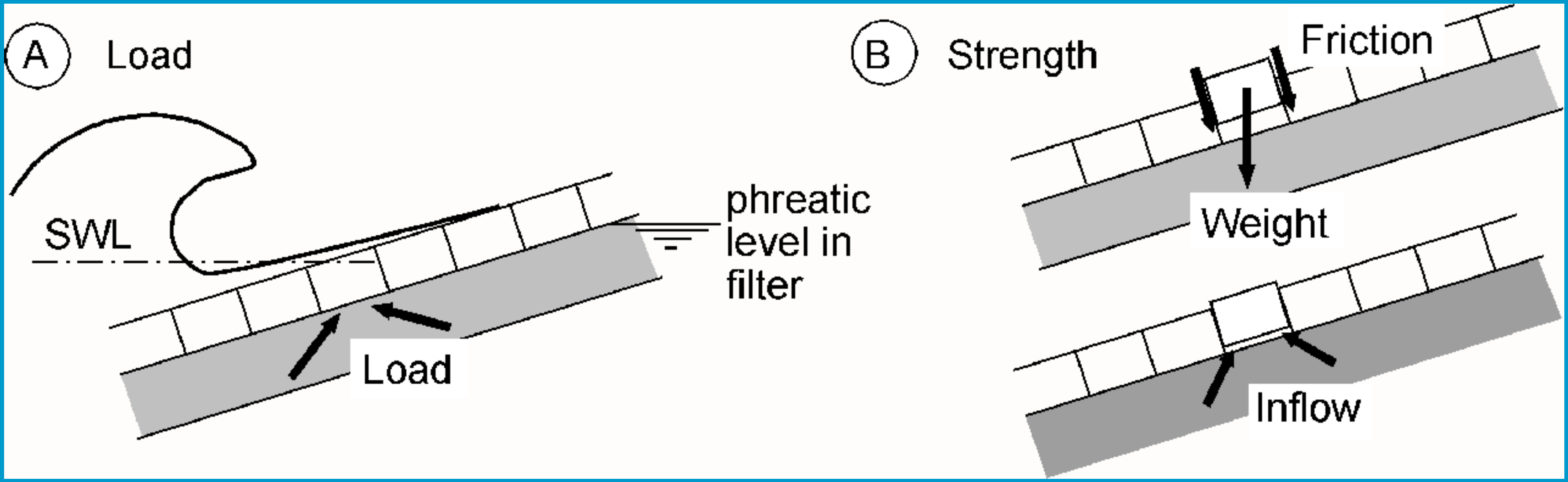
- The piston type failure



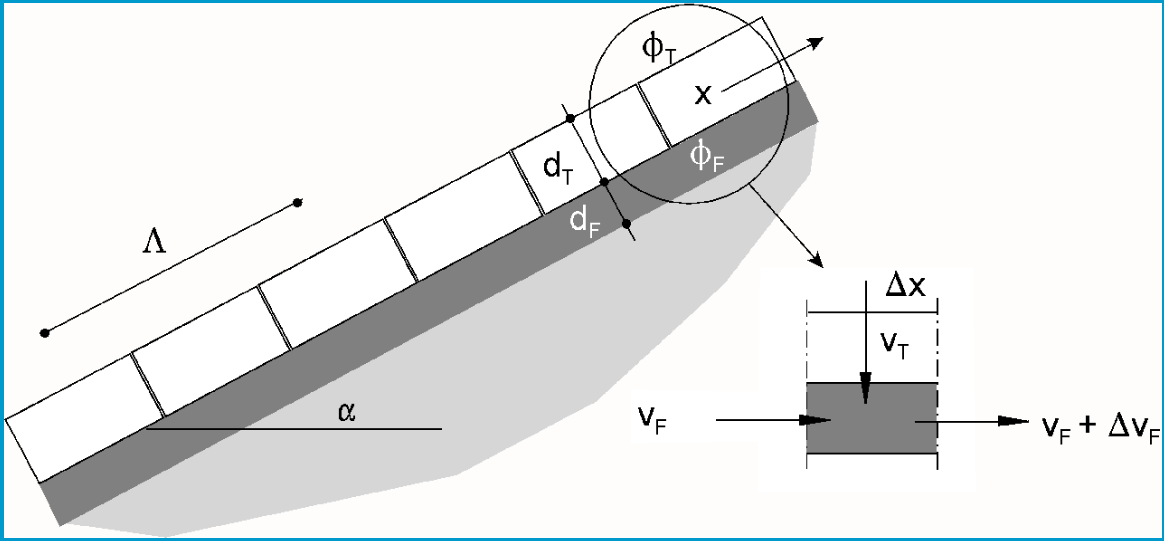
- The beam type failure



load and strength of block revetments



flow through block revetment and leakage length



flow in filter:

$$v_F = -k_F \frac{\mathrm{d}\,\phi_F}{\mathrm{d}\,x}$$

flow through top layer:

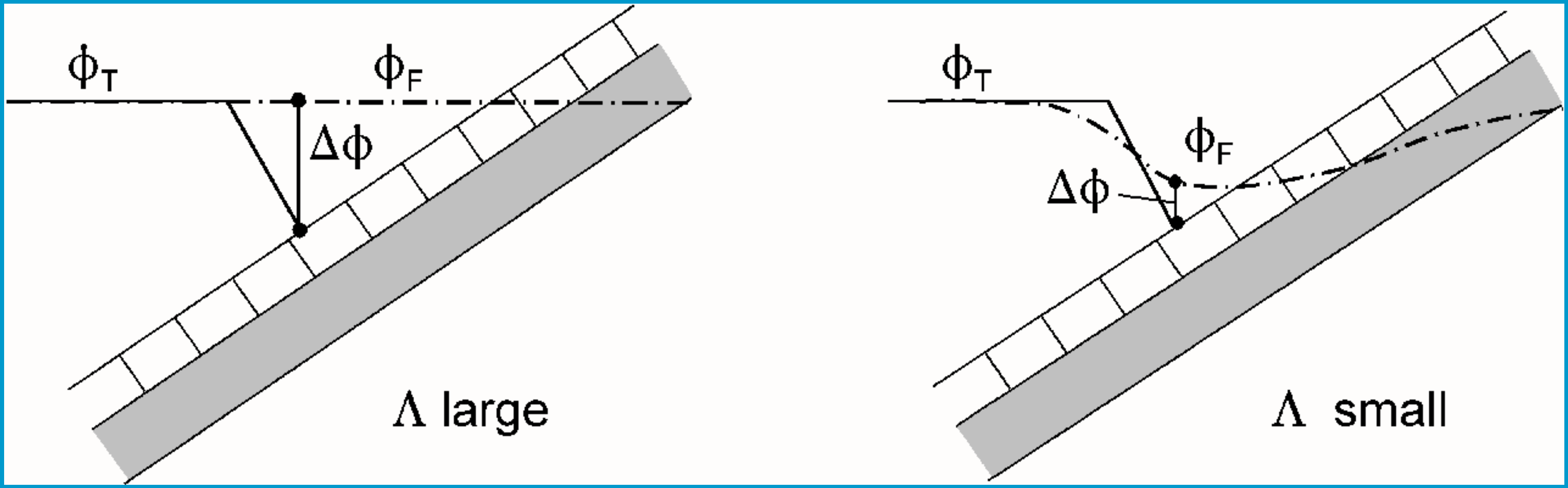
$$v_T = k_T \frac{(\phi_F - \phi_T)}{d_T}$$

Using continuity this leads to:

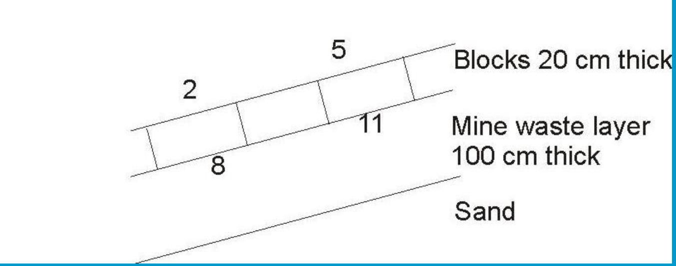
$$\frac{\mathrm{d}^2\phi_F}{\mathrm{d}\,x^2} = \frac{-k_T(\phi_F - \phi_T)}{k_F\, d_T\, d_F} = -\frac{(\phi_F - \phi_T)}{\Lambda^2} \quad \rightarrow \quad \phi_F - \phi_T = -\Lambda^2 \frac{\mathrm{d}^2\phi_F}{\mathrm{d}\,x^2}$$

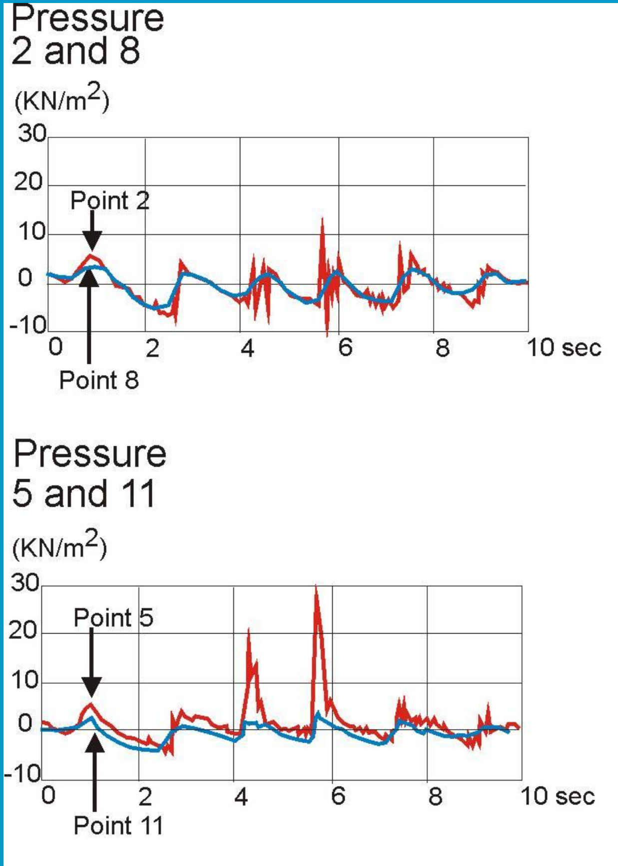
Φ_T and Φ_F are piezometric heads ($\Phi = p/\rho g + z$) on top layer and in filter layer

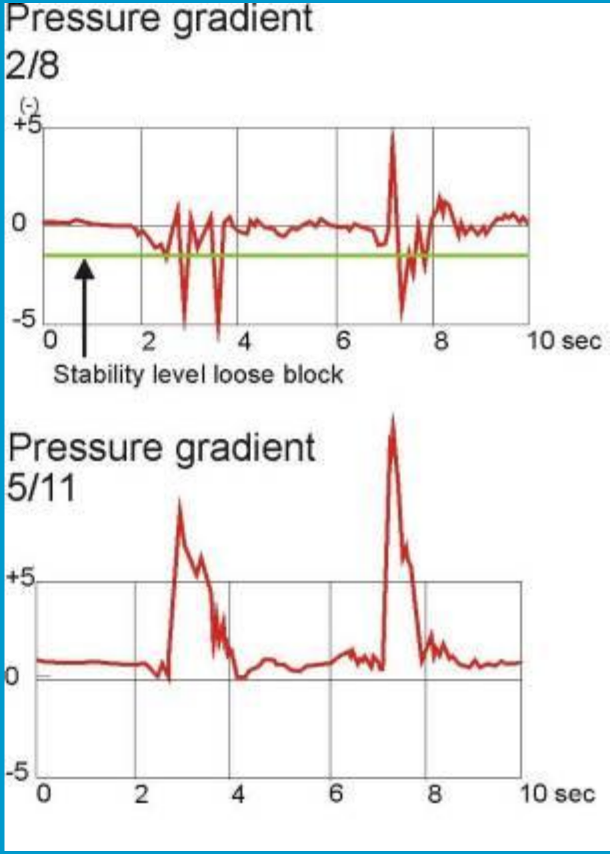
head difference over block for large and small leakage length



Measured head differences







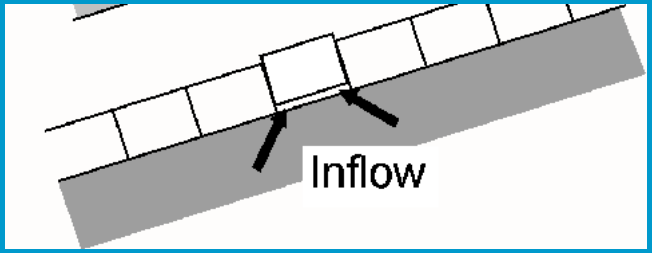
[Revetments and Numerical Simulation (8 min) (SteenZet.mpg](SteenZet.mpg)

Conclusion regarding leakage length

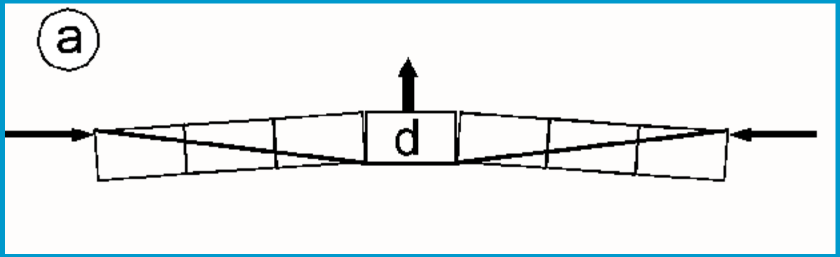
- small leakage length is best
- this means that top layer has to be more permeable than filter layer
- extreme case: make filter layer nearly impermeable
- practical example: blocks on clay
- However ....??
- execution problem
- creation of gullies

Two failure mechanisms for blocks

- The piston type failure



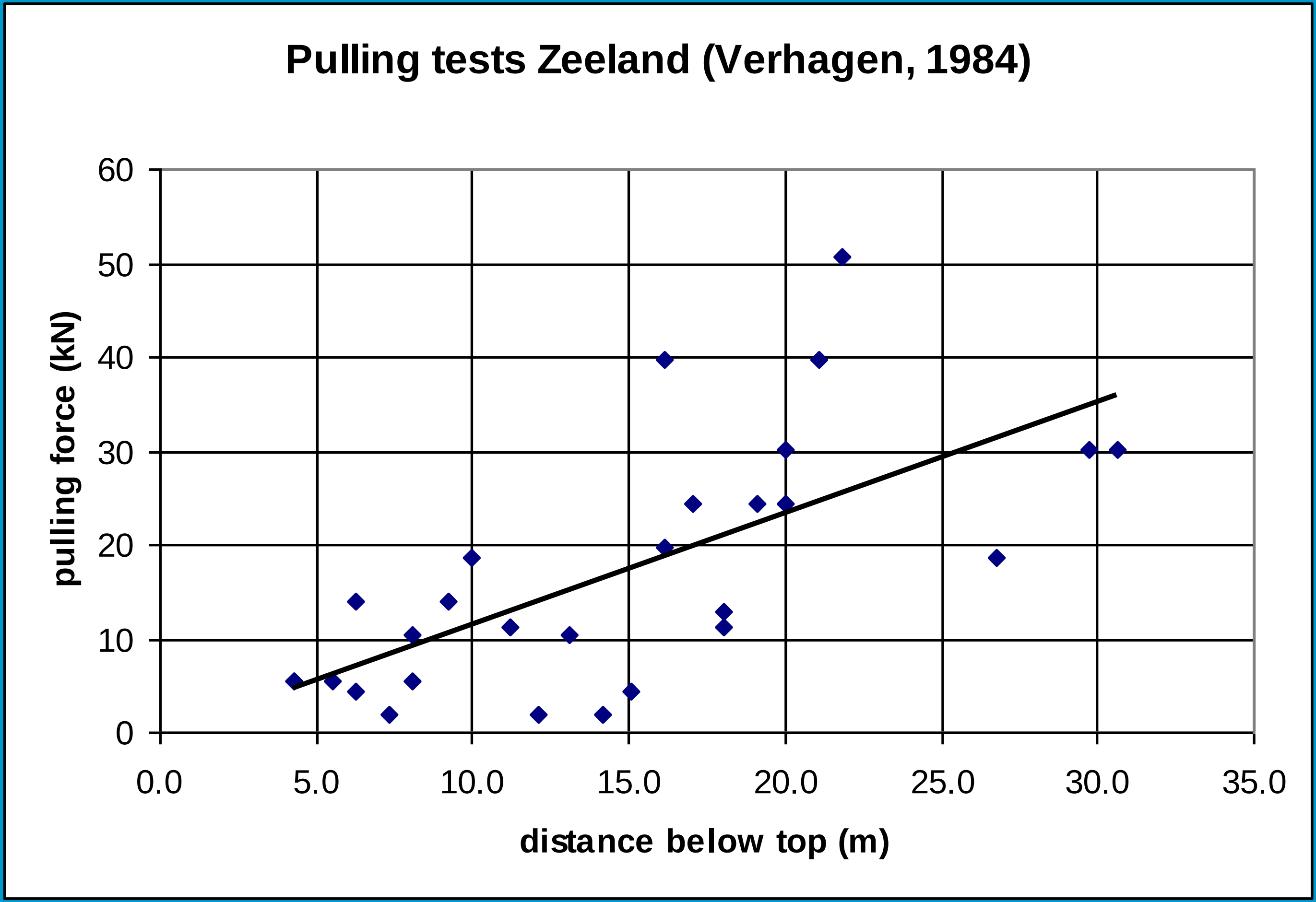
- The beam type failure



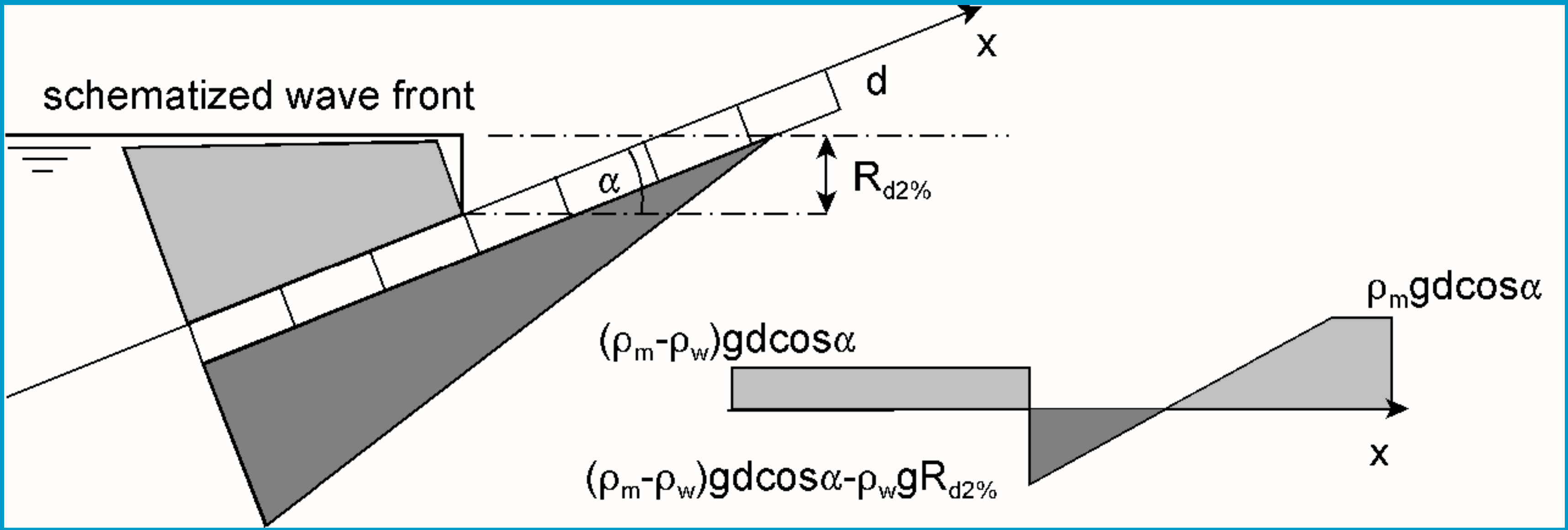
Pulling tests

Type of block	Average weight (kg)	Pulling force (kgf)	Stand.dev. of Pulling force (kgf)
Basalt 1	17	1763	1282
Basalt 2	32	2178	1248
Basalt 3	35	1528	1037
Basalt 4	50	8874	3324
Haringman	180	3764	2194
Vilvoordse	16	668	369

Pulling force vs. position



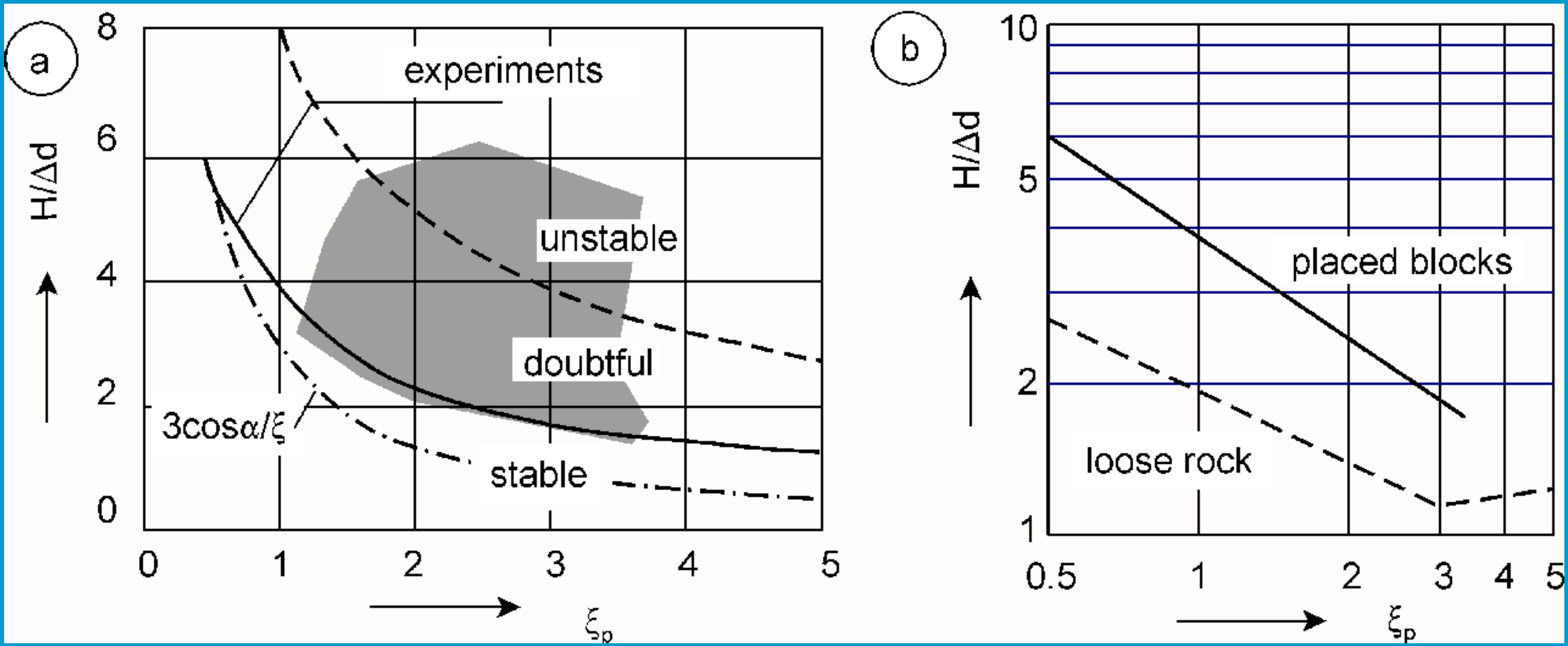
stability of block revetment



$$(\rho_m-\rho_w)gd\cos\alpha-\rho_w g0.33\xi H_s=0 \quad\rightarrow\quad \frac{H_s}{\Delta d}=3\frac{\cos\alpha}{\xi}$$

first guess

test results for placed blocks



the Pilarczyk formula

$$\frac{H_s}{\Delta_m D} \leq \Psi_u \, \Phi \frac{\cos\alpha}{\xi_p^b}$$

Ψ_u	system defined (stability) upgrading factor {for riprap by definition $\Psi_u = 1$}
Φ	stability factor
H_s	significant wave height
T_p	peak period of the waves
ξ_p	Iribarren-number for peak period
D	specific size of protection unit
α	slope angle
Δ_m	relative density of the system unit
b	exponent $0.5 < b < 1$ for riprap b=0.5, for smooth blocks b=1 on average $b \approx 2/3$

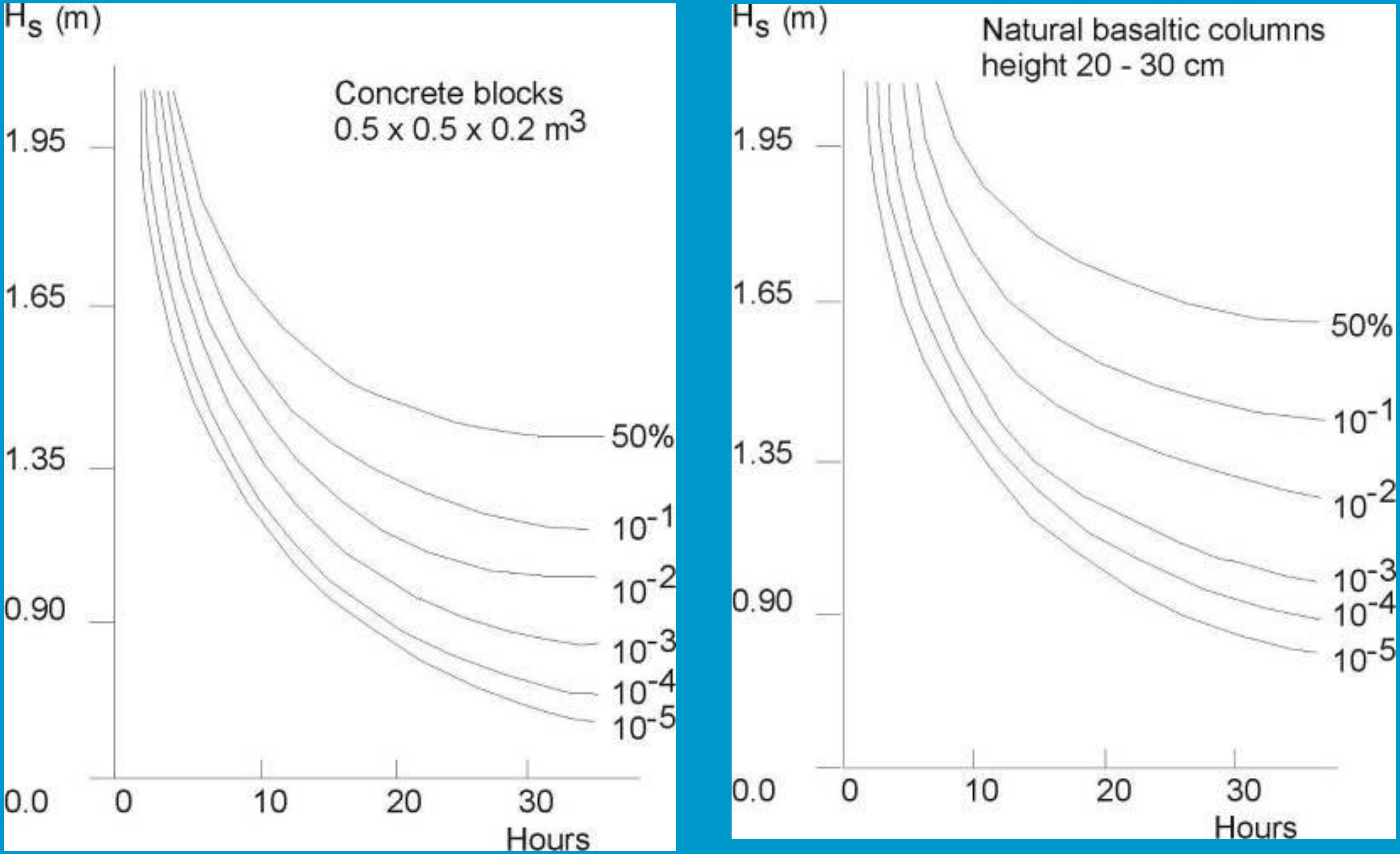
the Pilarczyk formula (2)

$$\Phi = 6.2 P^{0.18} \left(\frac{S^2}{N} \right)^{0.1} \qquad \textit{for} \quad \xi < 3, \textit{ breaking waves}$$

Φ = 2.0	for incipient motion of stones
Φ = 2.25	average value for incipient motion
Φ = 3.0	as a first approximation for max. tolerable damage

Ψ_u	1.0	riprap (by definition)
	1.0	poor quality pitched stone
	1.5	high quality pitched stone
	1.5	loose closed blocks
	2.0	high quality blocks (Basalton, Hydroblock)
	1.5	Pattern grouting
	2.0	Fixstone
	2.5	gabions
	2.5	Armorflex (cable system)

time effect


Hs (m)
Concrete blocks
0.5 x 0.5 x 0.2 m3
1.95
1.65
1.35
0.90
0.0
0
10
20
30
Hours
50%
10-1
10-2
10-3
10-4
10-5
Hs (m)
Natural basaltic columns
height 20 - 30 cm
1.95
1.65
1.35
0.90
0.0
0
10
20
30
Hours
50%
10-1
10-2
10-3
10-4
10-5

Asphalt

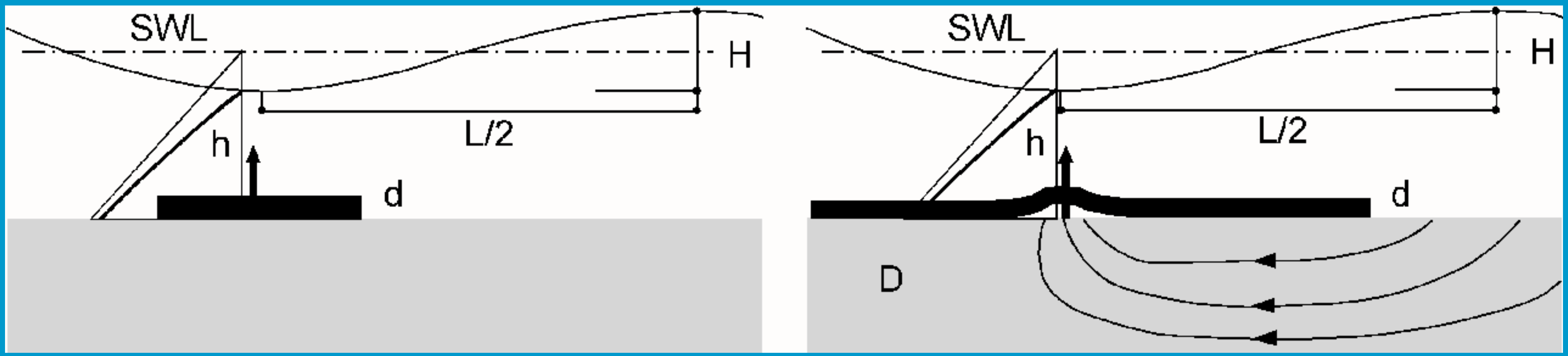
Technisch Rapport Asfalt voor Waterkeren
TAW 2002 (www.enwinfo.nl)
(not yet translated in English)

Types of asphalt revetments

		closed revetments			open revetments			
dike type	zone	asphalt concrete	asphalt mastic	penetrated riprap	partly penetrated riprap	open stone asphalt	open stone asphalt mat	sand asphalt
riverdike	I	-	+	+	o	-	+	o
	II	-	-	+	o	+	+	o
	III	+	-	o	o	+	o	o
	IV	o	-	o	o	o	o	o
lake dike	I	-	+	+	+	-	o	o
	III	+	-	o	+	+		o
	IV	o	-	o	+	+		o
seadike	I	-	+	+	+	-	+	o
	II	-	-	+	+	+	+	o
	III	+	-	+	+	+	+	o
	IV	+	-	o	o	+	o	o

Dike zones:
- I always below water
- II between low water and high water
- III between high water and design level
- IV run-up zone

impervious layers in waves

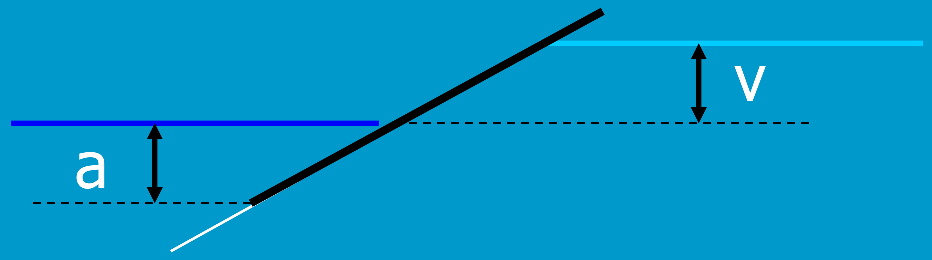


$$(\rho_m - \rho_w)\, g\, d > \rho_w\, g\, \frac{H}{2} \frac{1}{\cosh\left(\frac{2\pi}{L} h\right)}$$

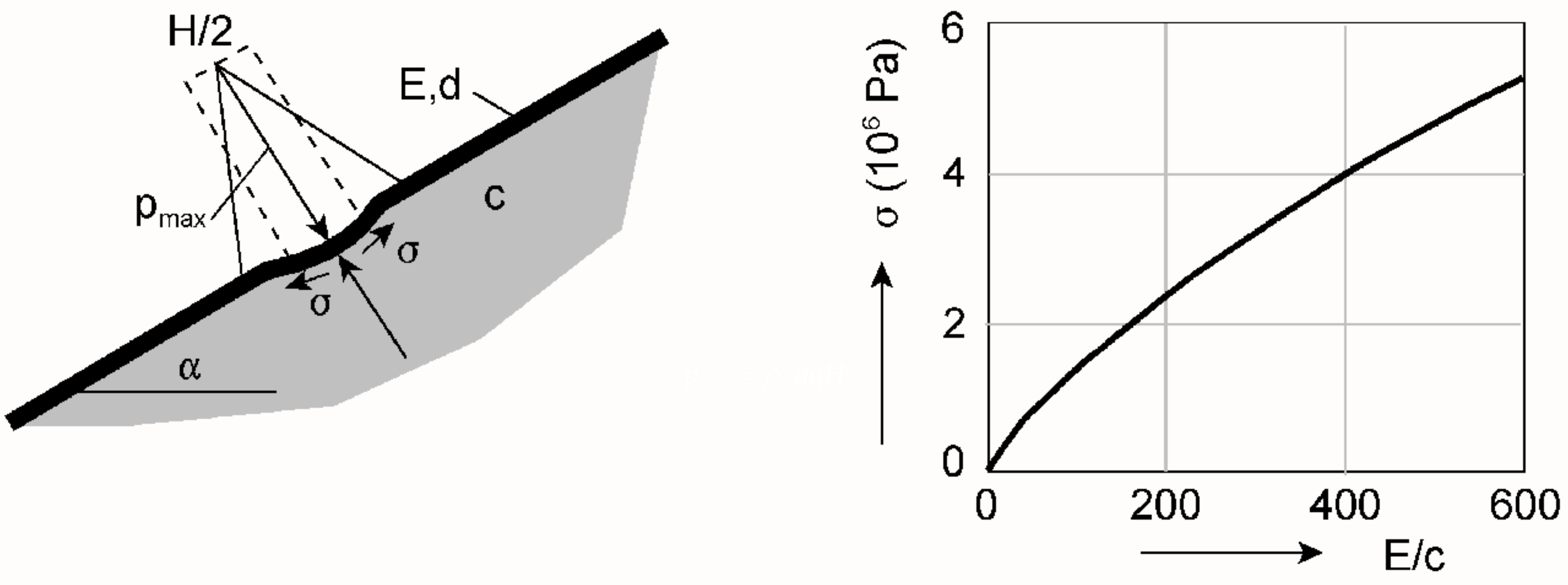
static pressure on impervious layers

$$d = 0.21 Q_n (a + v) \left(\frac{\rho_w}{\rho_a - \rho_w} \right) R_w$$

Q_n – reduction for slope
R_w – reduction for relative position of outer water level
a – depth of revetment under water
v – groundwater level above outer water level



wave impact on slope and influence material properties

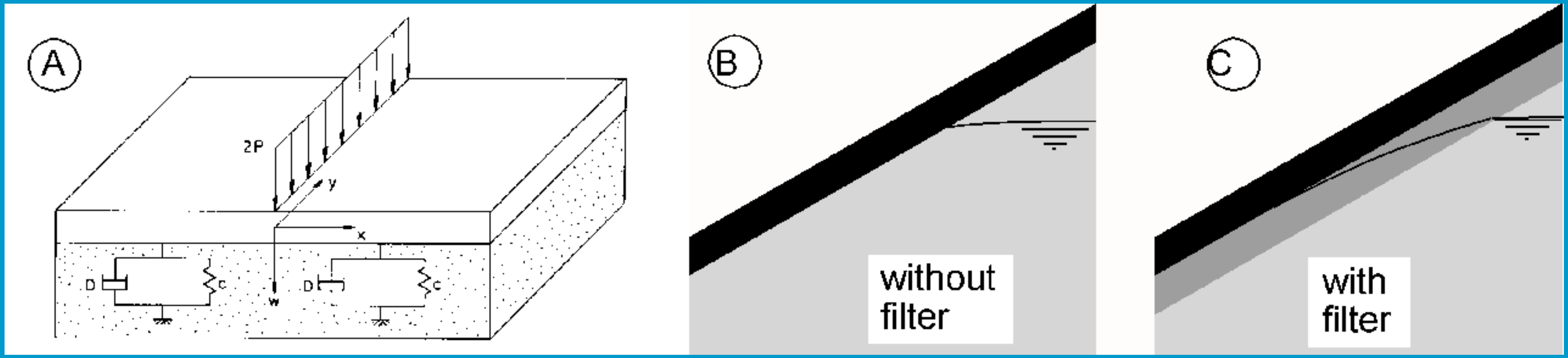


stiffness ratio

$$p_{\max} = \rho_w g q H$$

q - impulse factor (=3.4)
c - soil stiffness
E - soil elasticity

Loads on asphalt and influence filter



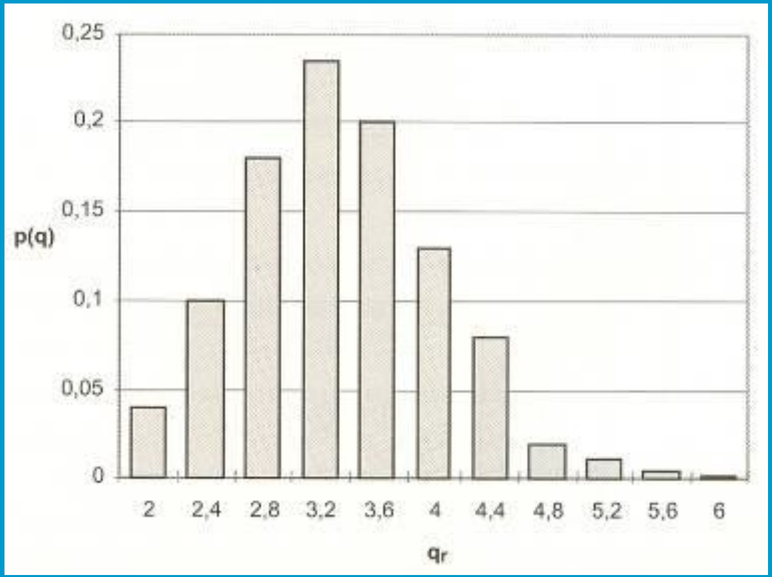
$$\sigma = \frac{3p_{\max}}{\beta^3 H d^2}\left[1 - \exp\left(\frac{-\beta H}{2}\right)\left(\cos\frac{\beta H}{2} + \sin\frac{\beta H}{2}\right)\right]$$

$$\text{in which}: \beta = \sqrt[4]{\frac{2.65}{d^3 \, E/c}}$$

But every wave is different….

Führböter [1988] has derived a probability function for the impact factor *q* for a slope 1:4

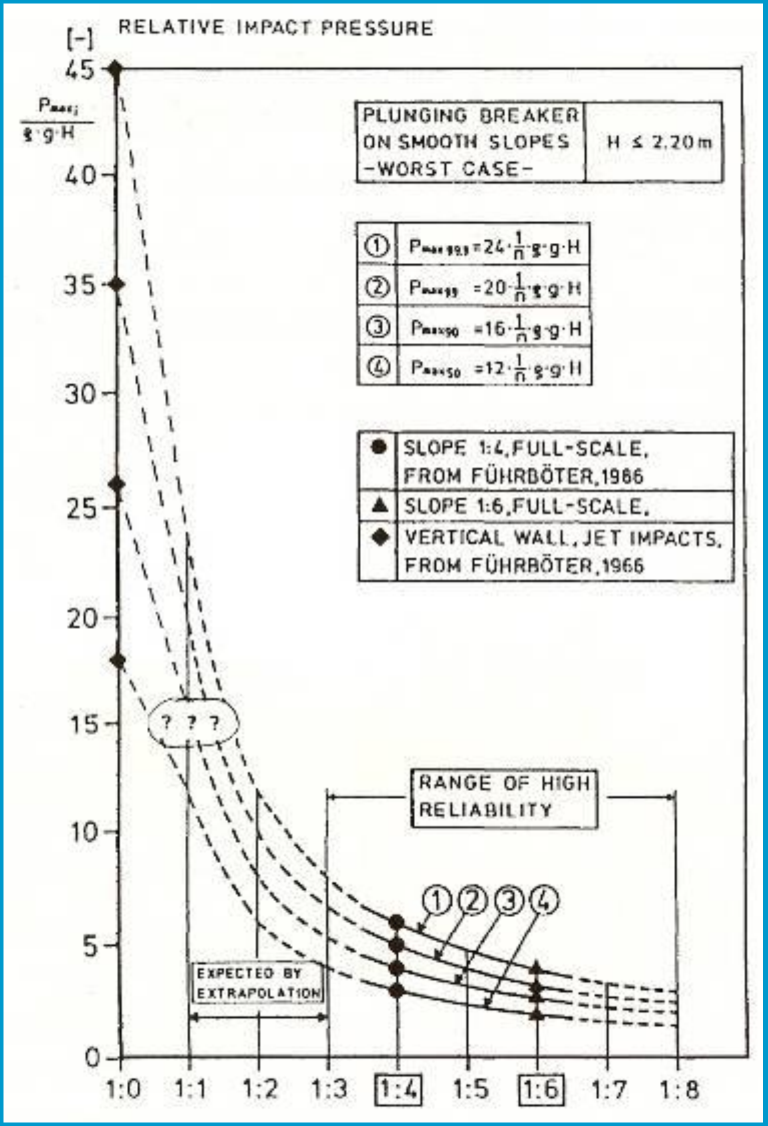
$$\Pr(q) = \frac{1}{\sigma_q \sqrt{2\pi}} \exp\left(-\frac{(q-\bar{q})^2}{2\sigma q^2} \right)$$



Relation impact factor and slope

For other slopes a linear interpolation with slope is made:

$$q = \frac{\tan\alpha}{1/4} q_r$$



Fatigue

The number of loads that leads to failure is:

$$N_f = k_f \sigma^{-a_f}$$

Note: This formula is NOT dimensionless

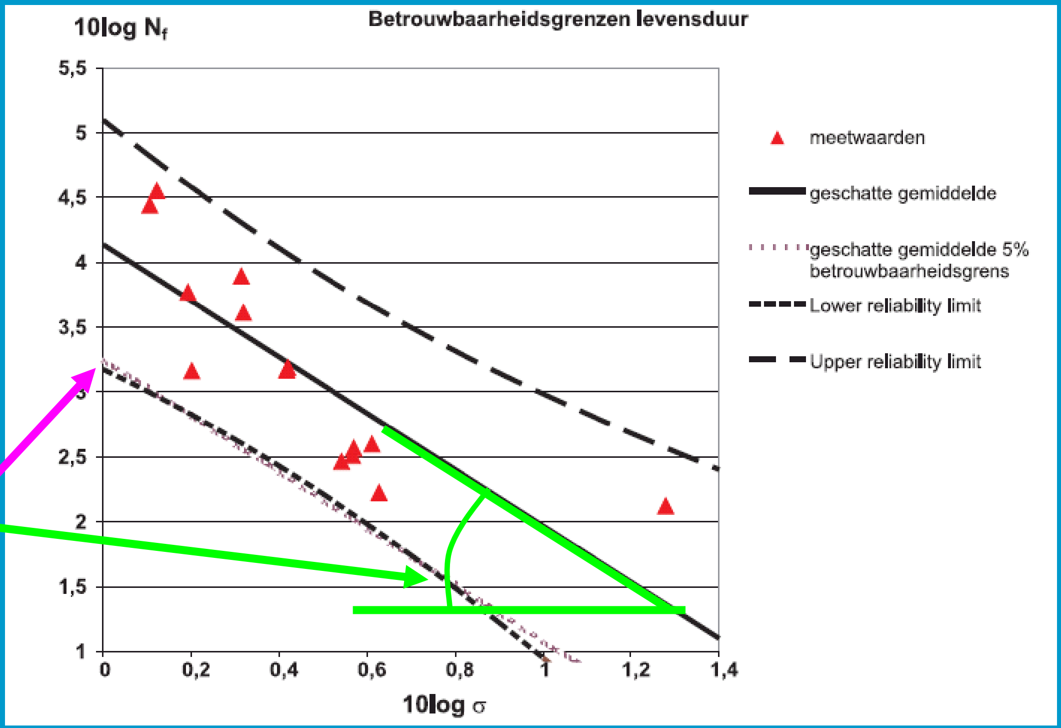
k_f and a_f are fatigue parameters of the asphalt, to be determined by the producer or from test samples of the placed asphalt

σ is the tension stress at the underside of the asphalt (in MPa)

So, with this formula I can determine for each given tension stress, the allowed number of load repetitions

Determination of the fatigue parameters

- Plot measured values
- calculate average and reliability limits
- linearize lower reliability limit
- a= slope of best fit
- log(k)= intercept



The Miner sum

Rule of Miner:

The cover layer will not fail as long as the following condition is true:

$$\sum \frac{n_i}{N_{f,i}} \leq 1$$

in which:

n_i - number of load repetitions i

$N_{f,I}$ - number of load repetitions i, leading to failure

Example

- I have three loads, one of 1 MPa, 1.5 MPa and 2 Mpa

$N_{1MPa} = 10^{3.02}\ 1^{-3.77} = 1000$

$N_{1.5MPa} = 10^{3.02}\ 1.5^{-3.77} = 217$

$N_{2MPa} = 10^{3.02}\ 2^{-3.77} = 73$

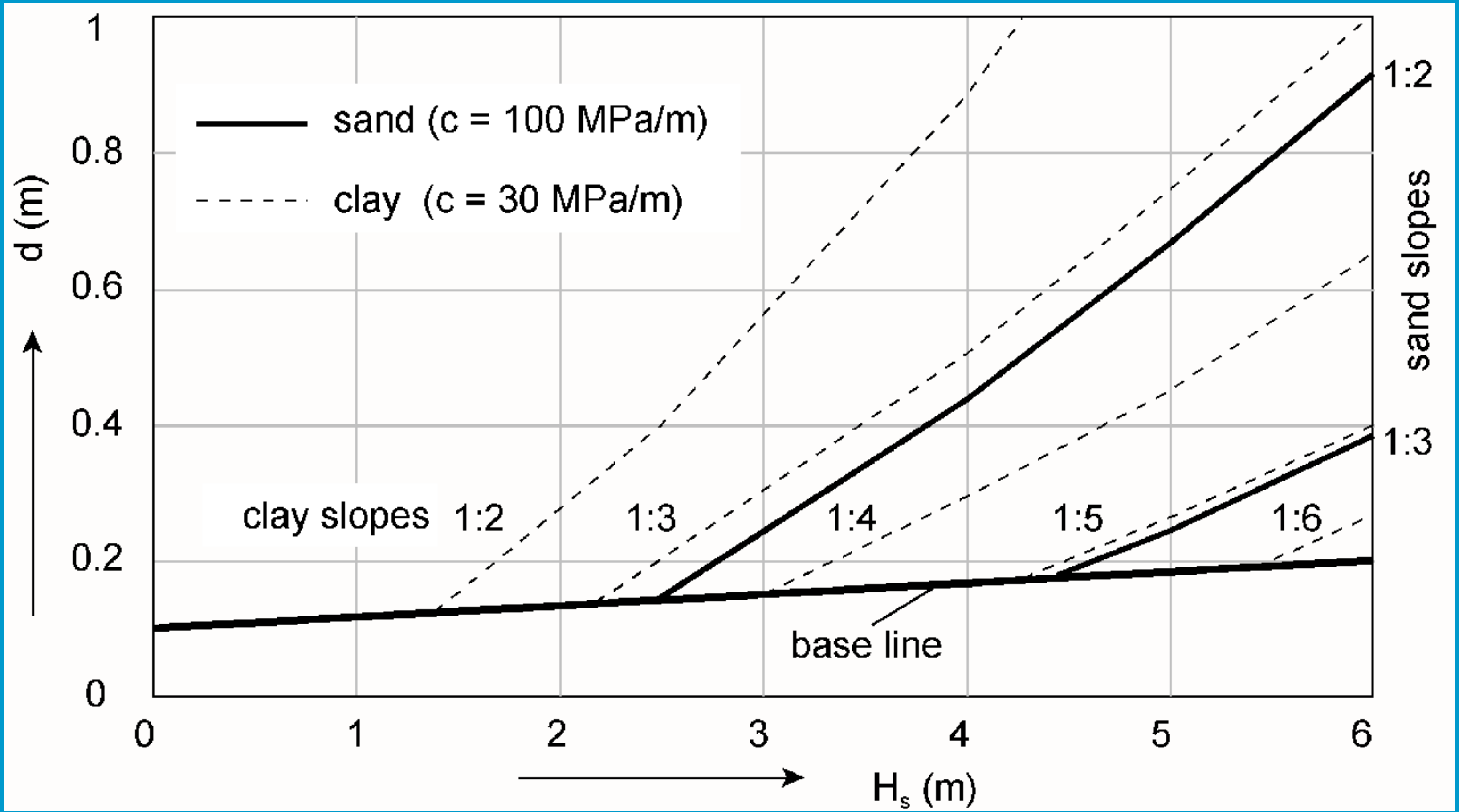
- Load 1 occurs 500 times, load 2 occurs 50 times and load 3 occurs 20 times.
- Minersum: $\frac{500}{1000} + \frac{50}{217} + \frac{20}{73} = 0.5 + 0.23 + 0.27 = 1$
- Conclusion: This type of load is exactly the limit

Practical calculation

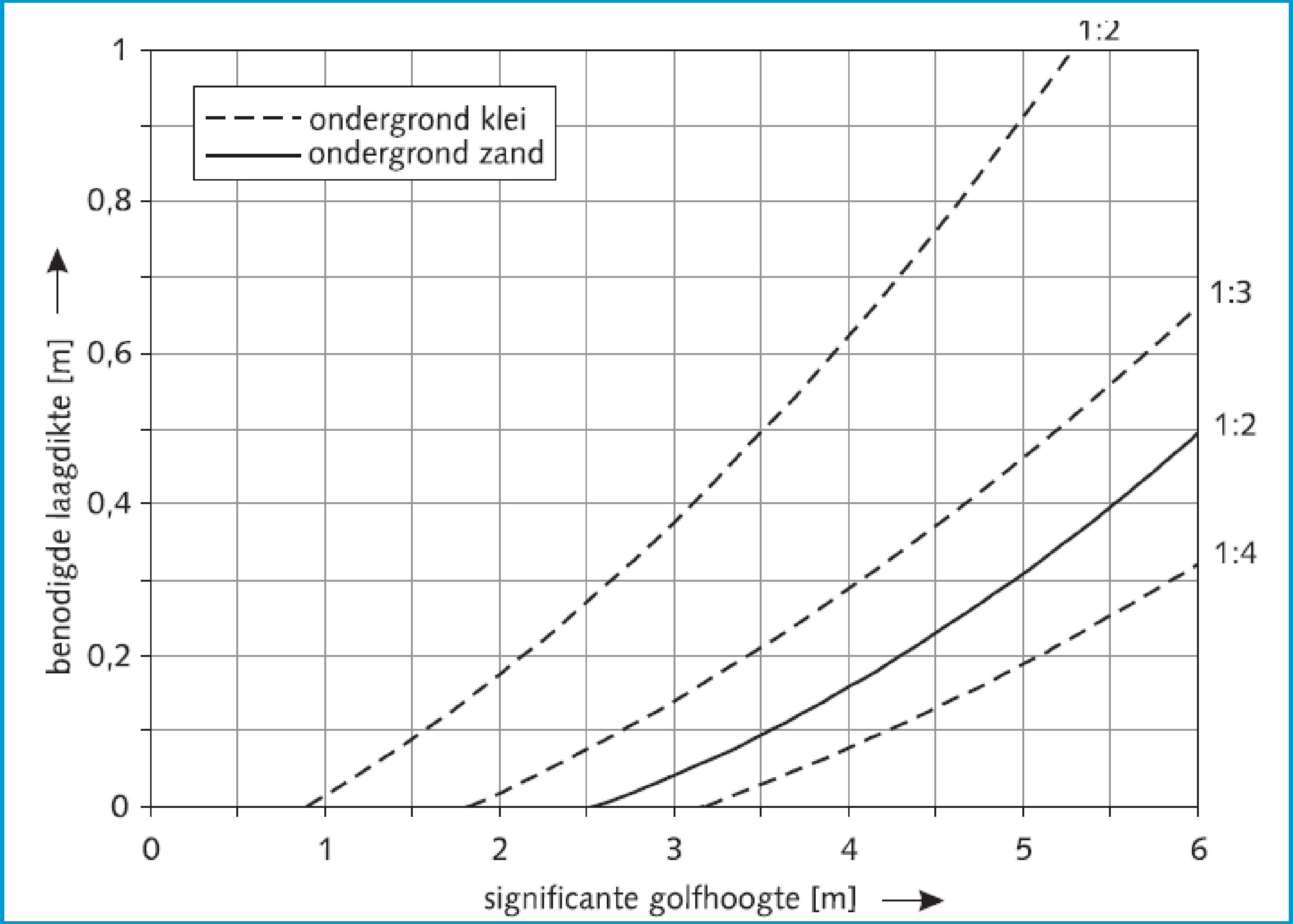
- Determine the different load levels during a storm (i.e. select a number of H/T classes)
- Calculate for each level of the revetment the number of loads for each load level
- Calculate for each combination the partial Miner-sum
- Add up all Miner sums and verify that this sum is <1.

- This can be done with the computer program Golfklap (only in Dutch, but downloadable from Blackboard)

necessary thickness of asphalt concrete on sand or clay



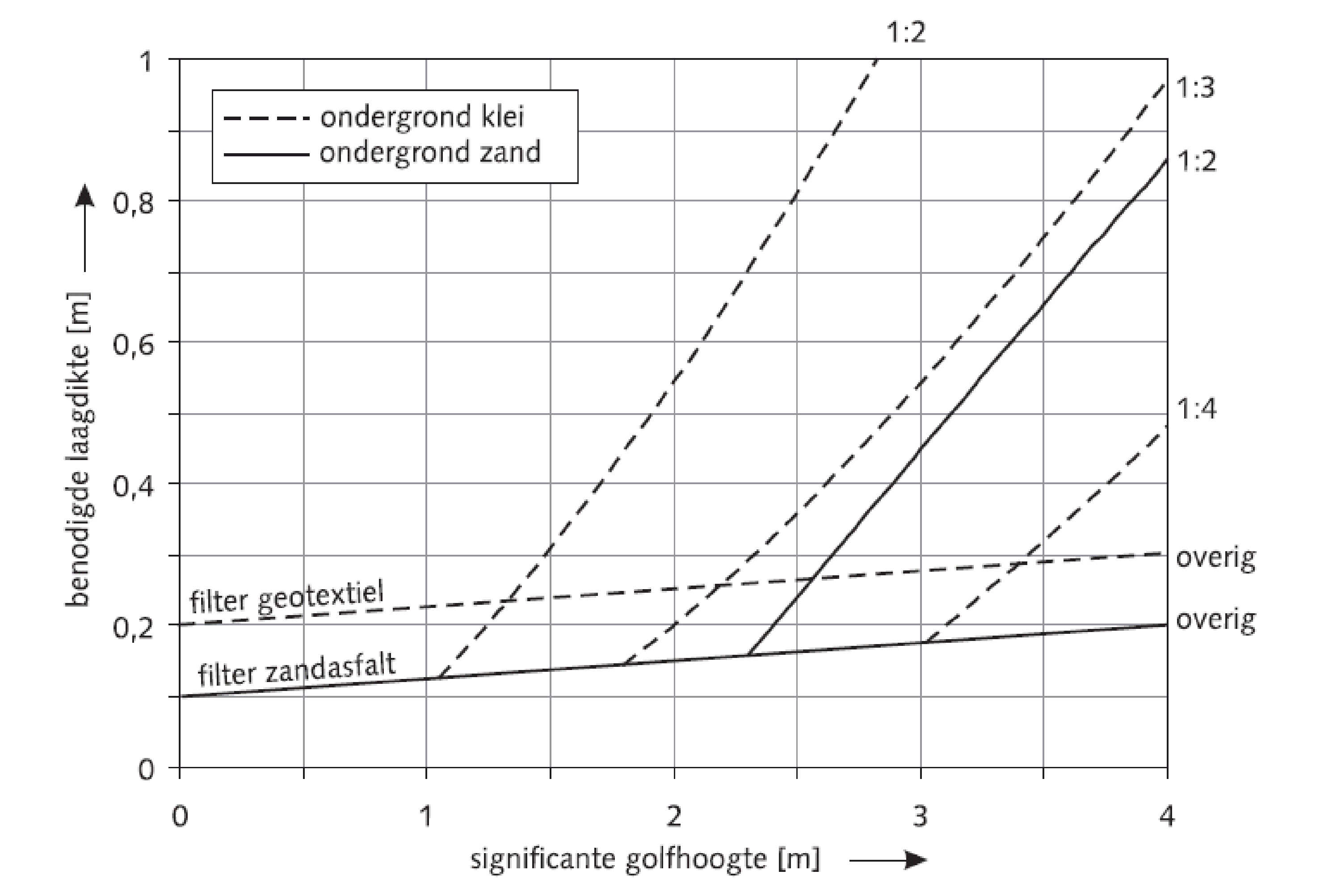
Asphalt penetration



Very open plate structures

- **Open stone asphalt**
  - Is able to follow subsoil settlements
  - Sensitive to fatigue
  - Sensitive to damage by abrasion
- **Colloidal concrete**
  - Very stiff and not very elastic
  - Very strong and not sensitive to abrasion in case of very good execution
- **Stones glued with polymers**
  - Not sensitive to fatigue
  - Very resistant against abrasion
  - Very elastic, but is maybe not able to cope with large deformations

Open stone asphalt



Glued revetments

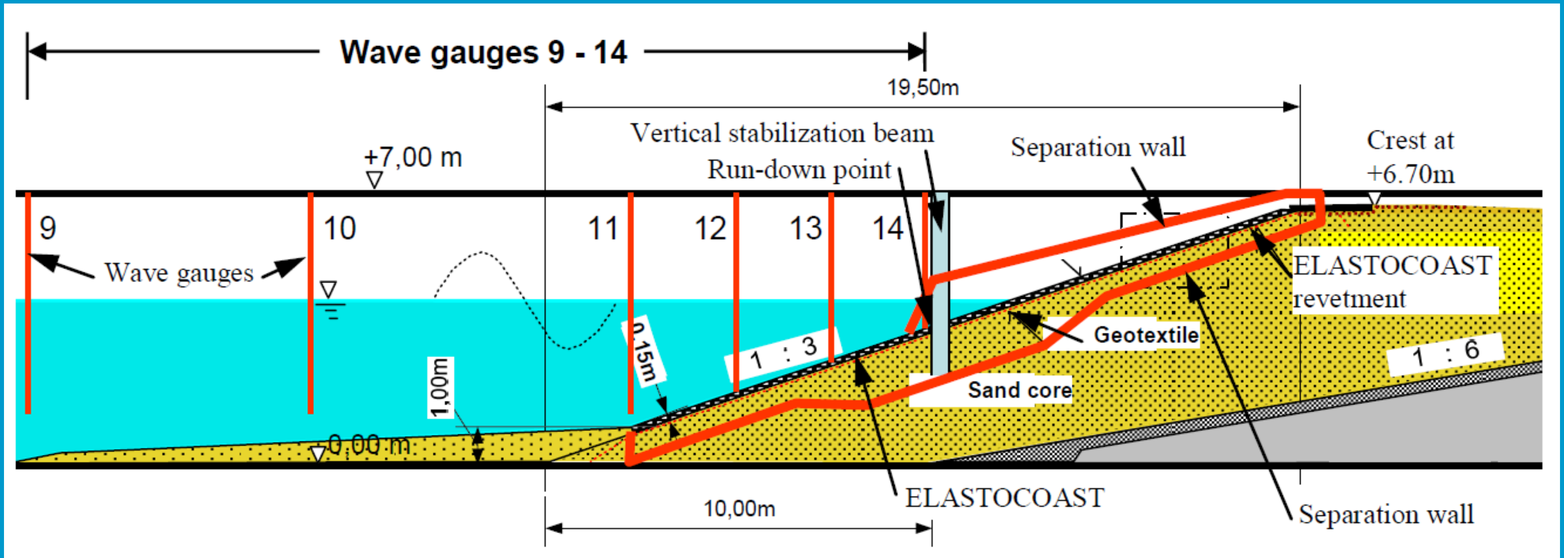
- It is possible to glue small stones
  - Elastocoast from BASF (using Polyurethane glue)
  - InfraElast from Rotim (using Epoxy)

What is Elastocoast ?

- Revetment structure of small stones glued together with polyurethane glue with a very strong bonding
- Details will be discussed by Bijlsma on Thursday afternoon (session C7)

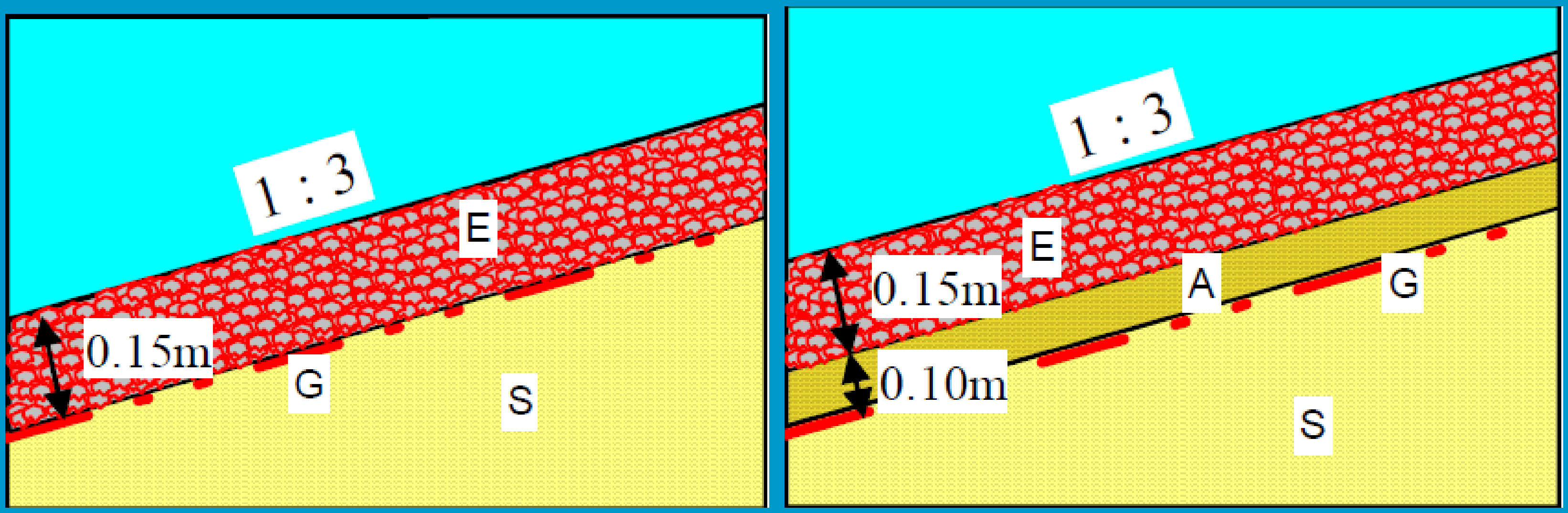
Small scale tests are complicated

- Nearly full scale tests were executed in the GWK-facility in Hannover
- Fully instrumented with nearly 100 sensors (waterpressure, displacement, wave)



Two layouts of the construction

- 15 cm Elastocoast on geotextile, directly on sand
- Same, plus additional 10 cm filter layer (consisting of same material, but without polyurethane)



Failure of the thin model

Regular waves
H = 1.3 m
T = 5 sec

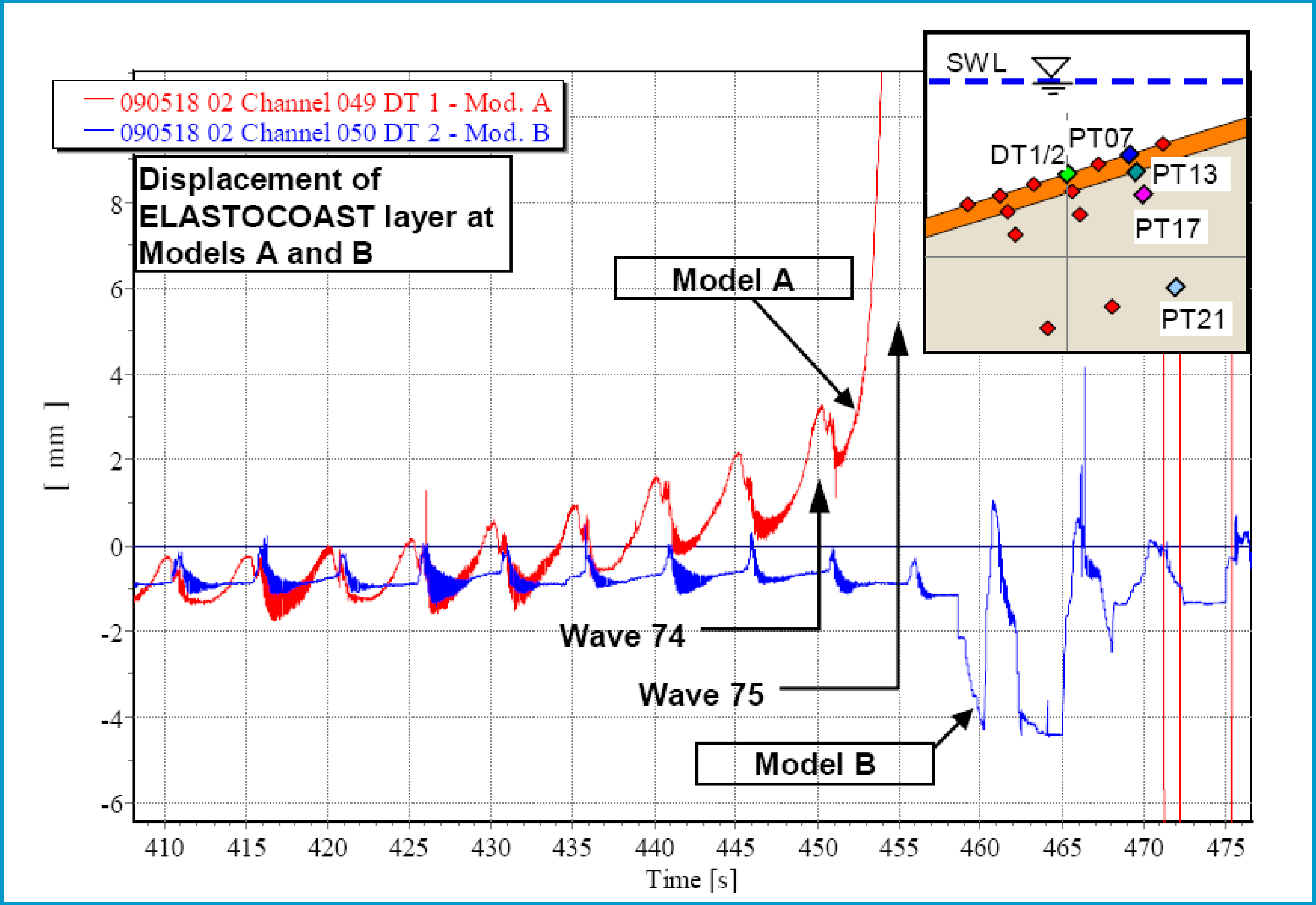
15 cm Elastocoast	15 cm Elastocoast
10 cm filter	
geotextile	geotextile
sand	sand

Test GWK, Oumeraci et.al, 2009

Failure because of exceeding strength of stone

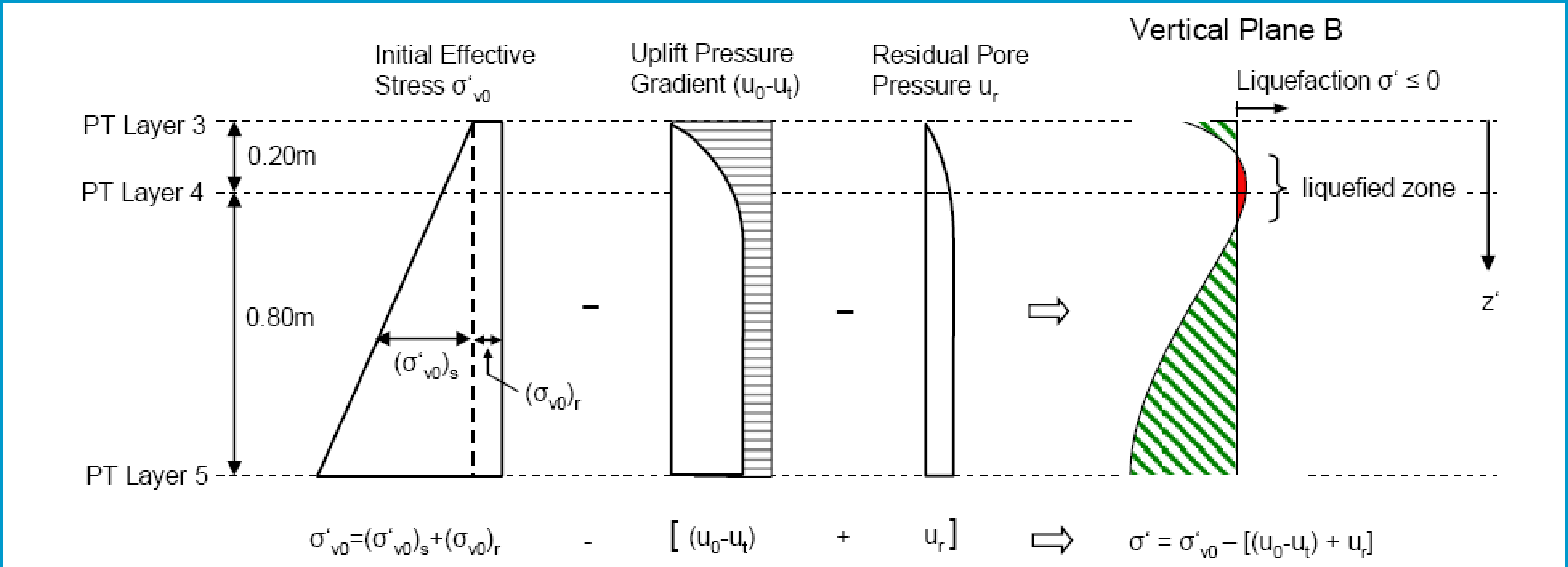
Test GWK, Oumeraci et.al, 2009

Displacement of the Elastocoast



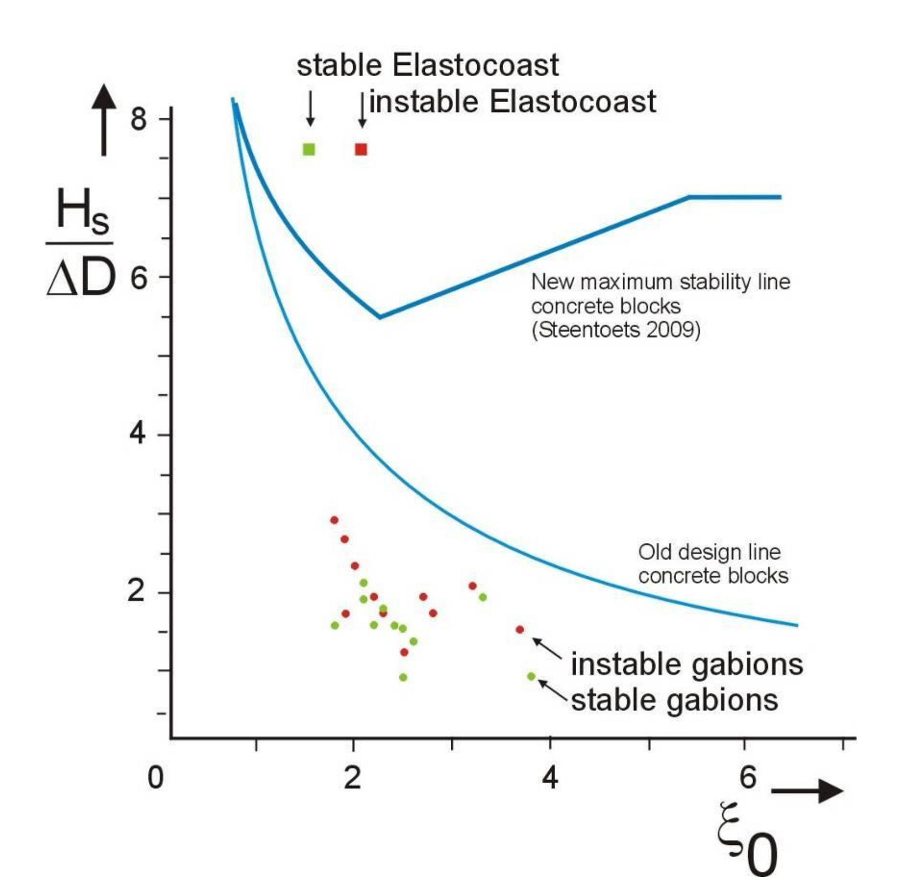
Test GWK, Oumeraci et.al, 2009

Origin of the failure



Test GWK, Oumeraci et.al, 2009

Stability relation for Elastocoast



- Elastocoast is much more stable than gabions
- Elastocoast seems to be more stable than concrete blocks

Preliminary conclusions

- Elastocoast seems to be more stable than concrete blocks
- Thin Elastocoast directly on sand may lead to liquefaction problems
- Breakage is caused by breakage of stones, and not of the bonding with the polyurethane