Bed, Bank and Shoreline protection

Chapter 7: Waves, loads

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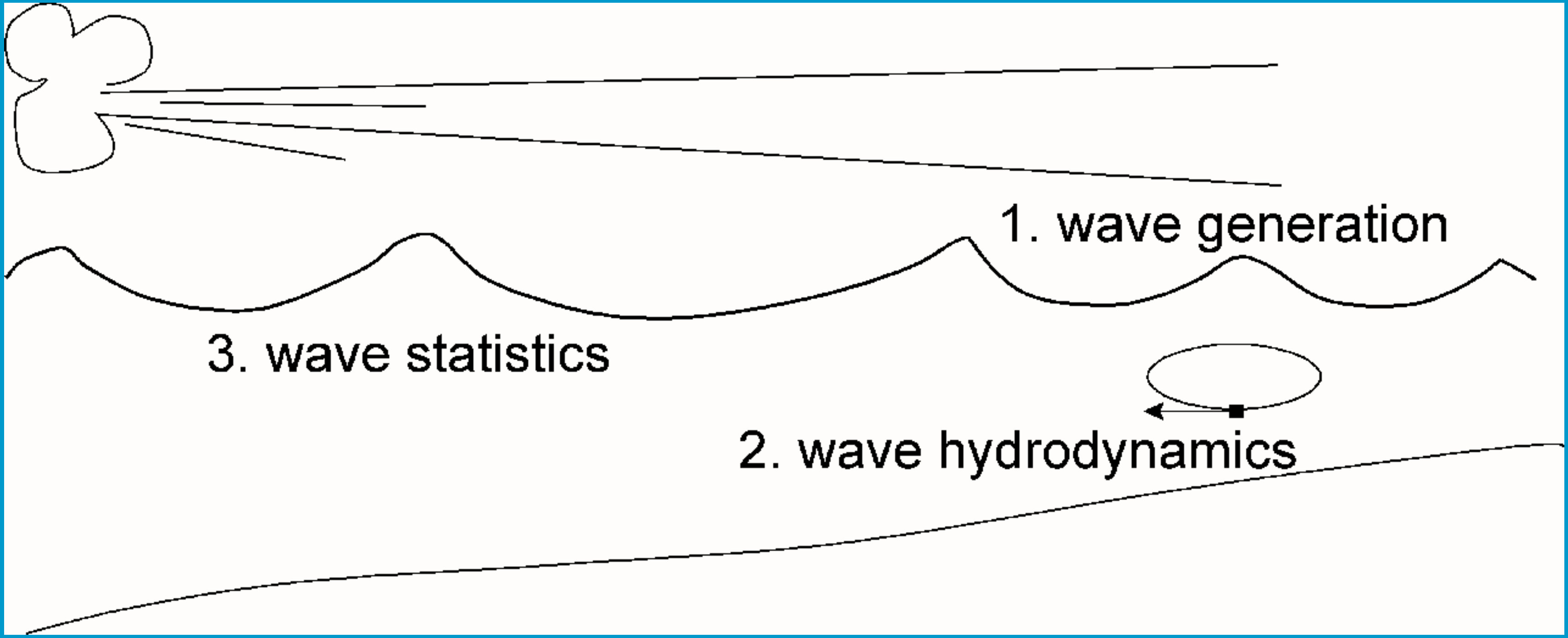
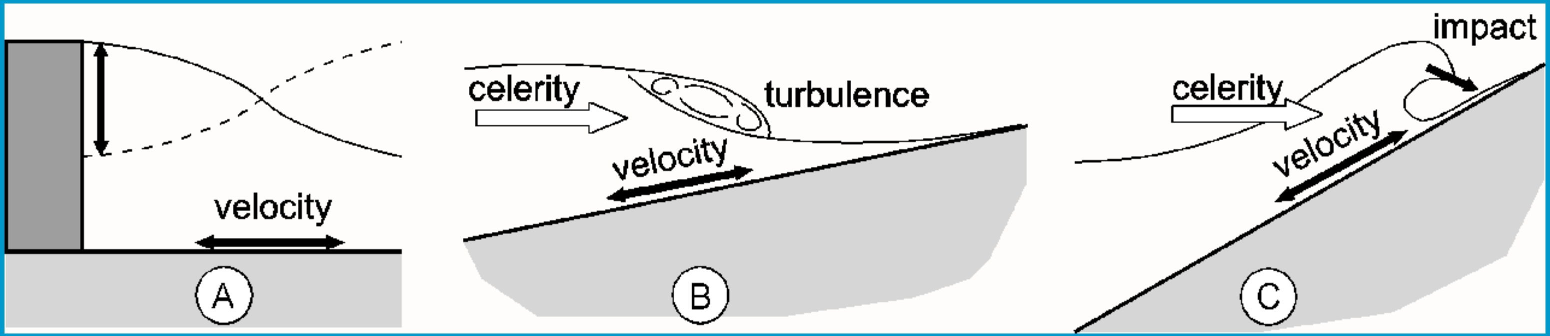
wave issues



examples of wave loads

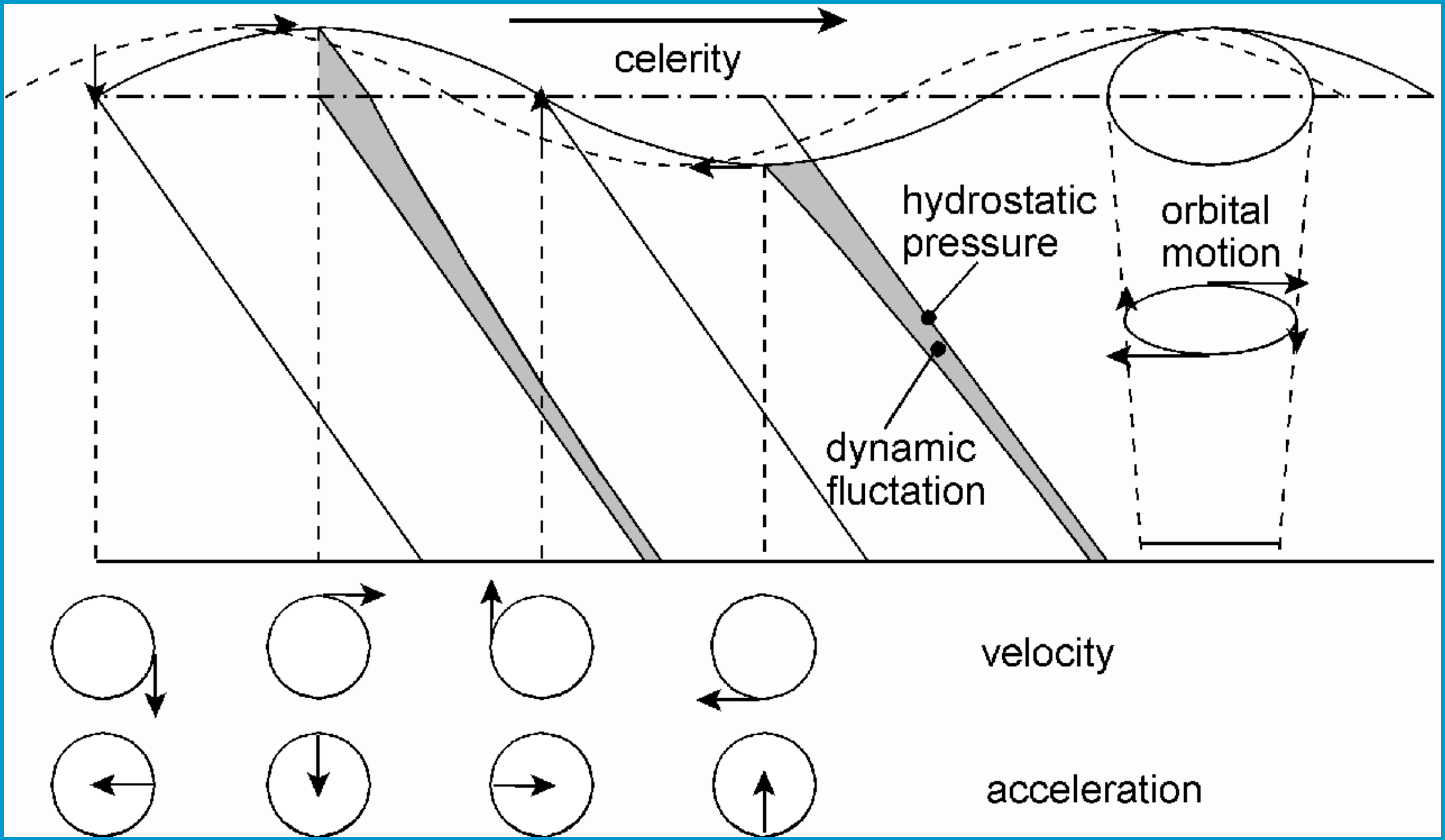


standing waves

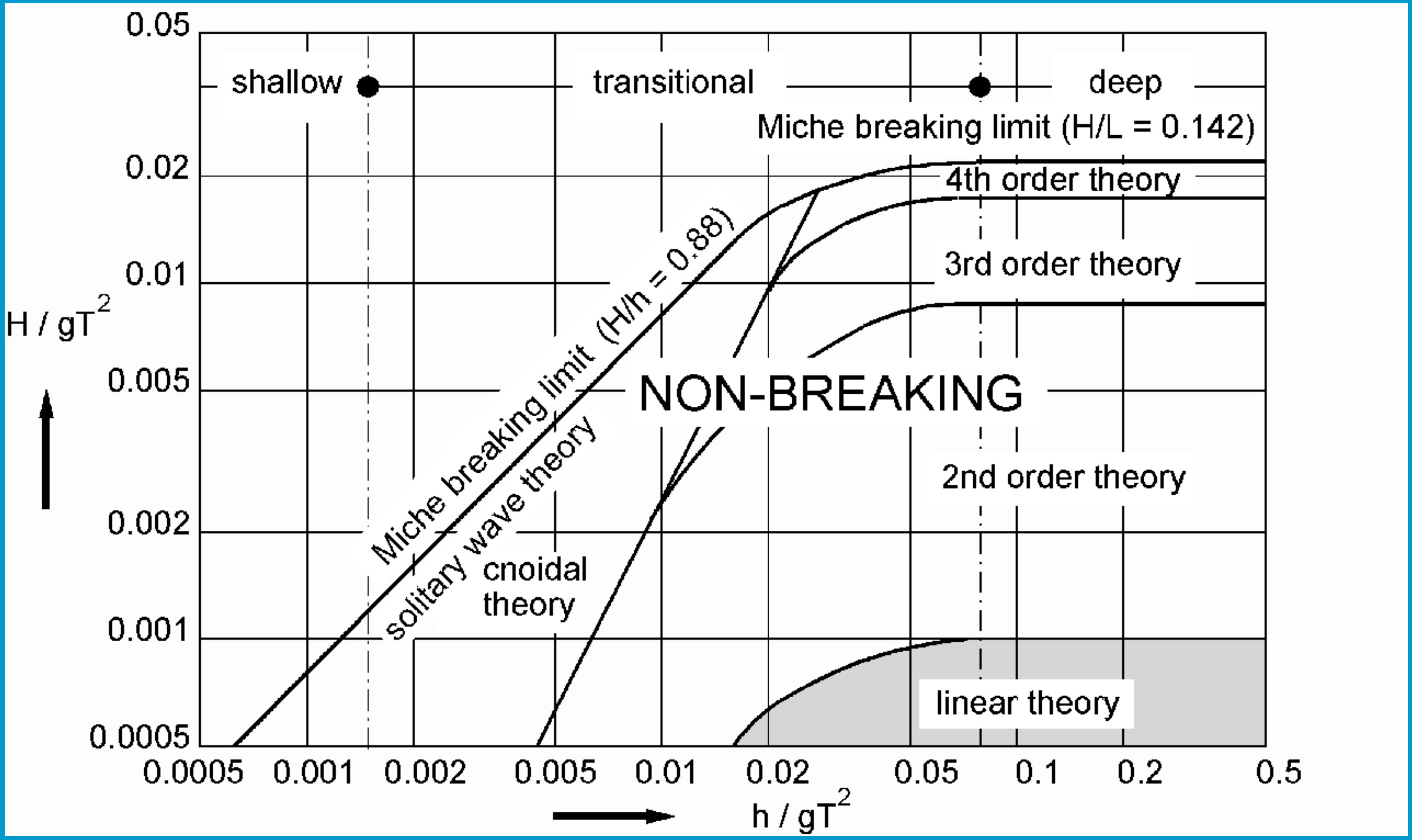
When waves reflect against a vertical, the sum of the incident wave and reflected wave give a standing wave.

In a standing wave orbital movement is different from a progressive wave

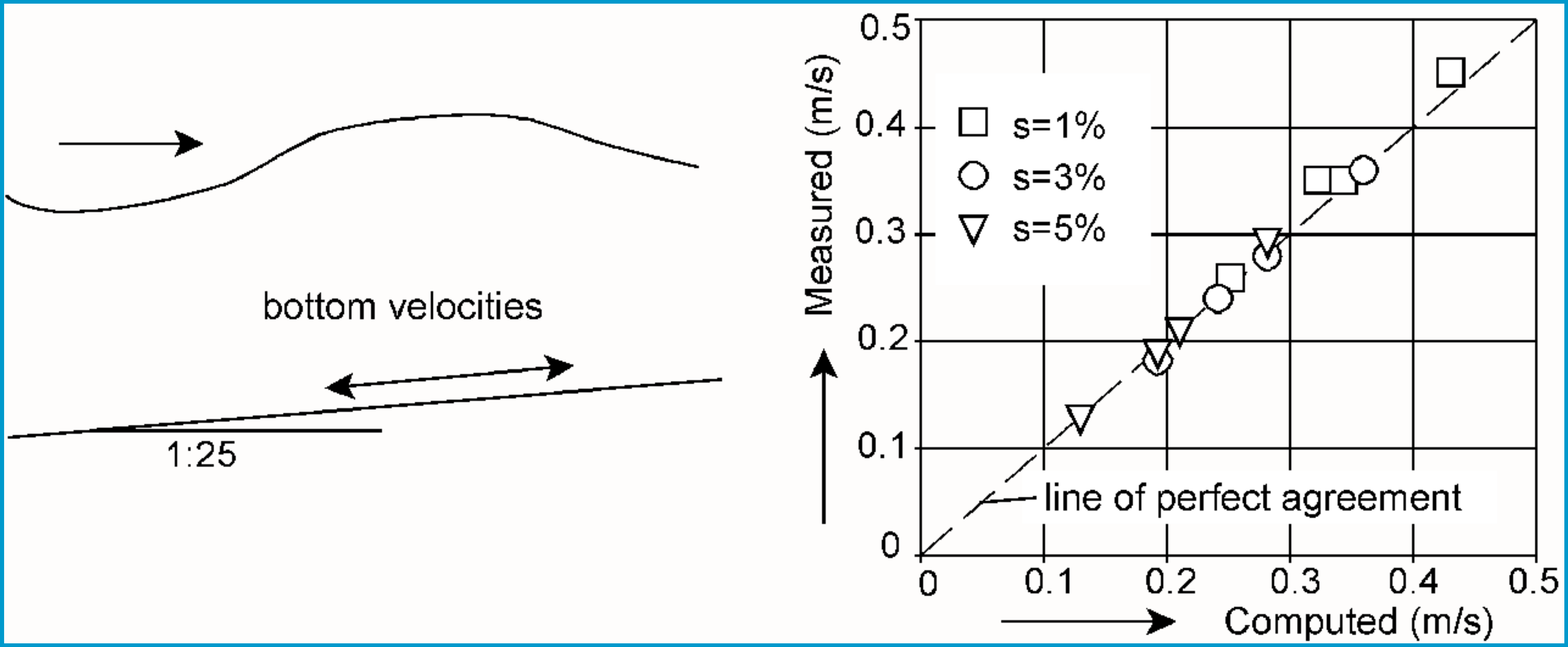
wave motion in periodic, unbroken wave



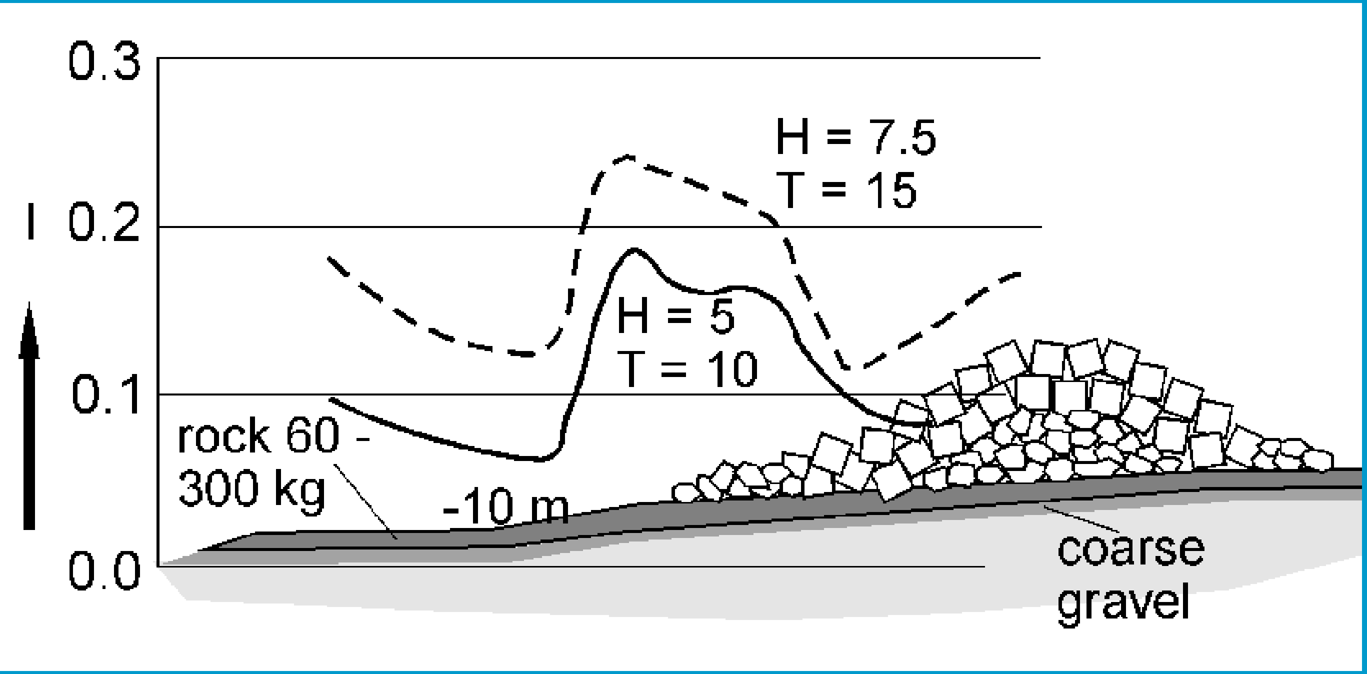
validity of wave theories



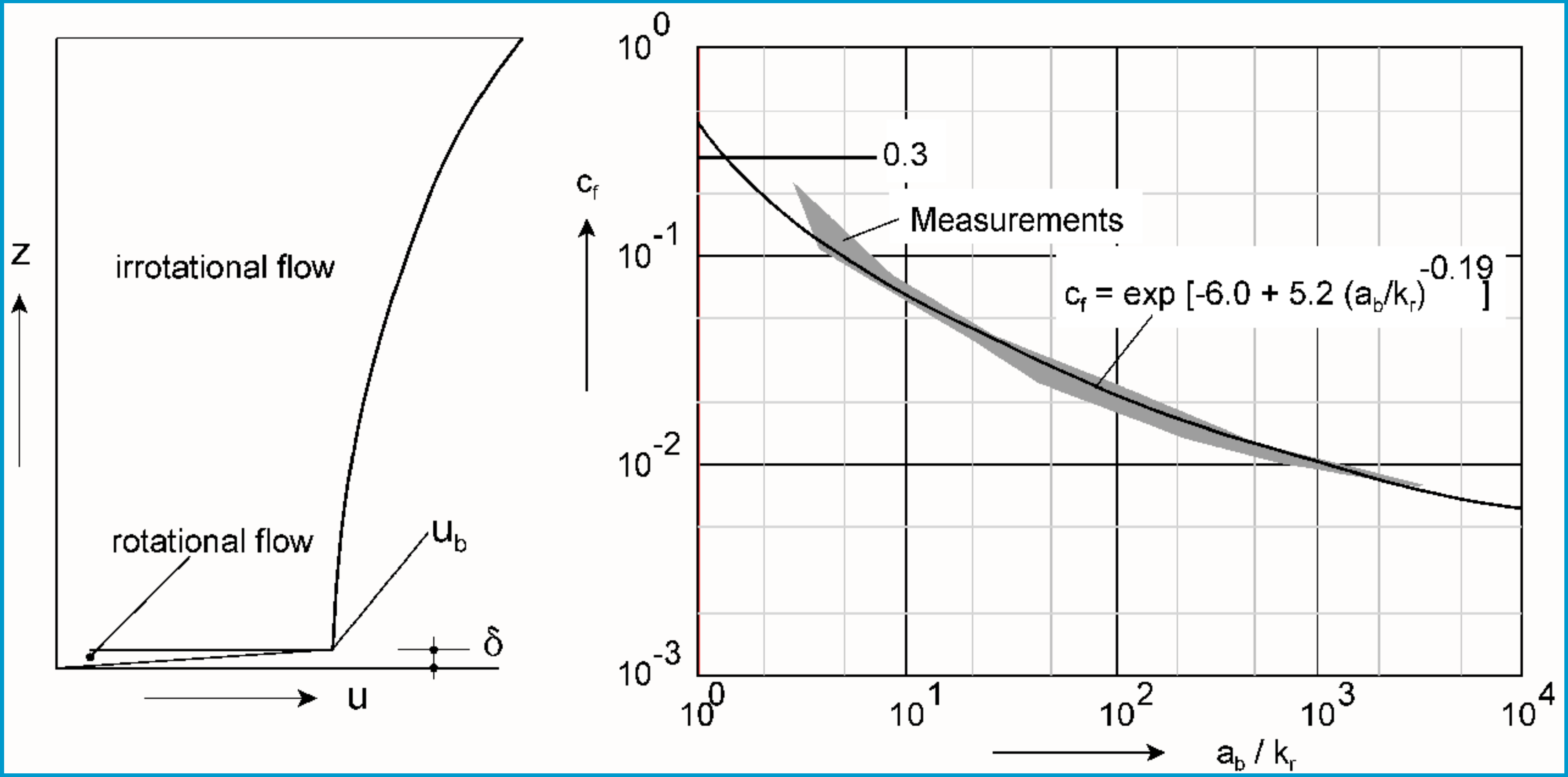
application of linear wave theory



gradient in filter under breakwater



friction under waves



friction factor and c_f

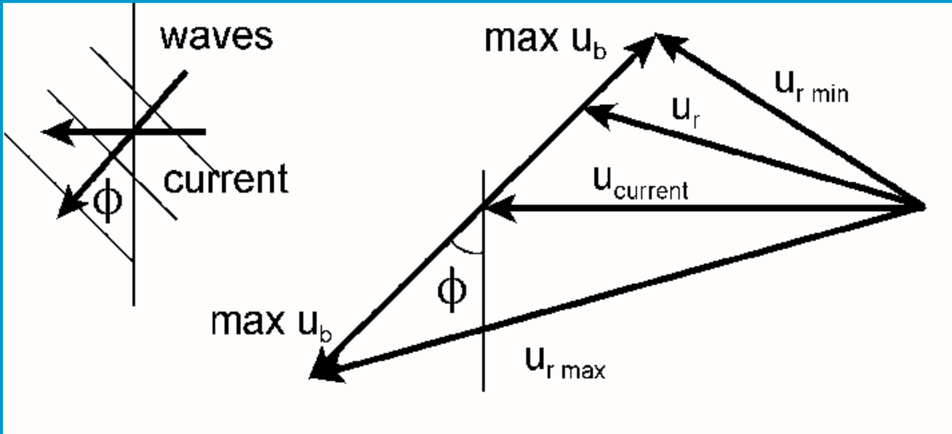
$$\hat{\tau}_w = \tfrac{1}{2}\rho c_f \hat{u}_b^2$$

$$\text{with: } \hat{u}_b = \omega a_b = \frac{\omega a}{\sinh kh}$$

$$u = \hat{u}_b \sin \omega t$$

$$c_f = \exp\left[-6.0 + 5.2\left(a_b / k_r\right)^{-0.19}\right] \quad \text{with: } c_{f\,\max} = 0.3$$

waves and currents (1)



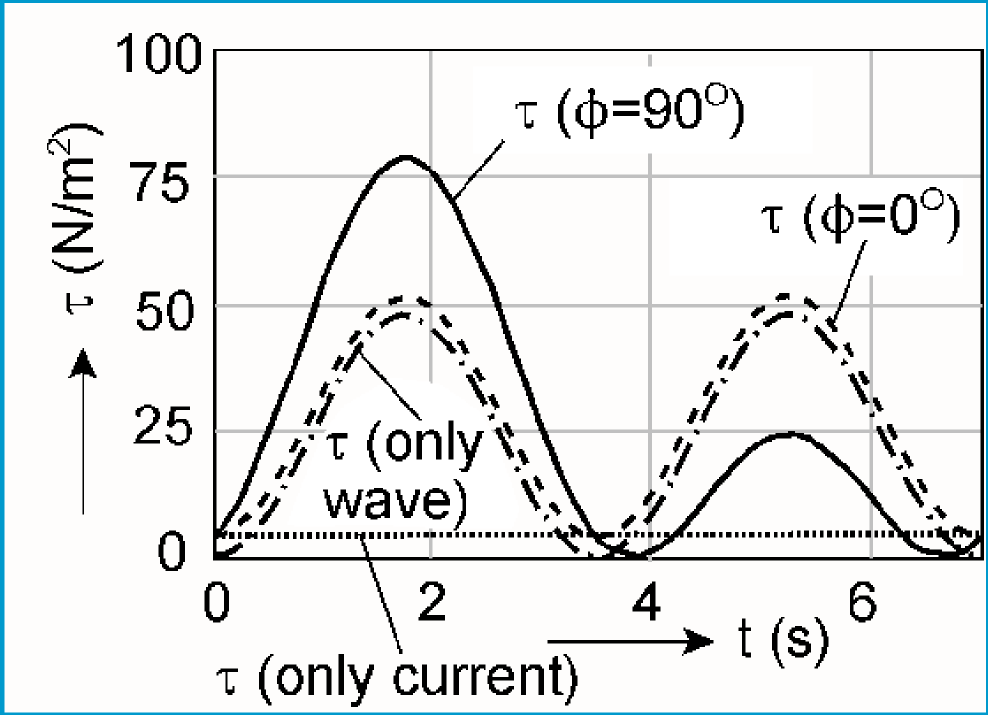
$$u_{c-t} = \frac{\sqrt{g}}{\kappa C} u_c \quad \text{and} \quad u_{b-t} = \frac{1}{\kappa}\sqrt{\frac{c_f}{2}} u_b \sin(\omega t)$$

$$u_r = \sqrt{\frac{g}{\kappa^2 C^2} u_c^2 + \frac{c_f}{2\kappa^2} u_b^2 \sin^2(\omega t) + 2\frac{\sqrt{g}}{\kappa C} u_c \frac{1}{\kappa}\sqrt{\frac{c_f}{2}} u_b \sin(\omega t) \sin(\phi)}$$

$$\tau_r = \rho \kappa^2 u_r^2$$

current and waves (2)

$H = 3$ m
$T = 7$ sec
$u_b = 1.29$ m/s
$c_f = 0.057$
$C = 50\ \sqrt{m}/s$



Nearshore effects

- Shoaling
- Refraction
- Diffraction
- Breaking
- Reflection

breaking waves

$$H_b = 0.142\,L\tanh\left(\frac{2\pi}{L}h\right)$$

$$\frac{H_b}{h} \approx 0.78\ (solitary\ wave)$$

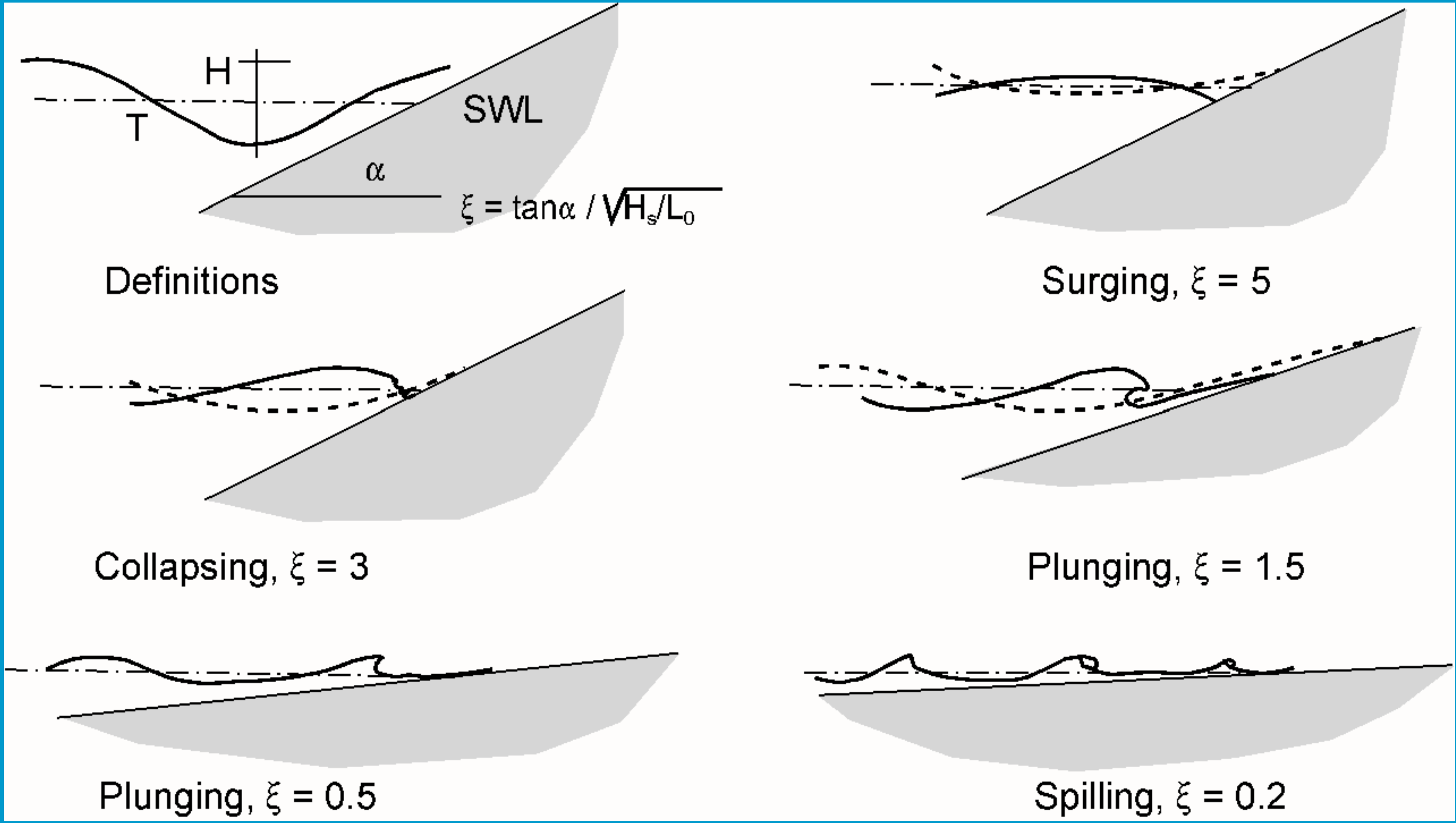
$$\frac{H_s}{h} \approx 0.4 - 0.5$$

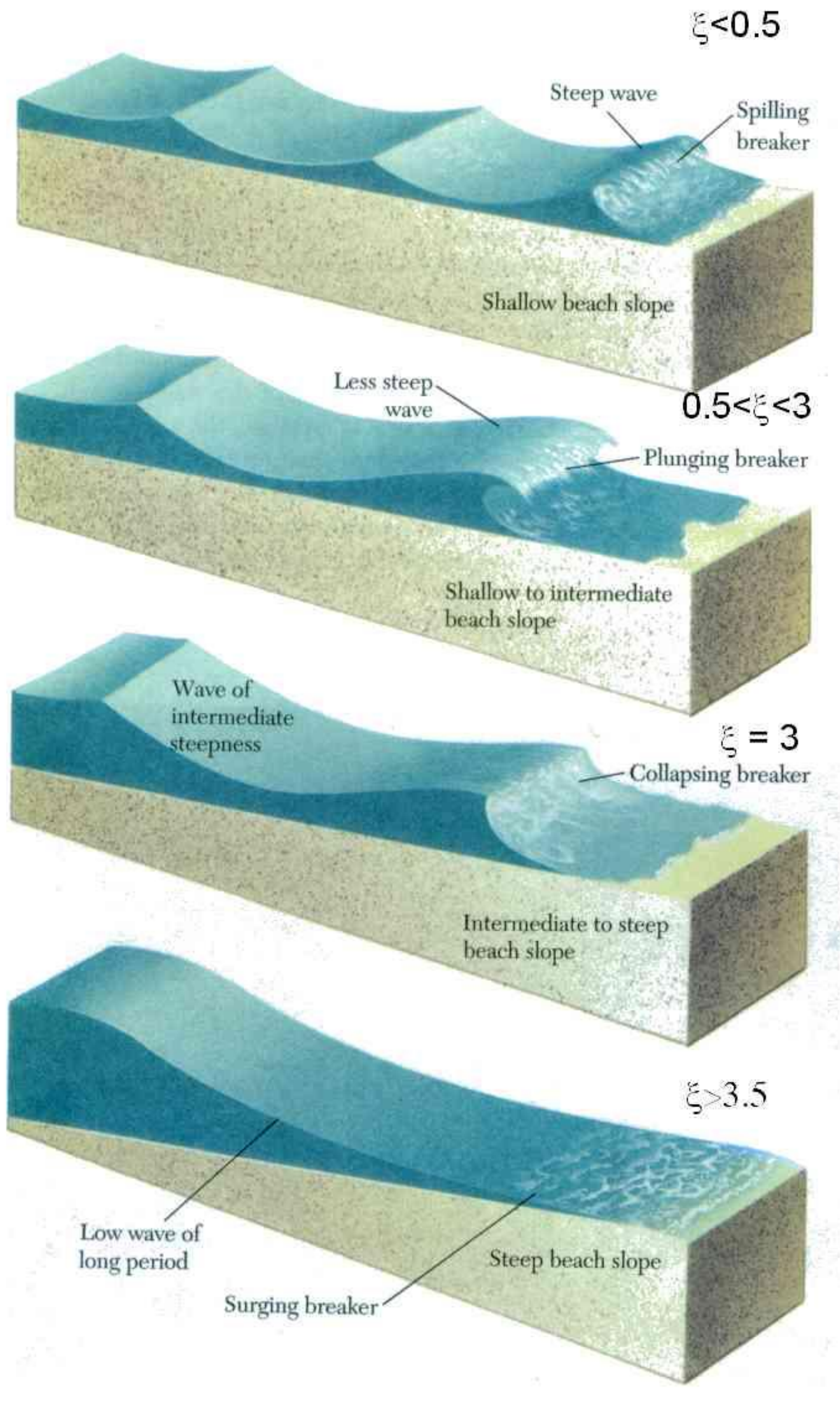
the iribarren number
(surf similarity parameter)

$$\xi = \frac{\tan\alpha}{\sqrt{H/L_0}}$$

tan α	**slope of the shoreline/structure**
H	**wave height**
L_0	**wave length at deep water**

breaker types (1)





breaker types

spilling $\xi < 0.5$

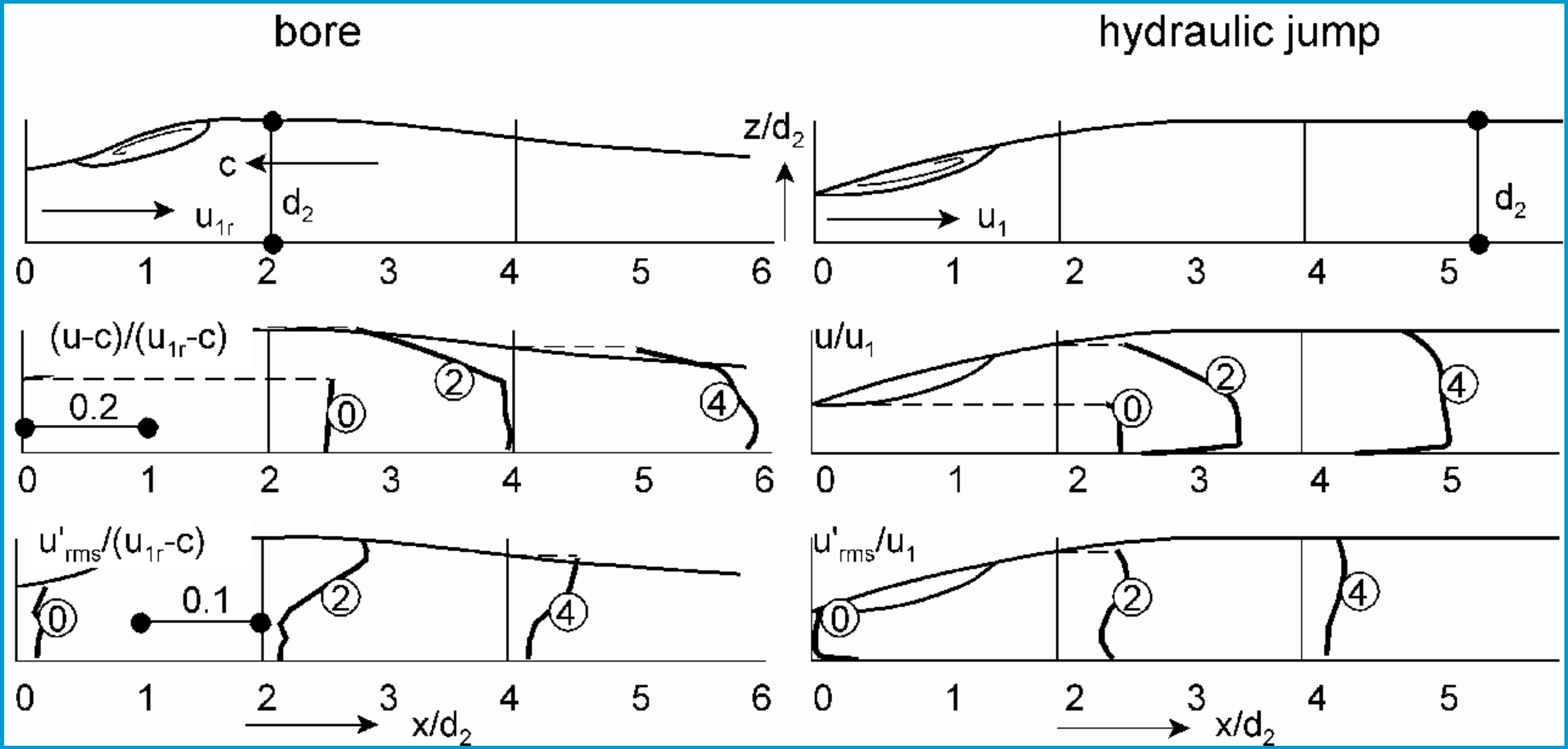
plunging $0.5 < \xi < 3$

collapsing $\xi = 3$

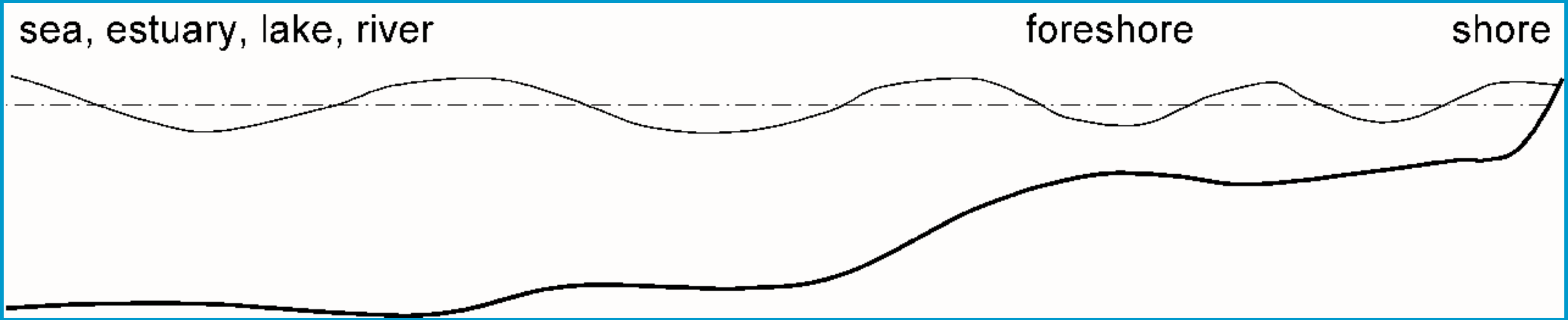
surging $\xi > 3$

TU Delft

bore and hydraulic jump



waves on a foreshore



$$D = \rho g \frac{H^3}{4T h}$$

energy loss in a bore (analogy to hydraulic jump)
(H= wave height, T= wave period, h= water depth

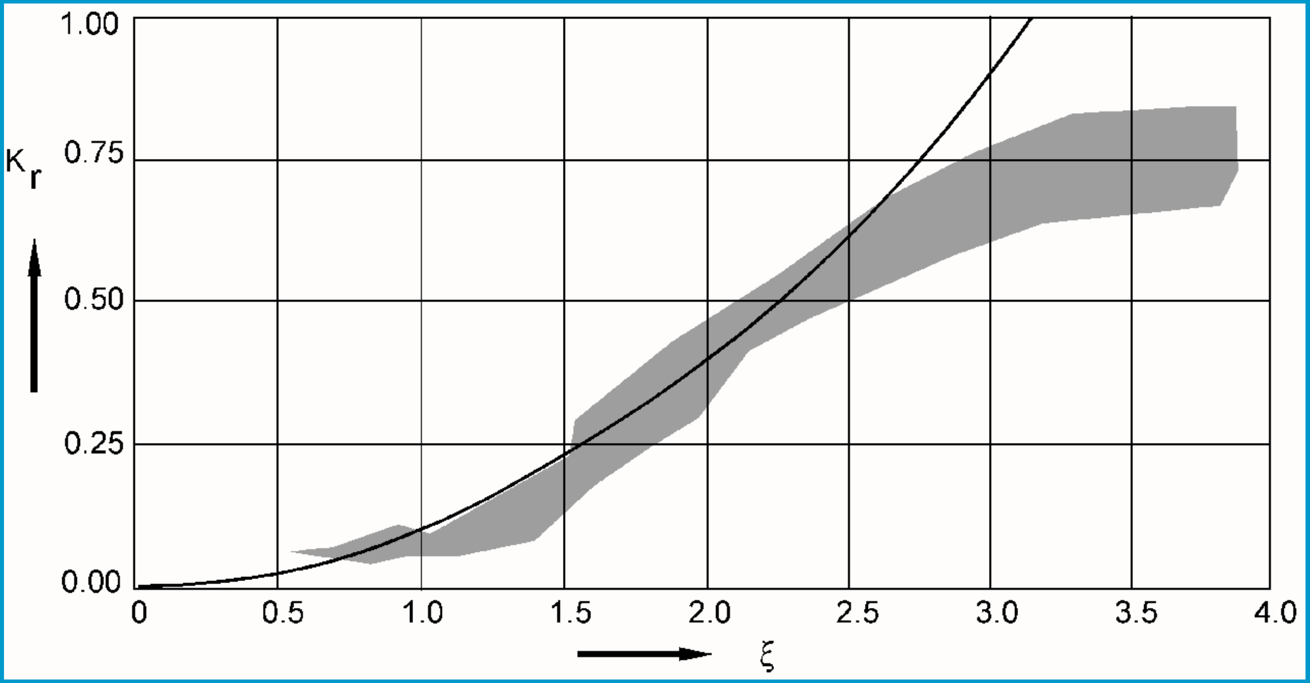
$$D = \frac{1}{4} Q_b \rho g \frac{H_m^2}{T_p}$$

energy loss during the breaking process (Battjes/Janssen method)

$$\frac{1 - Q_b}{\ln Q_b} = -\left(\frac{H_{rms}}{H_m}\right)^2$$

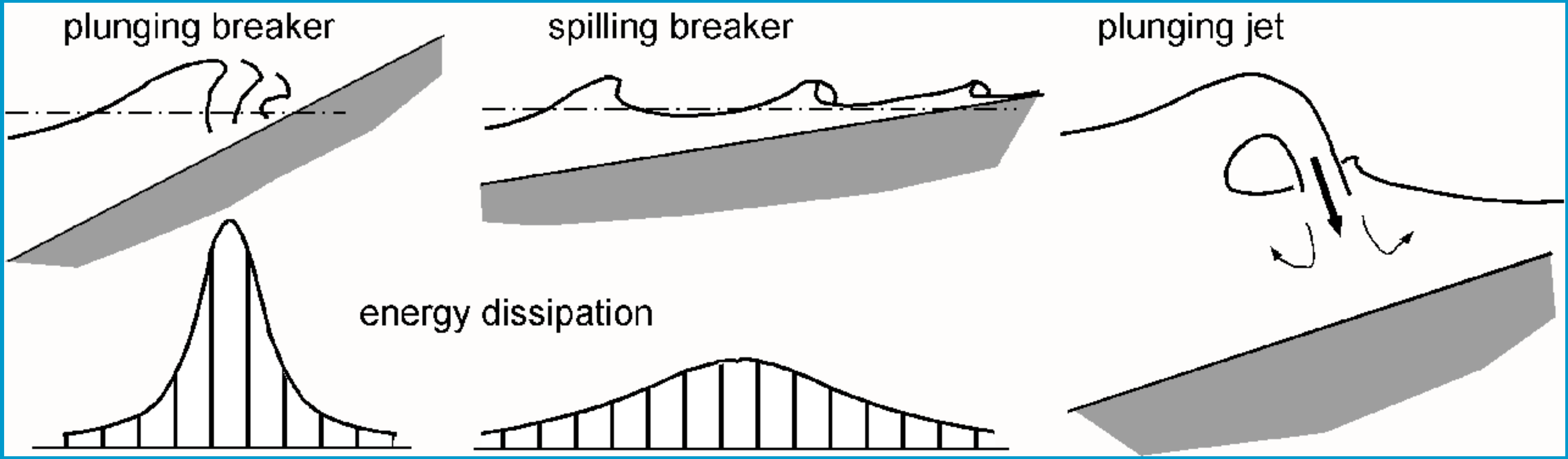
introducing wave distribution
Q_b is the fraction of all waves broken

reflection

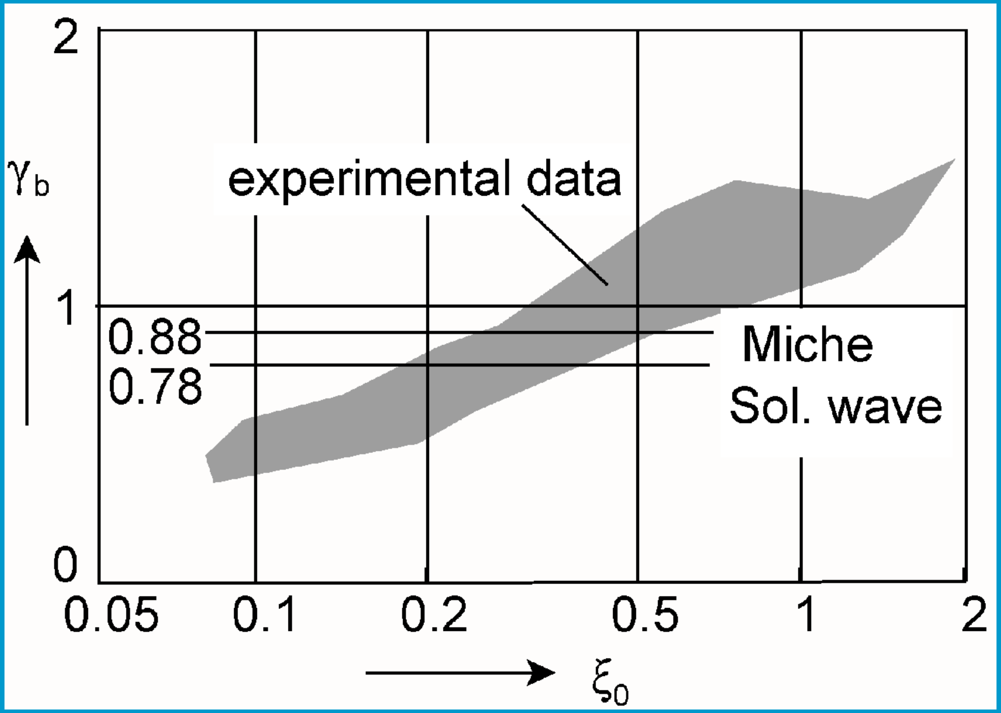


$$K_r = \frac{H_R}{H_I} \approx 0.1\xi^2$$

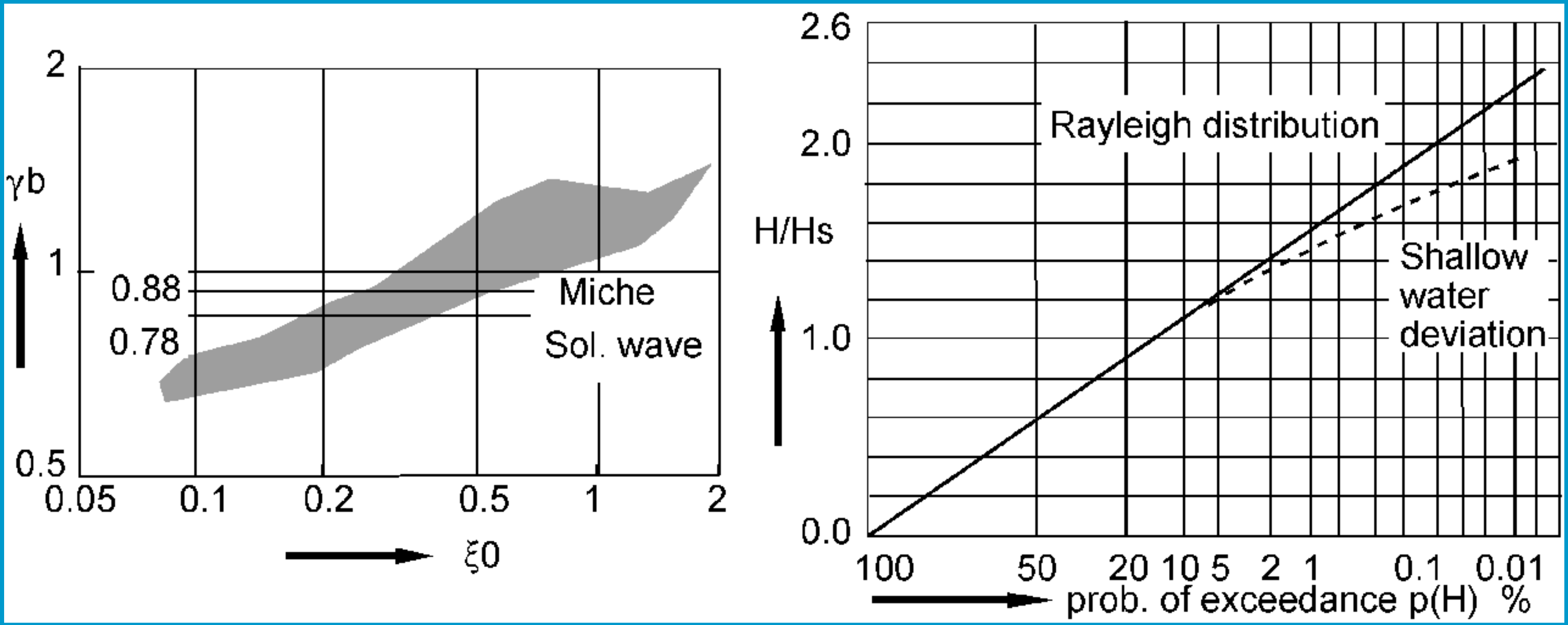
absorption


plunging breaker
spilling breaker
plunging jet
energy dissipation

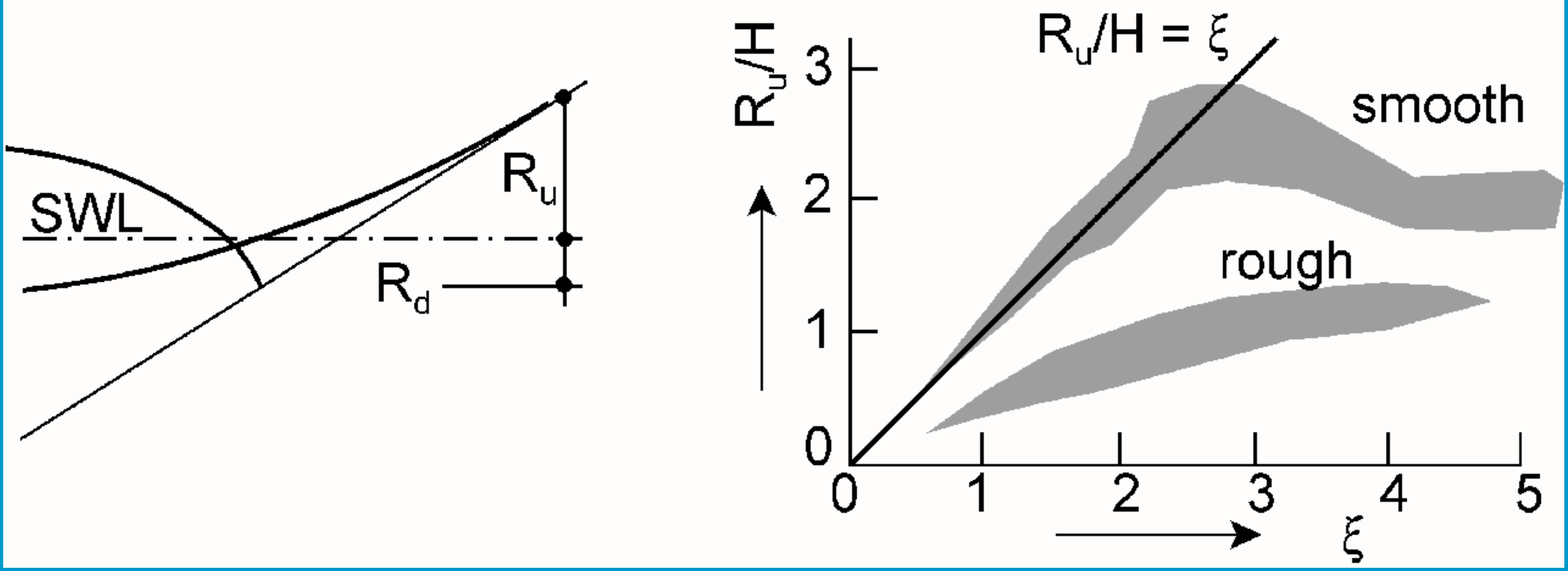
breakerdepth



change of distribution in shallow water



run up



2% run-up ($R_{2\%}$)

There is a linear relation between H and R_u
When we assume H is Rayleigh distributed then R_u is also Rayleigh distributed

$$\frac{R_n}{R_{2\%}} = 0.71\sqrt{-\frac{1}{2}\ln(n)}$$

n = exceedance percentage (e.g. 0.01)

Why 2% and not 3% ?????

Old Delft Formula

$$R_{2\%} = 8 H_s \tan\alpha$$

- Valid for $\tan\alpha < 1/3$ and relatively smooth slopes
- Valid for “normal” wave steepness (between 4 and 5 %)

Run-up calculation

$$\frac{R_u}{H} = \xi$$ Hunt's formula

$$R_{u2\%} = 1.5\gamma_r \gamma_\beta \gamma_B \gamma_f H_s \xi_p \quad (R_{u\,2\%\,\max} = 3H_s)$$

correction factors:

- γ_r roughness
- γ_β approach angle
- γ_B berm reduction
- γ_f foreshore reduction

EurOtop (TAW, Van der Meer)

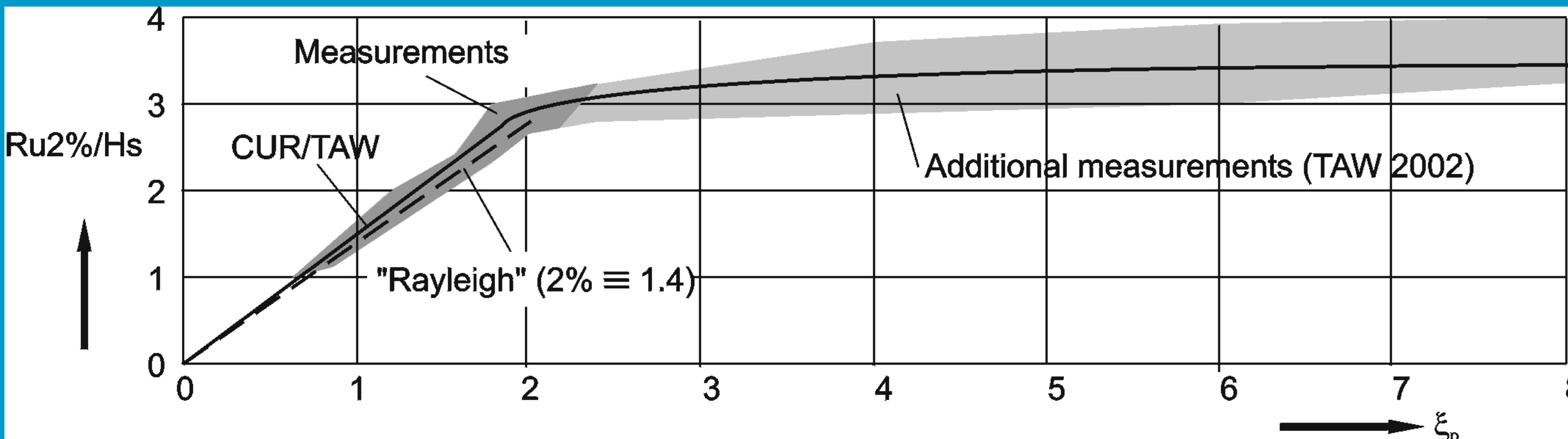
$$\frac{R_{2\%}}{H_s} = 1.65\,\xi_0 \qquad for \quad \xi_0 \leq 1.6$$

$$\frac{R_{2\%}}{H_s} = 4.0 - \frac{1.5}{\sqrt{\xi_0}} \qquad for\ 1.6 < \xi_0 < 10$$

H_s = significant wave height
ξ_0 = breaker parameter based on $T_{m-1,0}$

Dutch and English version downloadable from www.ENWinfo.nl
See also: www.overtopping-manual.com

EurOtop formula



example

Given: $H_s = 2.5$ m $T_p = 8$ s   smooth slope 1:4

How much is Run-up ??

Old Delft Formula: R = 8 * 2.5 / 4 = 5 m

New Delft Formula:

$T_{m-1,0} = 0.9\ T_p = 7.2$ s

$\xi = 0.25 / \sqrt{(2.5/\{1.56*7.2^2\})} = 1.42$

R = 1.65 * 2.5 * 1.42 = 5.86

friction values

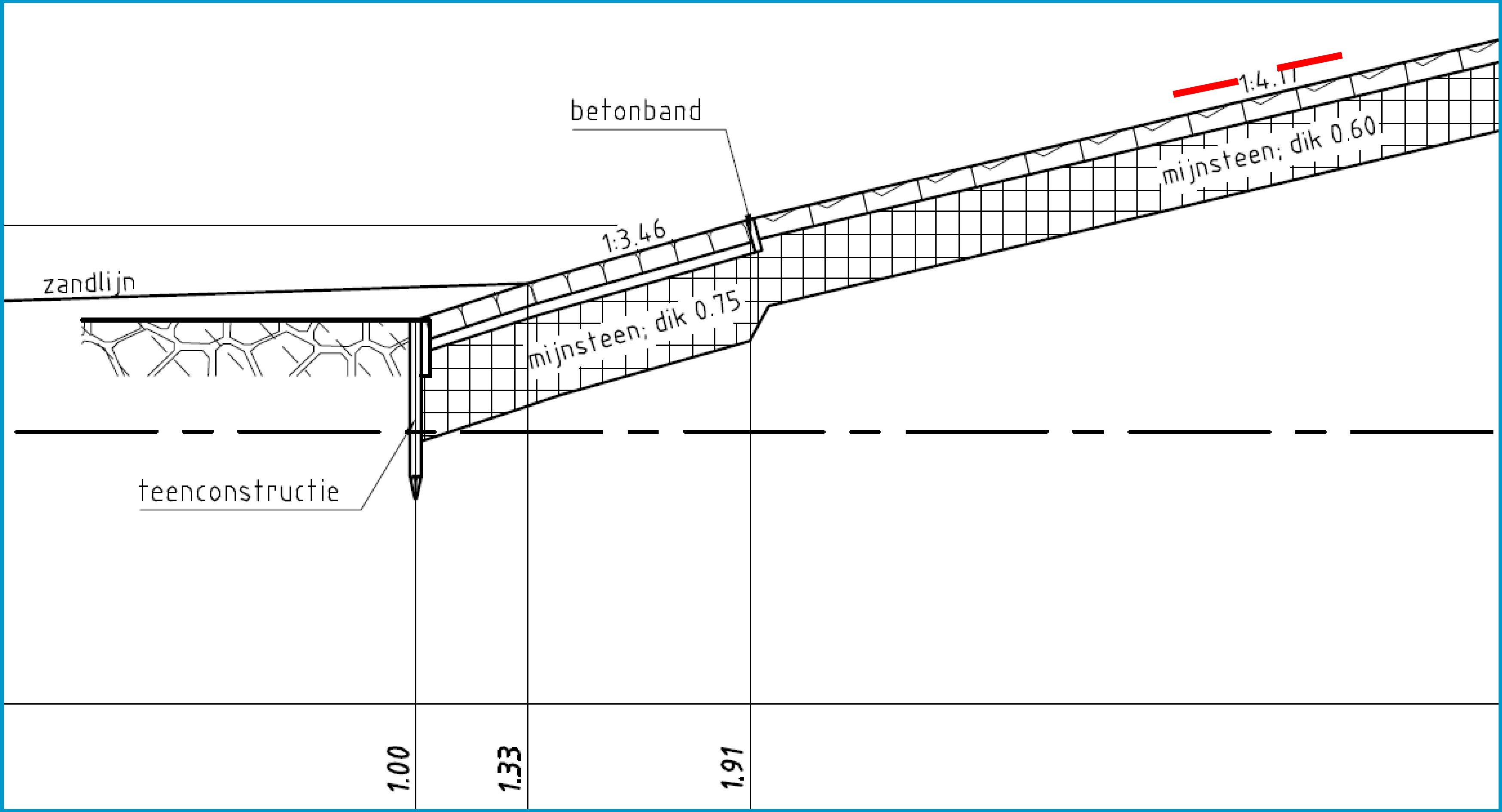
γ_r	Type of revetment
1.0	Asphalt, concrete, smooth blocks, grass, Sand-asphalt
0.95	Blocks in asphalt or concrete matrix, blocks with grass
0.90	Basalt, Basalton, Hydroblock, Haringman, Fixstone, Armorflex
0.85	Lessinische and Vilvoordse, small roughness blocks
0.80	riprap penetrated with asphalt
0.70	Single layer of riprap
0.55	Double layer of riprap

Run up on Elastocoast and Haringman

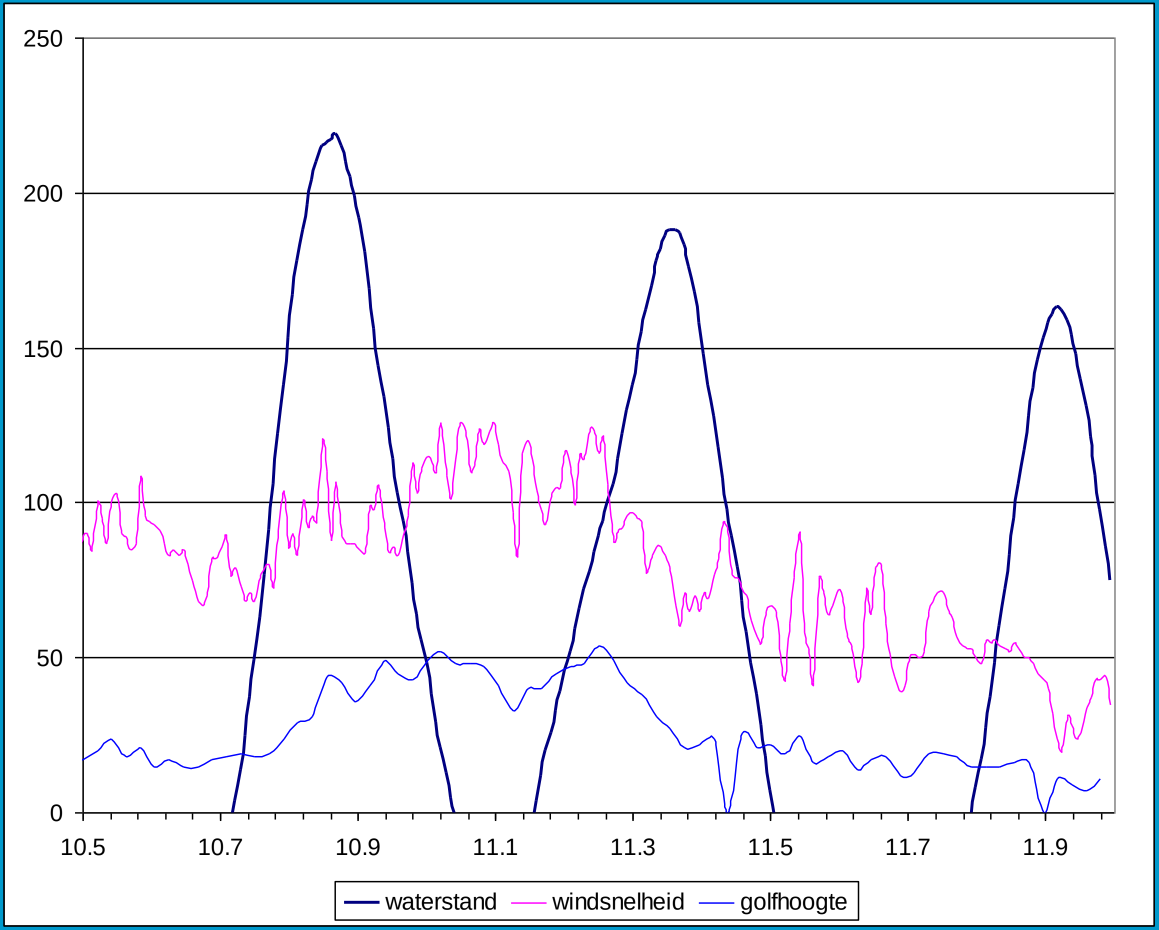
10 9 8 7 6

foto Elastogran-BASF

cross section of the location



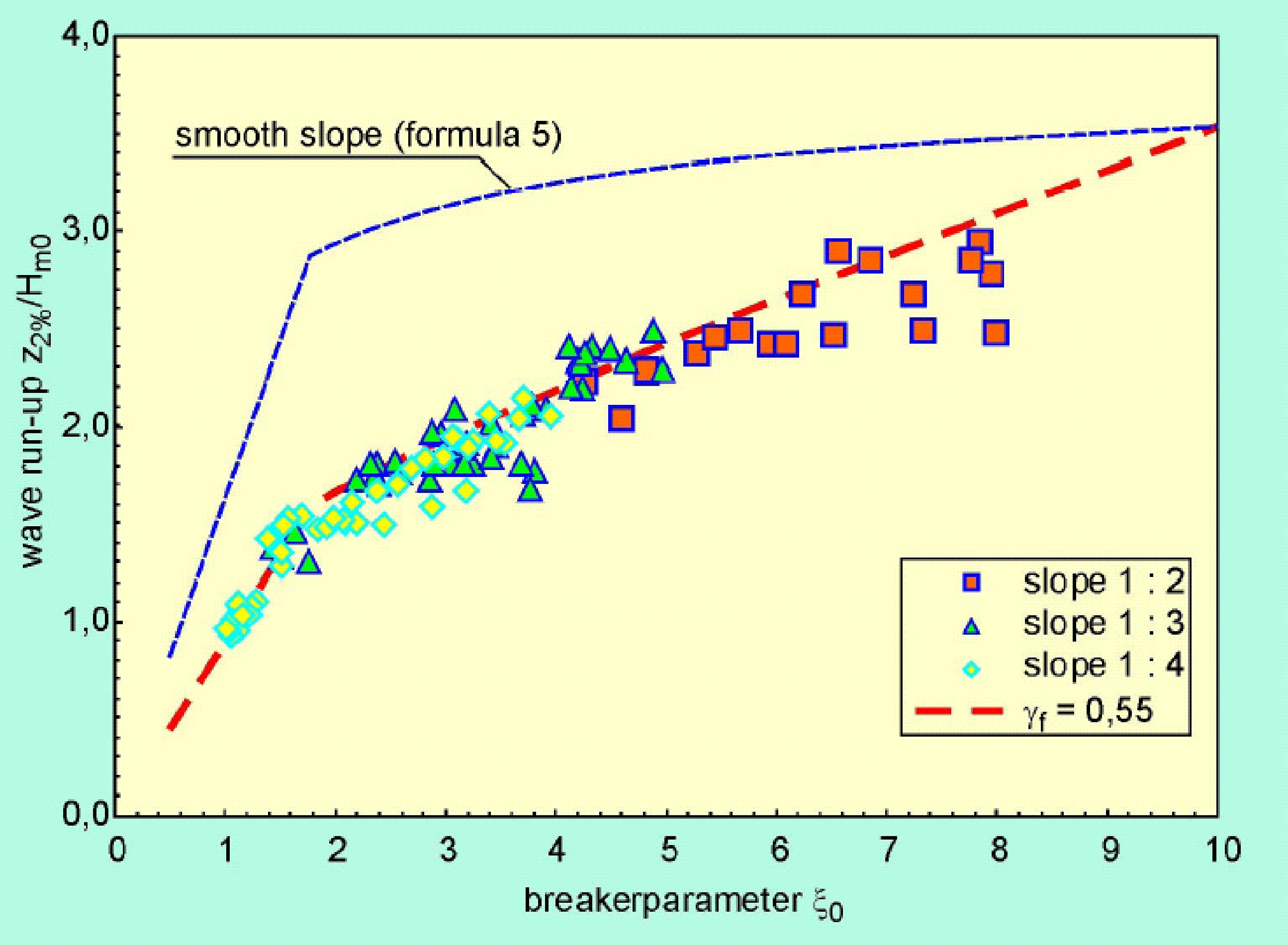
Waves and waterlevels on 12/10/2009



computation

case	Runup on slope	Runup above msl	Runup above HW	f*8Htanα
Har high	0.5*9/4 =1.25	3.25	3.25-2.20= 1.05	1.0
Har low	0.5*8/4= 1.00	3.00	3.00-1.90= 1.10	1.0
Elas high	0.5*4/4= 1.00	3.00	3.00-2.20= 0.80	0.80
Elas low	0-.5*3/4= 0.75	2.75	2.75-1.90= 0.85	0.80

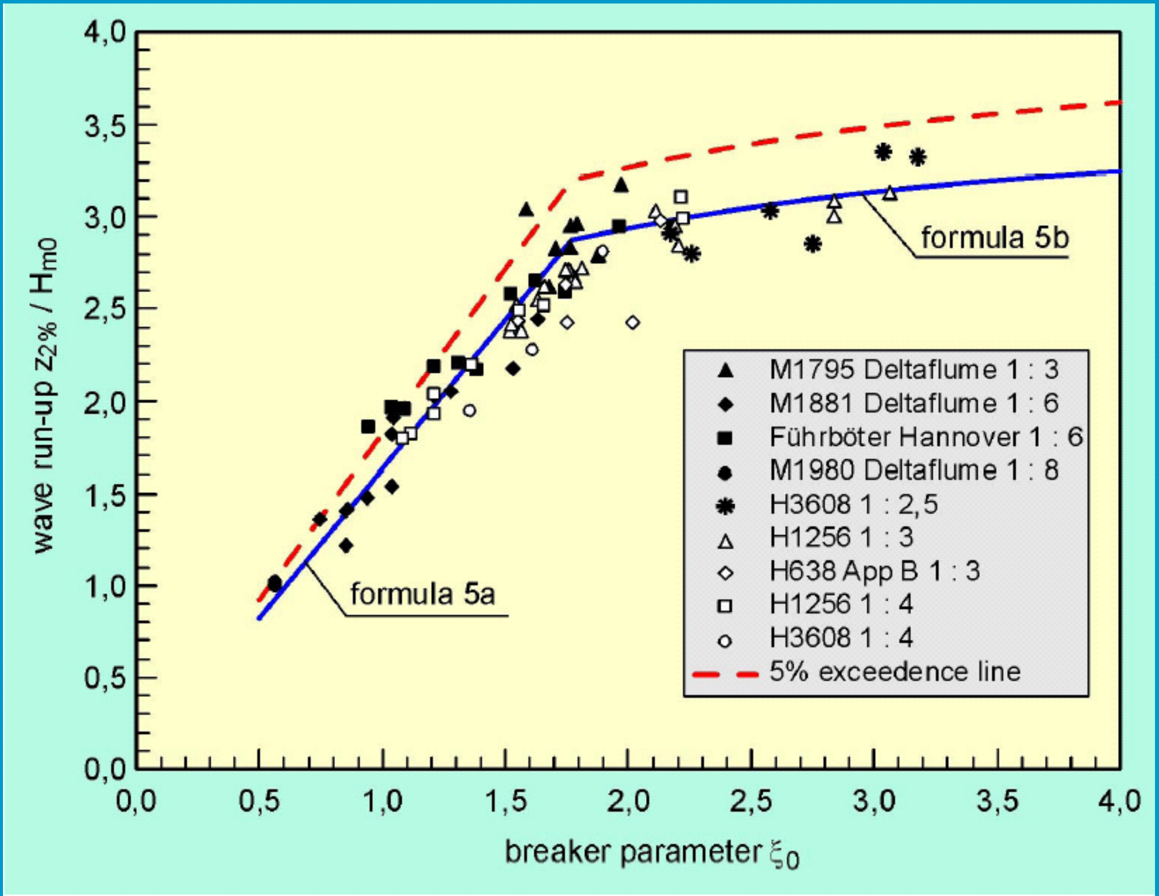
Very rough slopes



Example with Cress

run demo Cress

Relative run-up measurements
deep water, smooth slope



from TAW-report
Run-up and Overtopping
may 2002

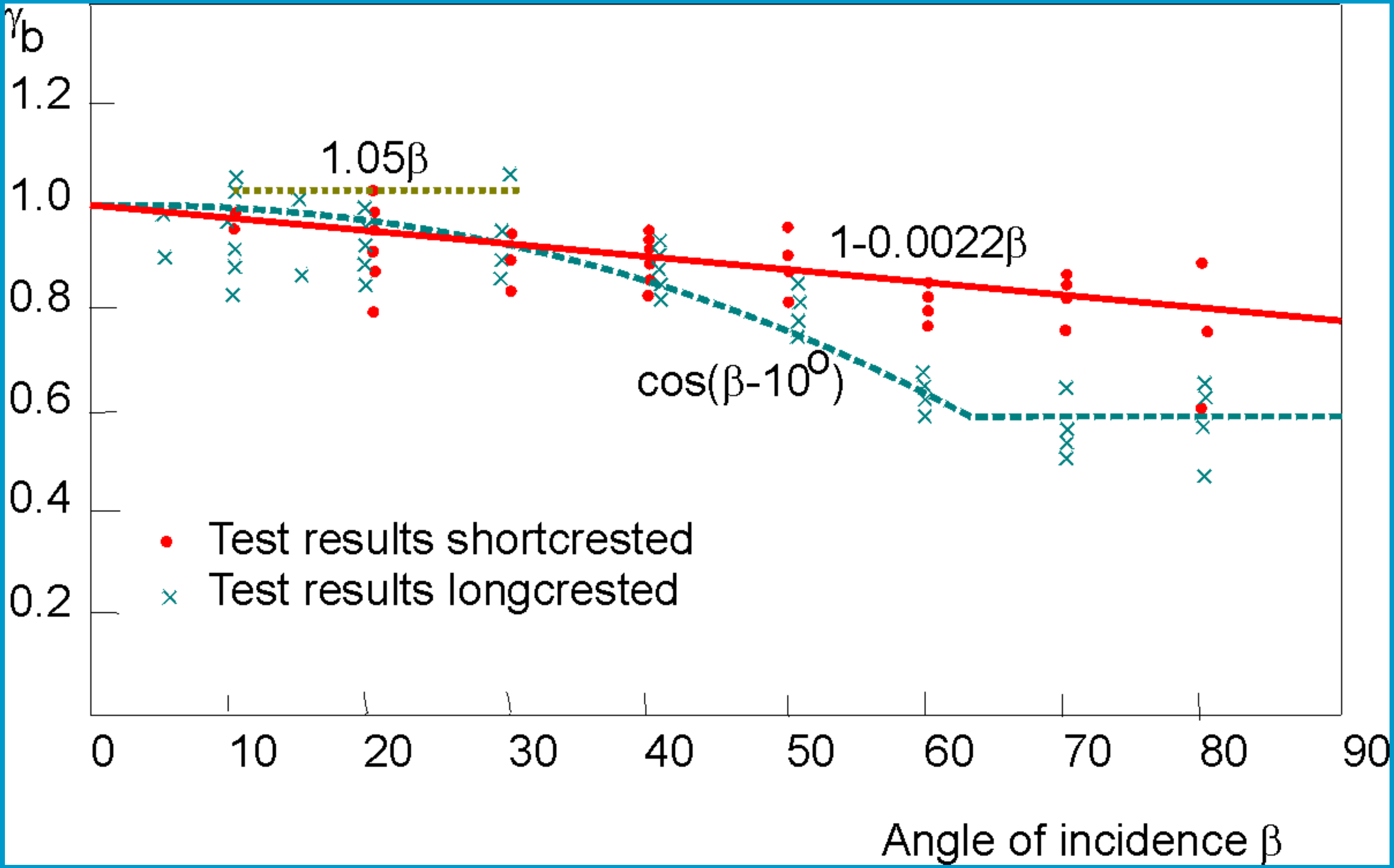
Short crested and long crested

Old equations assumed regular waves
New equation assumes shortcrested waves

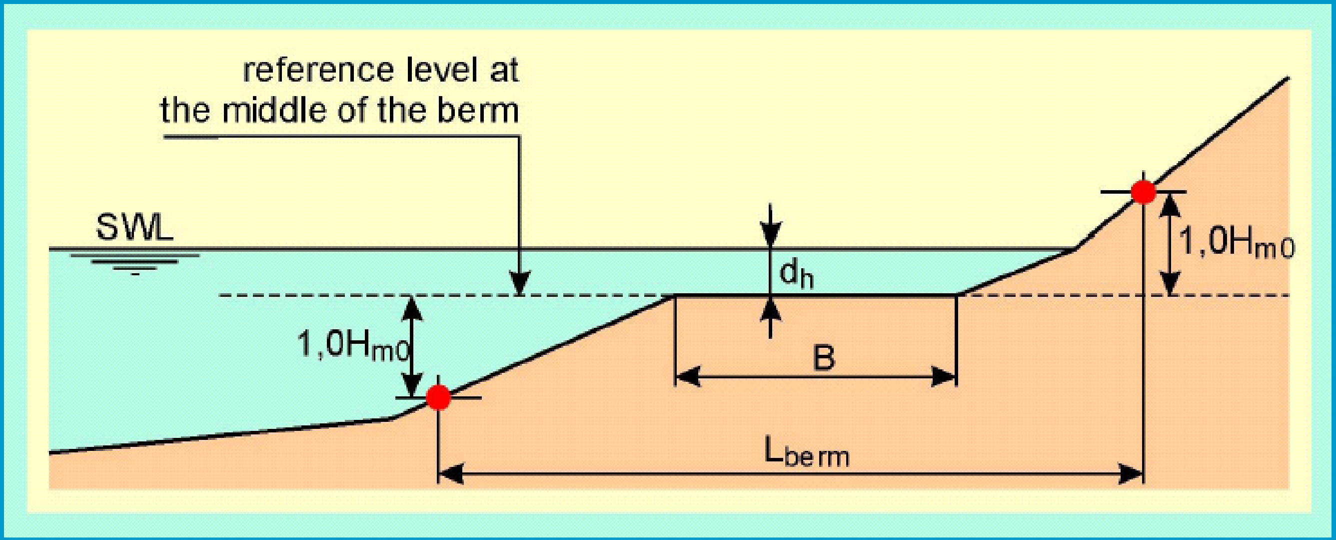
Important for oblique wave attack:

Van der Meer: $\gamma_b = 1 - 0.0022\ \beta$
Old Equation: $\gamma_b = \cos(\beta - 10°)$ for $\beta < 65°$

shortcrested and longcrested



berm effect

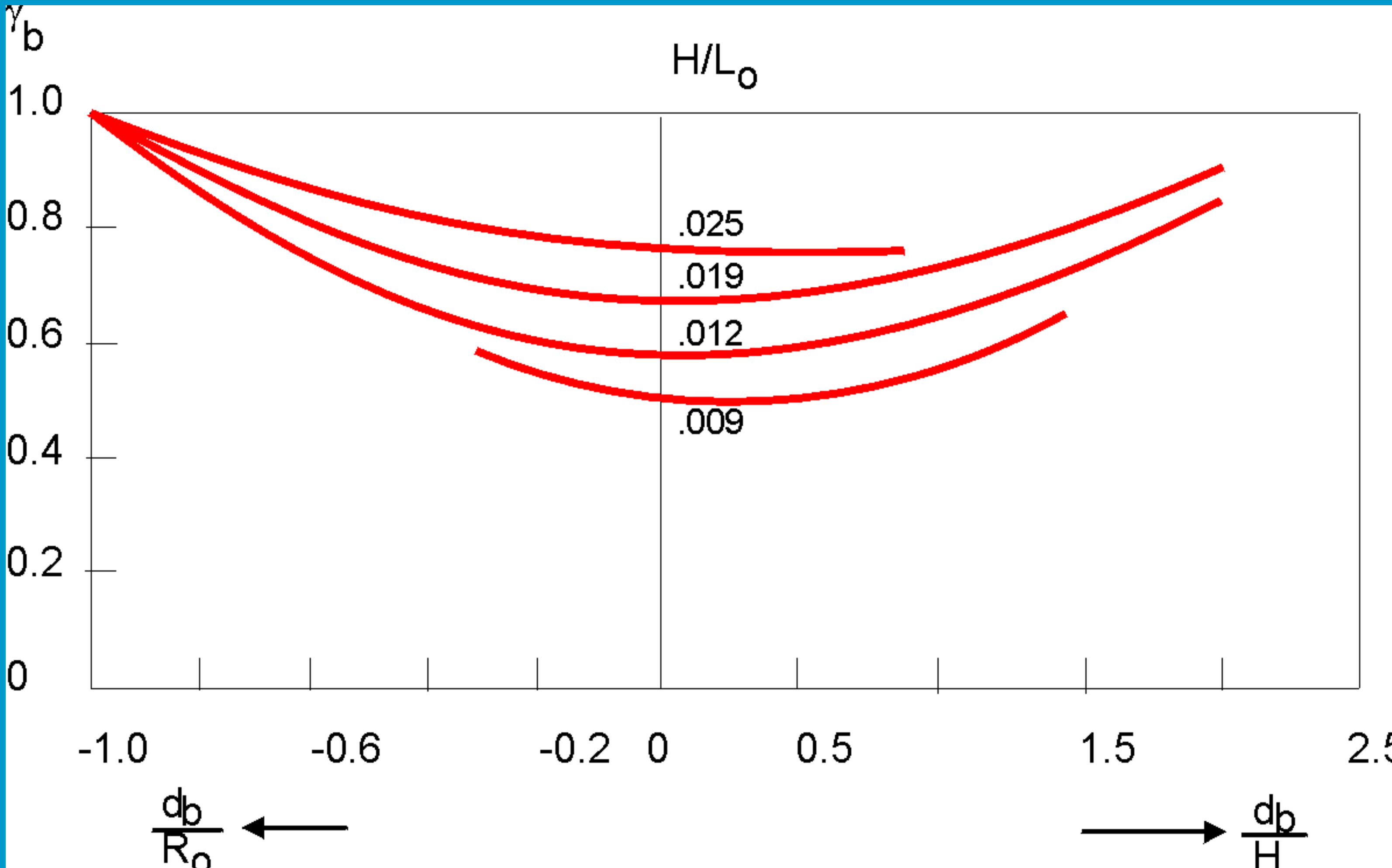


$$\gamma_B = 1 - \frac{B_B}{L_B}\left(0.5 + 0.5\cos\left[\pi\frac{d_h}{x}\right]\right)$$

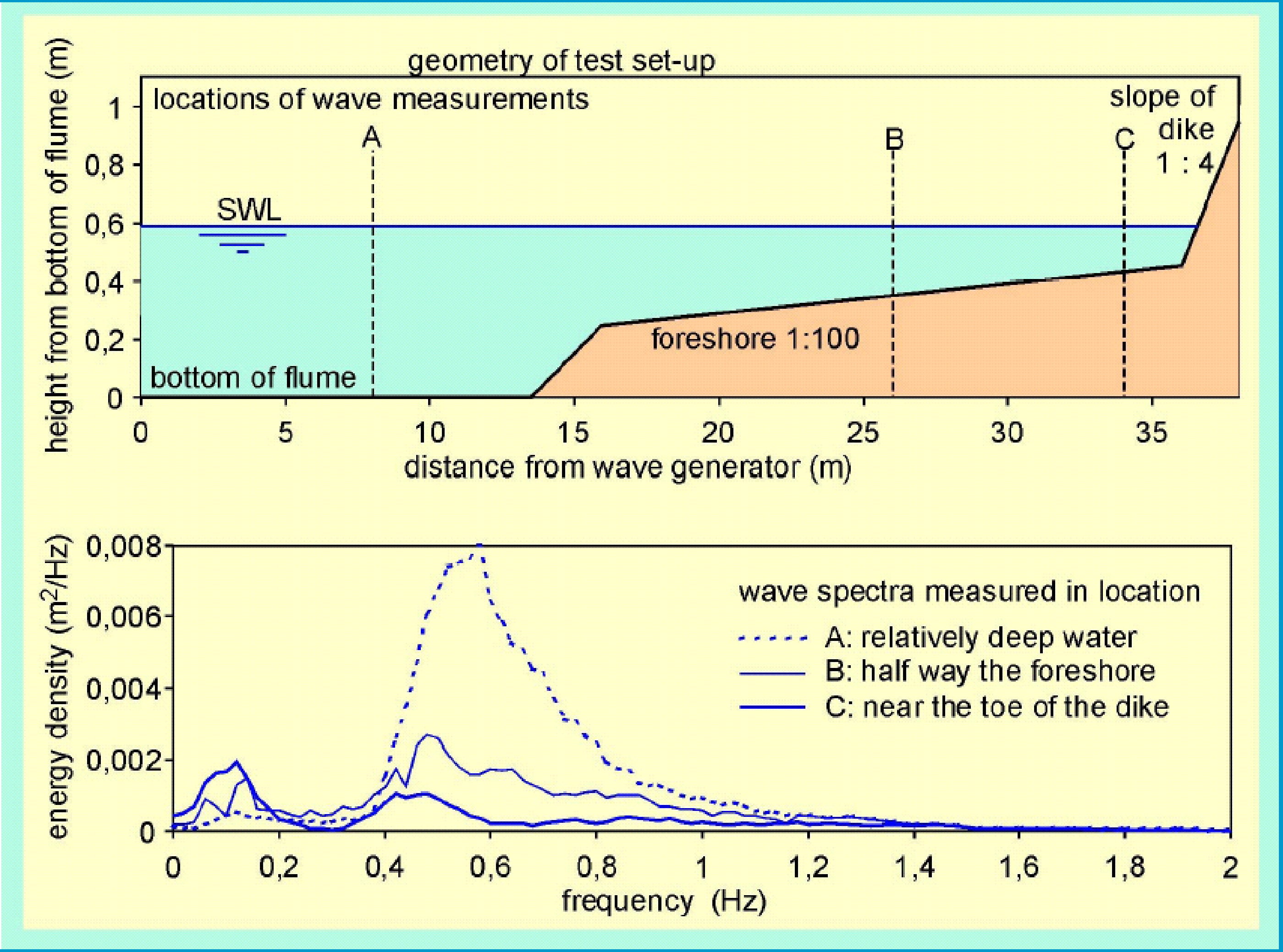
$$x = z_{2\%} \quad if\ z_{2\%} > -d_h > 0 \qquad (aboveSWL)$$

$$x = 2H_{m0}\ if\ 2H_{m0} > d_h > 0 \qquad (belowSWL)$$

berm effect (2)



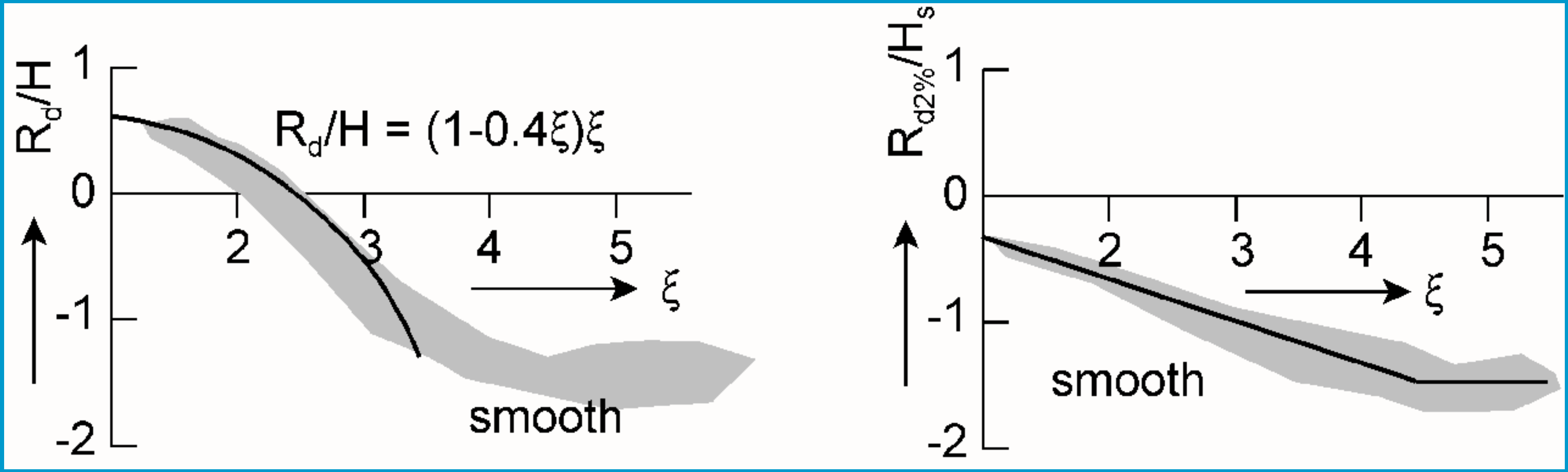
Shallow foreshore



Shallow foreshore

- Parameter $T_{m-1,0}$ is a good descriptor
- Use SwanOne to calculate $T_{m-1,0}$

run-down



$$R_d = R_u\left(1-0.4\xi\right) = H\left(1-0.4\xi\right)\xi$$

$$R_{d\,2\%} = -0.33\,H_s\,\xi_p$$

$$\left(R_{d\,2\%\,\max} = -1.5\,H_s\right)$$

Overtopping (1)

- Basically same type of equations as for run-up
- Usually wave overtopping is expressed in a discharge q
  However, this is a time-averaged discharge

$$Q = a \exp\left(b \frac{R}{\gamma} \right)$$

$$for\ breaking\ (plunging) \quad a = 0.067,\ b = 4.75,\ \sigma_b = 0.5$$

$$for\ non-breaking \quad a = 0.2, \quad b = 2.6, \quad \sigma_b = 0.35$$

Q is dimensionless overtopping
R is dimensionless freeboard

Overtopping (2)

$$Q = \frac{q}{\sqrt{gH_s^3}} \sqrt{\frac{h/L_0}{\tan\alpha}}$$

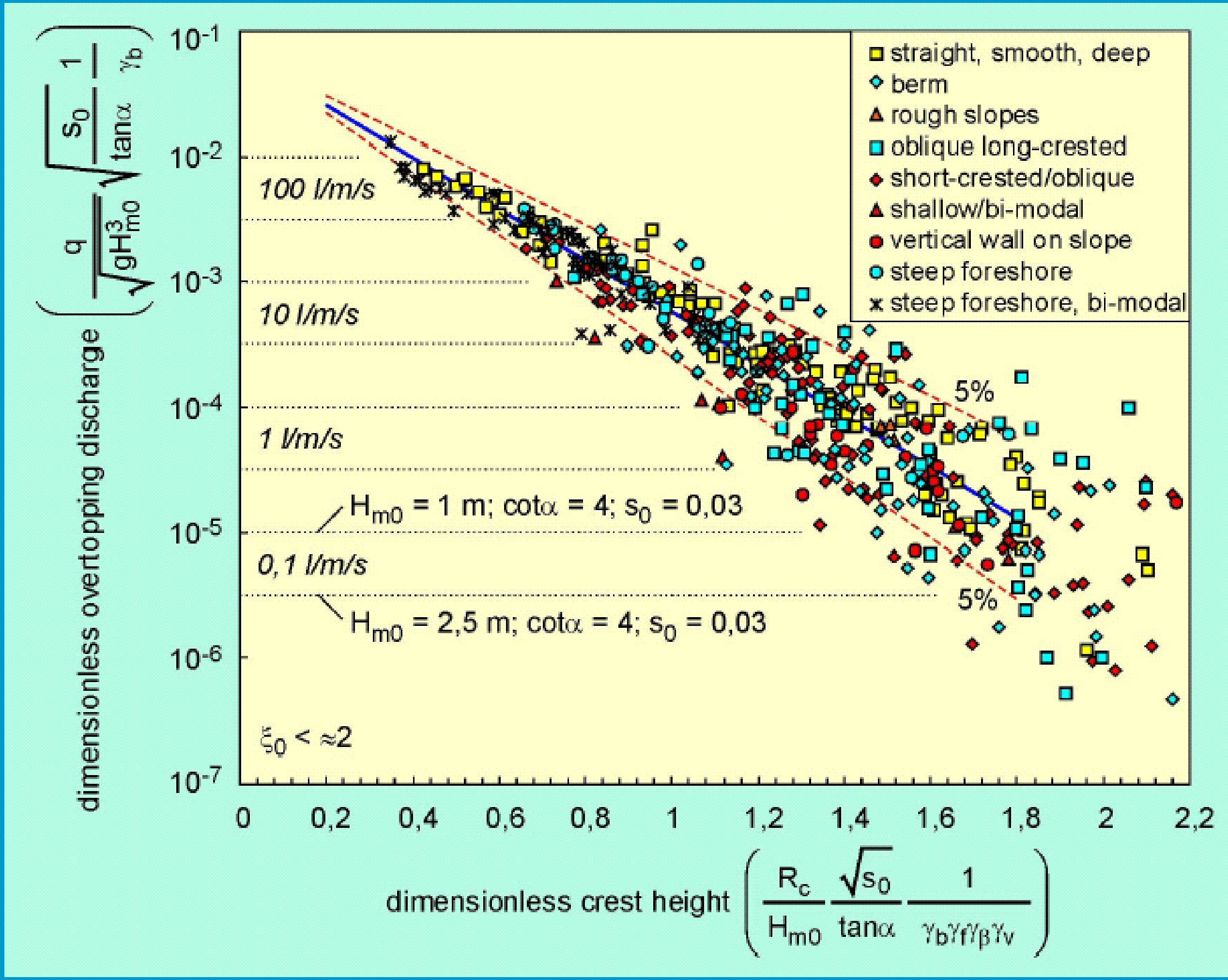
In case of non-breaking, this root should be 1

$$R = \frac{h_k}{H_s} \frac{1}{\xi}$$

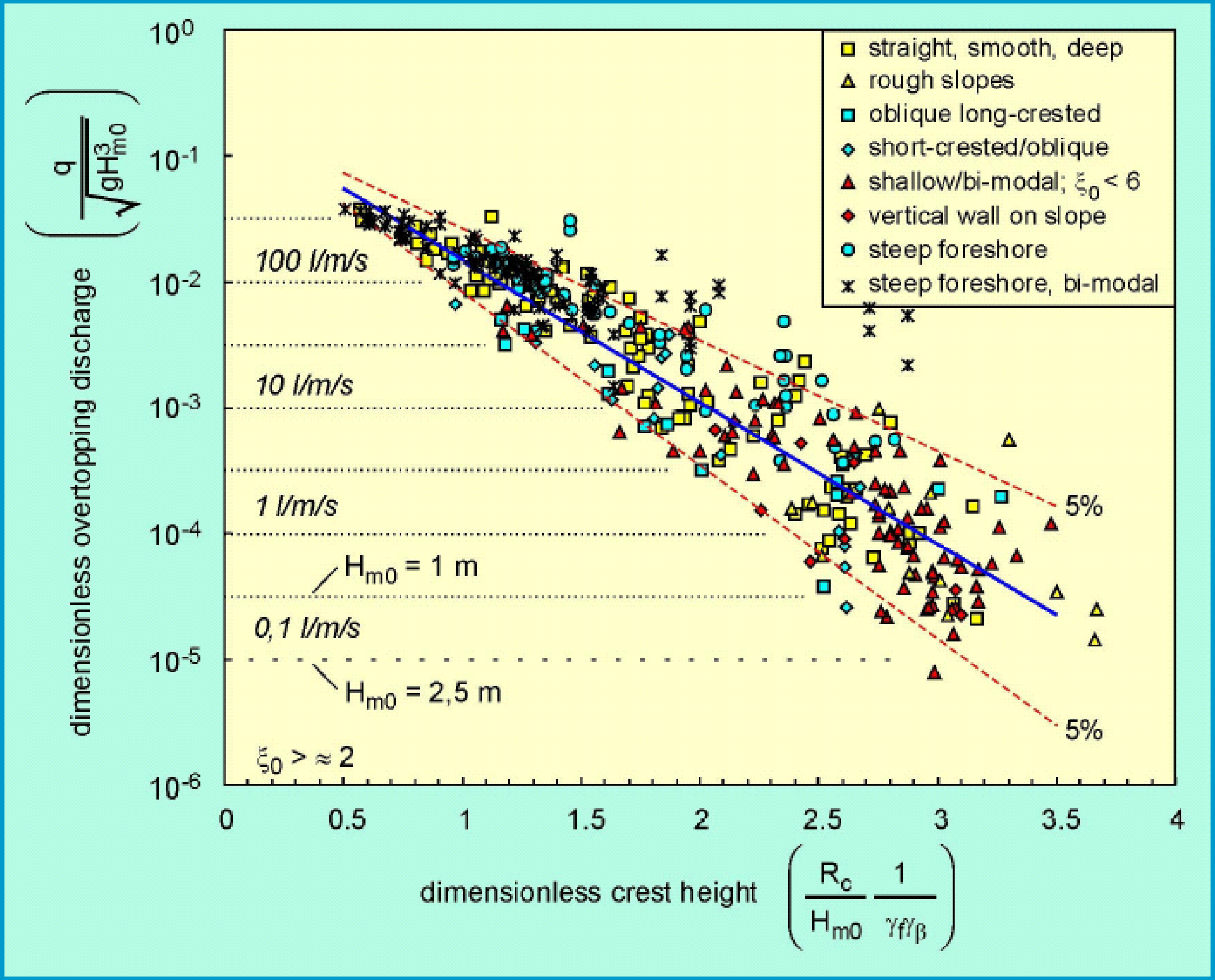
In which:

- **q = average overtopping (m^3/s per meter)**
- **h_k = crest freeboard (m)**

Measured overtopping (breaking)



Measured overtopping (non-breaking)



Overtopping (3)

- Reduction coefficients are equal to Run-up
- However:
  correction for angle of incidence:
  $\gamma_b = 1 - 0.0033\ \beta$

run demo Cress

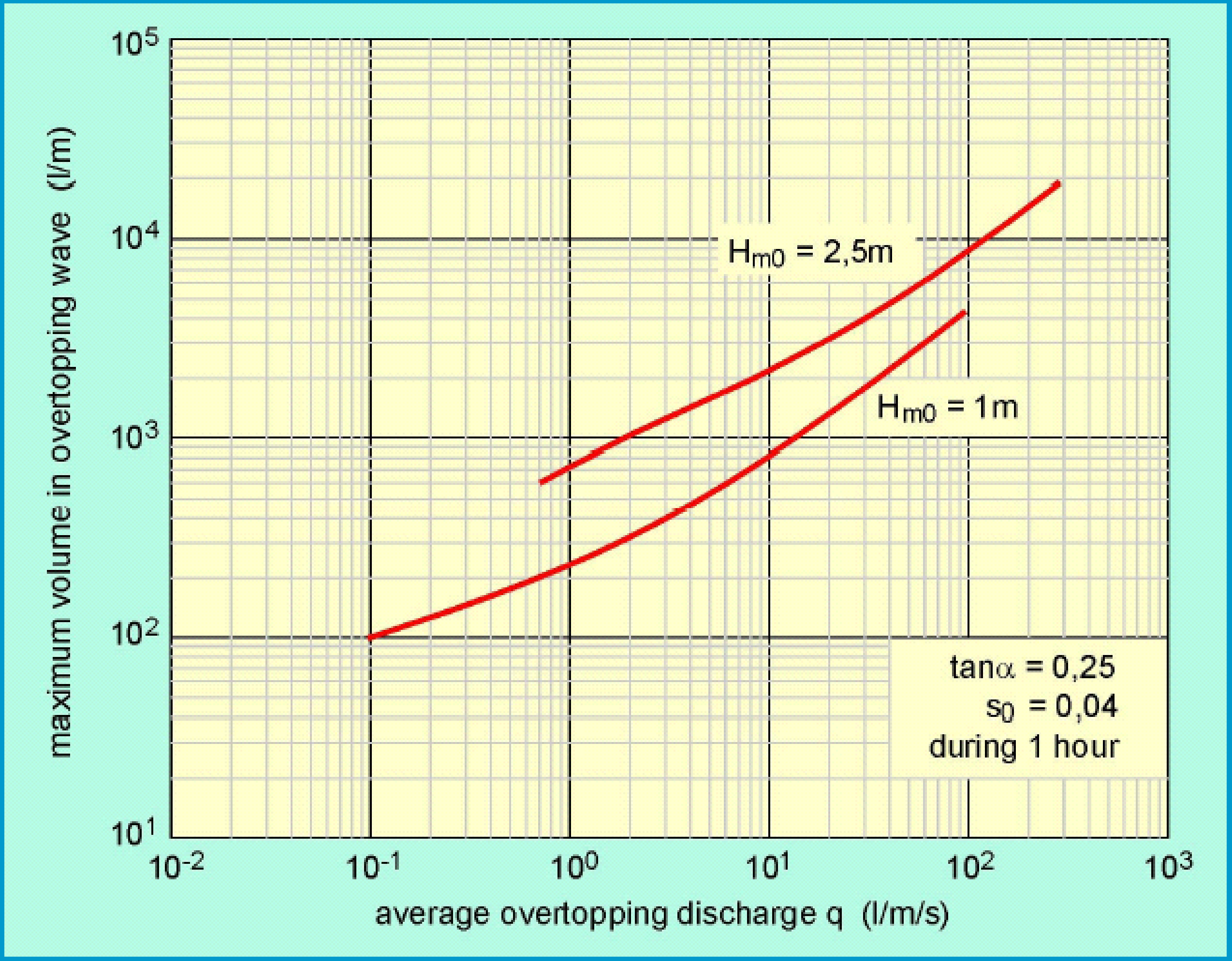
Overtopping vs. Run-up

- For design inner slope overtopping is more relevant than run-up
- In the past overtopping could not be computed
- In modern design, apply overtopping rules

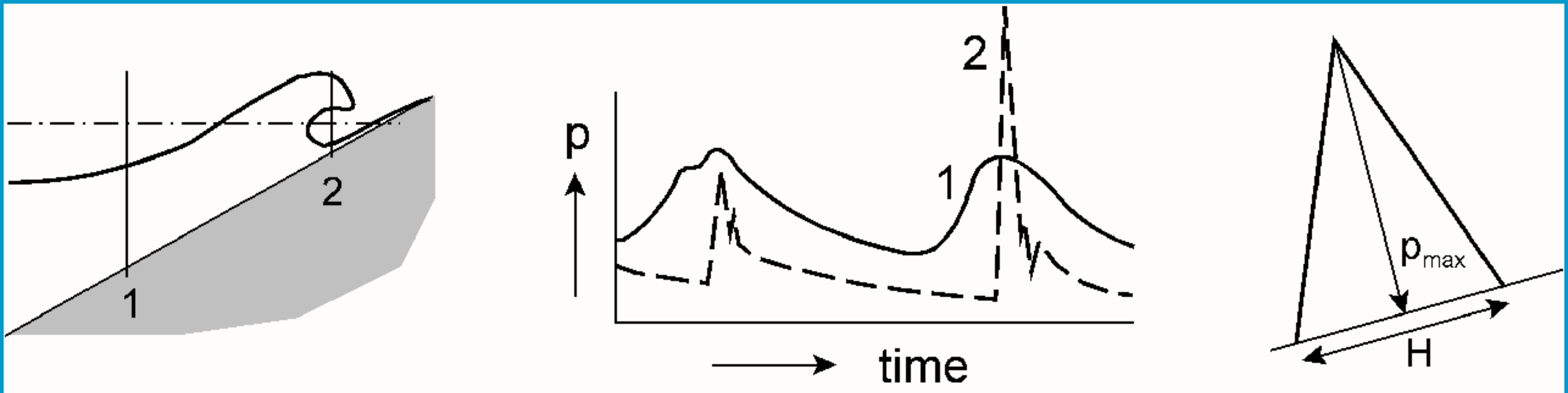
Allowable overtopping

- Dutch dikes:
  - any slope q < 0.1 l/s
  - normal slope q < 1.0 l/s
  - high quality slope q < 10 l/s
- For breakwaters much higher values can be applied
- For safe passages of cars q< 0.001 l/s
- For safe passage of pedestrians q< .005 l/s
- For no damage to buildings q< .001 l/s
- For acceptable damage to buildings q< .02 l/s

Overtopping per wave



wave impacts



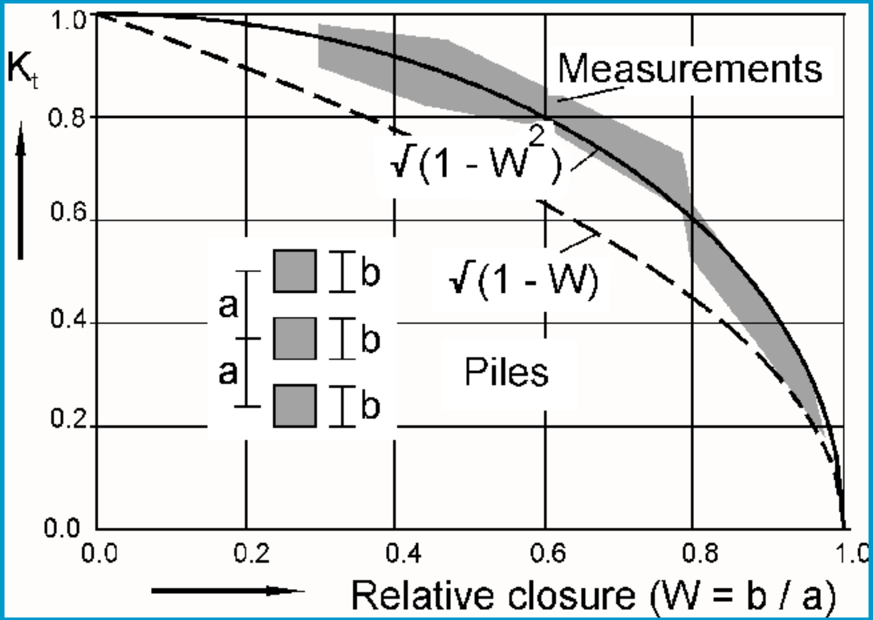
$$p_{\max 50\%} \approx 8\,\rho_w\, g\, H_s \tan\alpha$$

$$p_{\max 0.1\%} \approx 16\,\rho_w\, g\, H_s \tan\alpha$$

load reduction - pile screens

$$\frac{1}{8}\rho g H_I^2 = \frac{1}{8}\rho g H_T^2 + \frac{1}{8}\rho g H_R^2 + \text{absorption}$$

$$K_T = \sqrt{F_T / F_I} = H_T / H_I \approx (H_T)_{1/3} / (H_I)_{1/3}$$

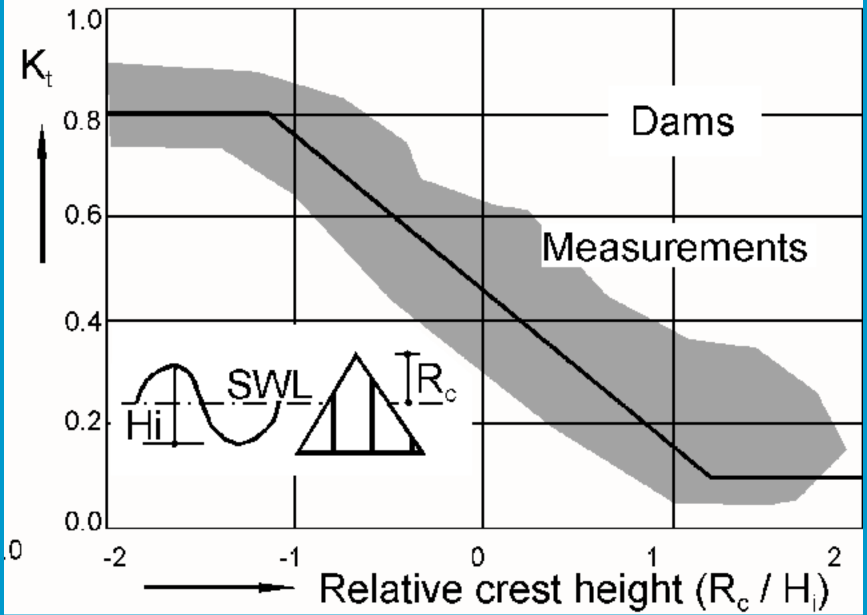


$$H_T = \sqrt{(1-W)\,H_I^2}$$

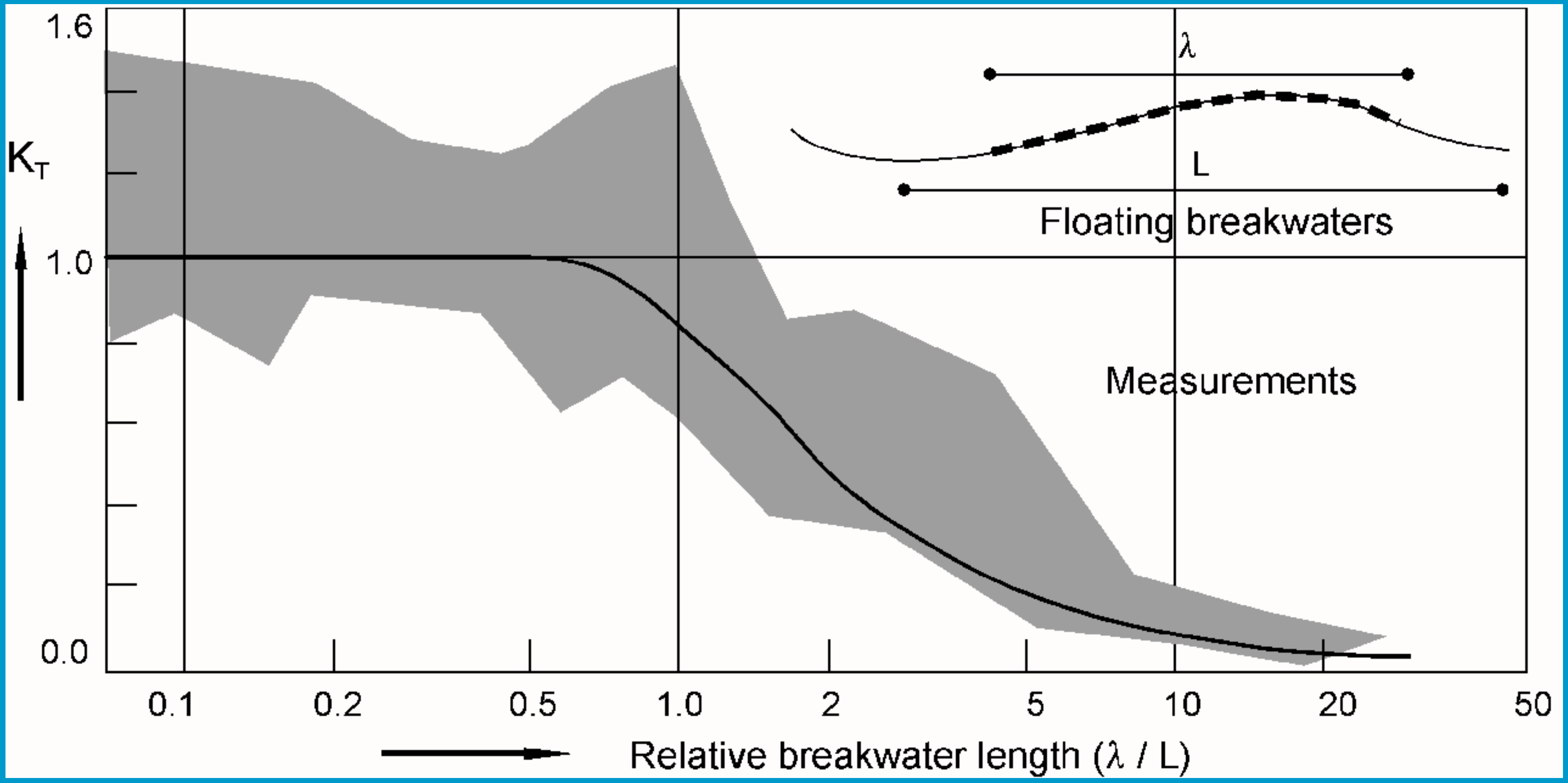
$$\rightarrow \quad K_T = \sqrt{(1-W)}$$

$$K_T = \sqrt{1-W^2}$$

wave transmission over low breakwaters



floating breakwaters



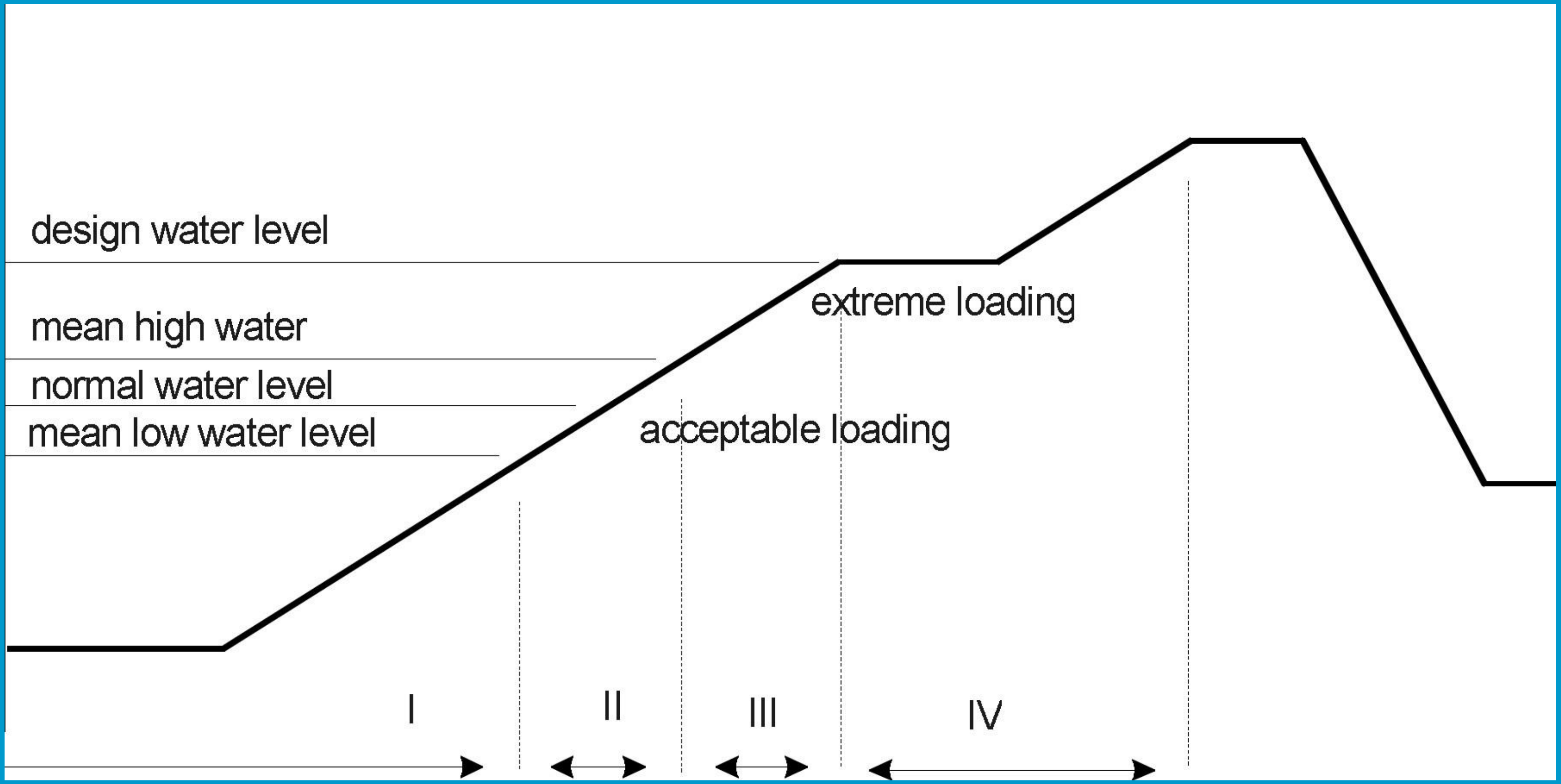
Example dike height determination

- Dike height is:
  - design water level
  - + freeboard
- Design water level is:
  - astronomical tide (high water)
  - + storm surge (wind set-up)
  - + margin for seiches and gust bumps
  - + surcharge for
    - sea level rise
    - increase of tidal amplitude
  - + settlement of subsoil

for text of examples, see “blackboard”

- freeboard is:
  - 2% Run-up height or
  - height determined by critical overtopping (e.g. 1 l/s) but
  - at least 50 cm

Dike height determination (2)



Sea dike design (input)

Design a seadike for 1/100 and 1/1000, using Hook of Holland boundary conditions.

	1/100	1/1000
Waterlevel	3.5 + MSL	4.25 + MSL
Wind speed	22 m/s	25 m/s
Wave height	6.5 m	7.5 m

Depth in front of dike: 5 m below MSL
Slope of dike 1:4
Berm at SWL, berm width 10 m

Design life is 50 years

Sea dike design (levels)

Sea level rise now 20 cm/century
Rise tidal amplitude 10 cm
So, (20+10)/2 = 15 cm surcharge

And what to do with accelerated sea level rise (climate change) ??

Summary table:

Frequency	1/100	1/1000
Design level	3.50 m+MSL	4.25 m+MSL
Gust bump	0.50 m	0.50 m
Seiches	0.00 m	0.00 m
Sealevel rise	0.15 m	0.15 m
Design water level	4.15 m+MSL	4.90 m +MSL

Sea dike design (waves)

Design wave height is depth limited, in this situation a breaker index $\gamma = 0.5$ may be used

Frequency	1/100	1/1000
Water depth	5+4.15 = 9.15 m	5+4.90 = 9.90 m
Design wave	4.45 m	4.95 m

Sea dike design (freeboard)

Frequency	1/100	1/1000
Total run-up	6.92 m	8.28 m
Freeboard (q= 1 l/s)	7.88 m	8.54 m

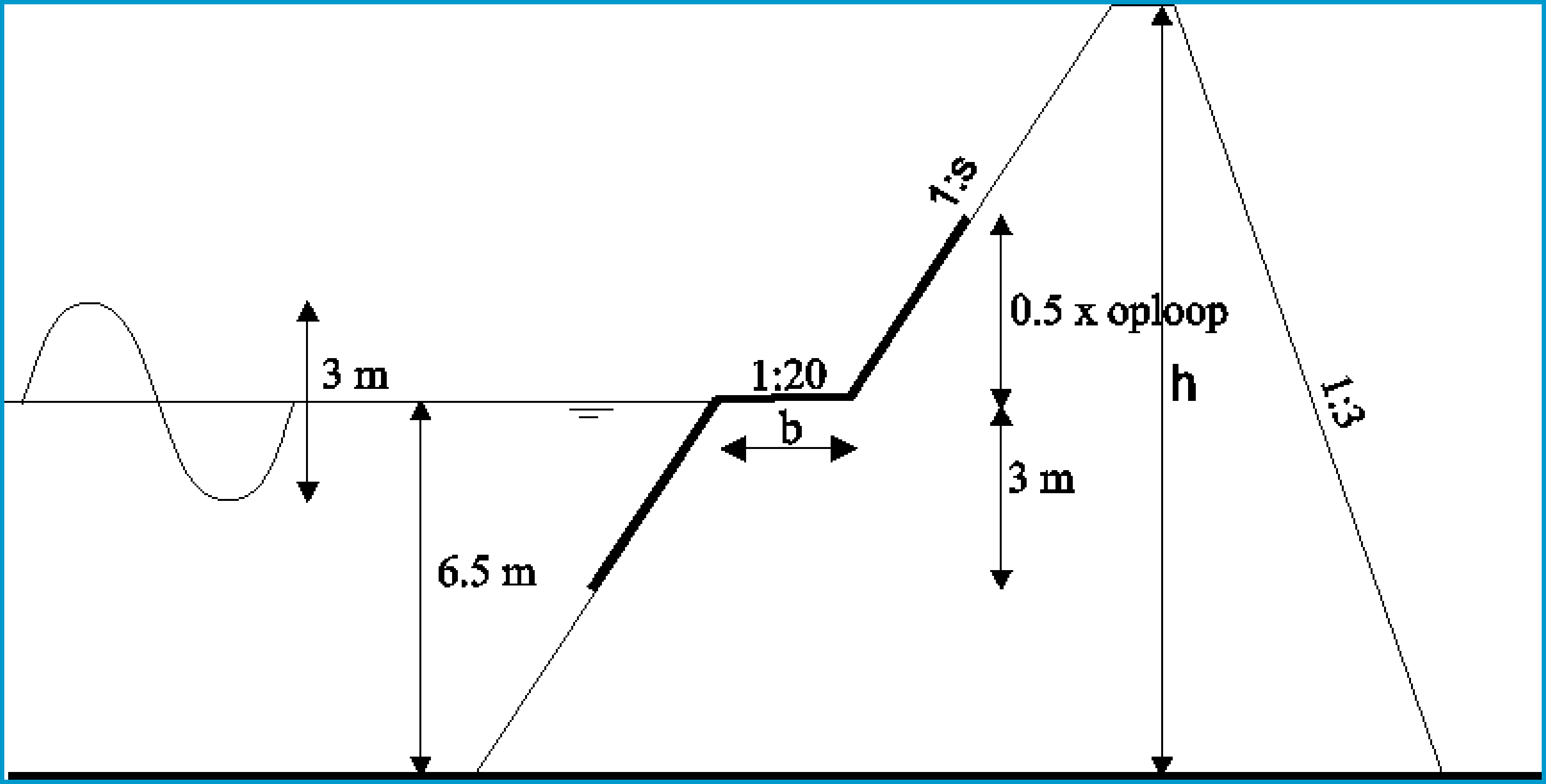
Sea dike design (crest level)

Design crest level is
design water level + freeboard + settlement

assume good quality subsoil, 50 cm settlement during design life

frequency	1/100	1/1000
Ultimate crest level Above MSL; using run-up	4.15+6.92=11.10 m	4.90+8.28=13.20 m
Construction crest level, above MSL	11.6 m	13.7 m

berm and slope optimization (1)



berm and slope optimization (2)

Given: wave height 3 m
wave period 8 seconds
SWL = berm height
berm at 1:20
slope with Basalton (1:3. 1:5) or grass (1:8)
depth of dike below SWL = 6.5 m
crest width = 2 m
inner slope 1:2

berm and slope optimization (3)

Slope	1 : 3			1 : 5			1 : 8		
Overtopping Berm width	0.1 l/m/s	1.0 l/m/s	10 l/m/s	0.1 l/m/s	1.0 l/m/s	10 l/m/s	0.1 l/m/s	1.0 l/m/s	10 l/m/s
0 m	17.9	15.4	12.9	13.2	11.7	10.2	10.4	9.5	8.6
5 m	15.7	13.7	11.7	12.1	11.1	9.8	10.2	9.3	8.5
10 m	14.3	12.6	10.8	11.9	10.7	9.5	10.0	9.2	8.4
15 m	13.4	11.9	10.4	11.4	10.3	9.2	9.8	9.0	8.3

required crest level for a given berm, a given slope and a given discharge

berm and slope optimization (4)

Slope	1 : 3			1 : 5			1 : 8		
Overtopping Berm width	0,1 l/m/s	1,0 l/m/s	10 l/m/s	0,1 l/m/s	1,0 l/m/s	10 l/m/s	0,1 l/m/s	1,0 l/m/s	10 l/m/s
0 m	997 €14955	742 € 11134	525 € 8750	723 € 10850	571 € 8564	437 € 6548	616 € 9235	515 € 7731	424 € 6360
5 m	803 € 12051	623 € 9345	467 € 6999	642 € 9635	548 € 8213	436 € 6544	625 € 9377	527 € 7902	447 € 6703
10 m	707 € 10606	567 € 8497	437 € 6548	655 € 9829	544 € 8165	445 € 6675	635 € 9525	549 € 8234	470 € 7048
15 m	663 € 9945	546 € 8192	443 € 6642	640 € 9602	543 € 8136	455 € 6817	645 € 9680	561 € 8415	493 € 7395

volume of the dike, using simple geometry; earth moving cost calculated using €15/m^3

berm and slope optimization (5)

Slope	1 : 3			1 : 5			1 : 8		
Overtopping Berm width	0,1 l/m/s	1,0 l/m/s	10 l/m/s	0,1 l/m/s	1,0 l/m/s	10 l/m/s	0,1 l/m/s	1,0 l/m/s	10 l/m/s
0 m	109 € 820	94 € 708	79 € 596	108 € 807	96 € 717	84 € 627	116 € 873	107 € 799	97 € 724
5 m	101 € 759	89 € 669	77 € 579	104 € 778	96 € 718	85 € 640	119 € 894	109 € 820	101 € 754
10 m	97 € 734	88 € 657	77 € 576	107 € 804	98 € 732	88 € 660	122 € 915	113 € 849	104 € 783
15 m	97 € 730	88 € 663	79 € 596	108 € 812	99 € 746	91 € 680	125 € 936	116 € 870	108 € 812

purchase of land needed for the construction of the dike, values in m^2; cost using € 7.50/ m^2

berm and slope optimization (6)

Slope Berm width	1 : 3	1 : 5	1 : 8
0 m	8.3 m 22.6 m2 (€ 2713)	5.0 m 28.0 m2 (€ 2244)	3.0 m 0 m2 (€ 250)
5 m	6.7 m 19.8 m2 (€ 2630)	4.4 m 26.3 m2 (€ 2351)	2.8 m 0 m2 (€ 250)
10 m	5.6 m 17.8 m2 (€ 2641)	4.0 m 25.0 m2 (€ 2500)	2.6 m 0 m2 (€ 250)
15 m	5.0 m 16.6 m2 (€ 2747)	3.6 m 23.7 m2 (€ 2648)	2.5 m 0 m2 (€ 250)

cost of the revetment

1:3 € 120/m²

1:5 € 80/m²

1:8 no cost (grass)

cost of the berm

€ 50/m²

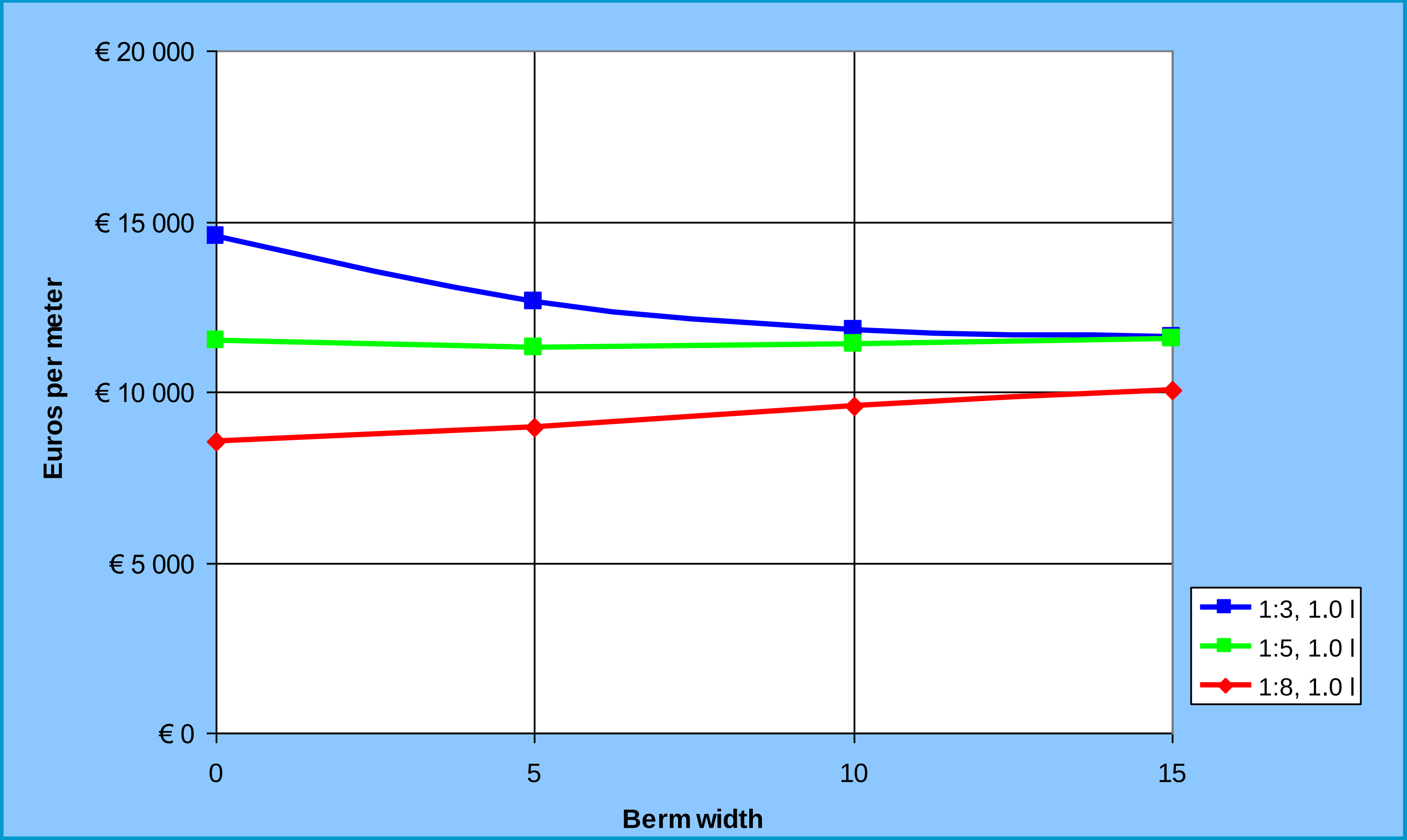
berm and slope optimization (7)

Slope	1 : 3			1 : 5			1 : 8		
Overtopping Berm width	0.1 l/m/s	1.0 l/m/s	10 l/m/s	0.1 l/m/s	1.0 l/m/s	10 l/m/s	0.1 l/m/s	1.0 l/m/s	10 l/m/s
0 m	€18,489 (100%)	€14,555 (79%)	€11,184 (60%)	€13,901 (75%)	€11,525 (62%)	€ 9,419 (51%)	€10,108 (55%)	€8,529 (46%)	€7,084 (38%)
5 m	€15,439 (84%)	€12,643 (68%)	€10,207 (55%)	€12,765 (69%)	€11,283 (61%)	€9,536 (52%)	€10,521 (57%)	€8,972 (49%)	€7,707 (42%)
10 m	€13,980 (76%)	€11,795 (64%)	€9,765 (53%)	€13,132 (71%)	€11,397 (62%)	€9,835 (53%)	€10,940 (59%)	€9,583 (52%)	€8,331 (45%)
15 m	€13,422 (73%)	€11,602 (63%)	€9,984 (54%)	€13,062 (71%)	€11,530 (62%)	€10,144 (55%)	€11,366 (61%)	€10,035 (54%)	€8,957 (48%)

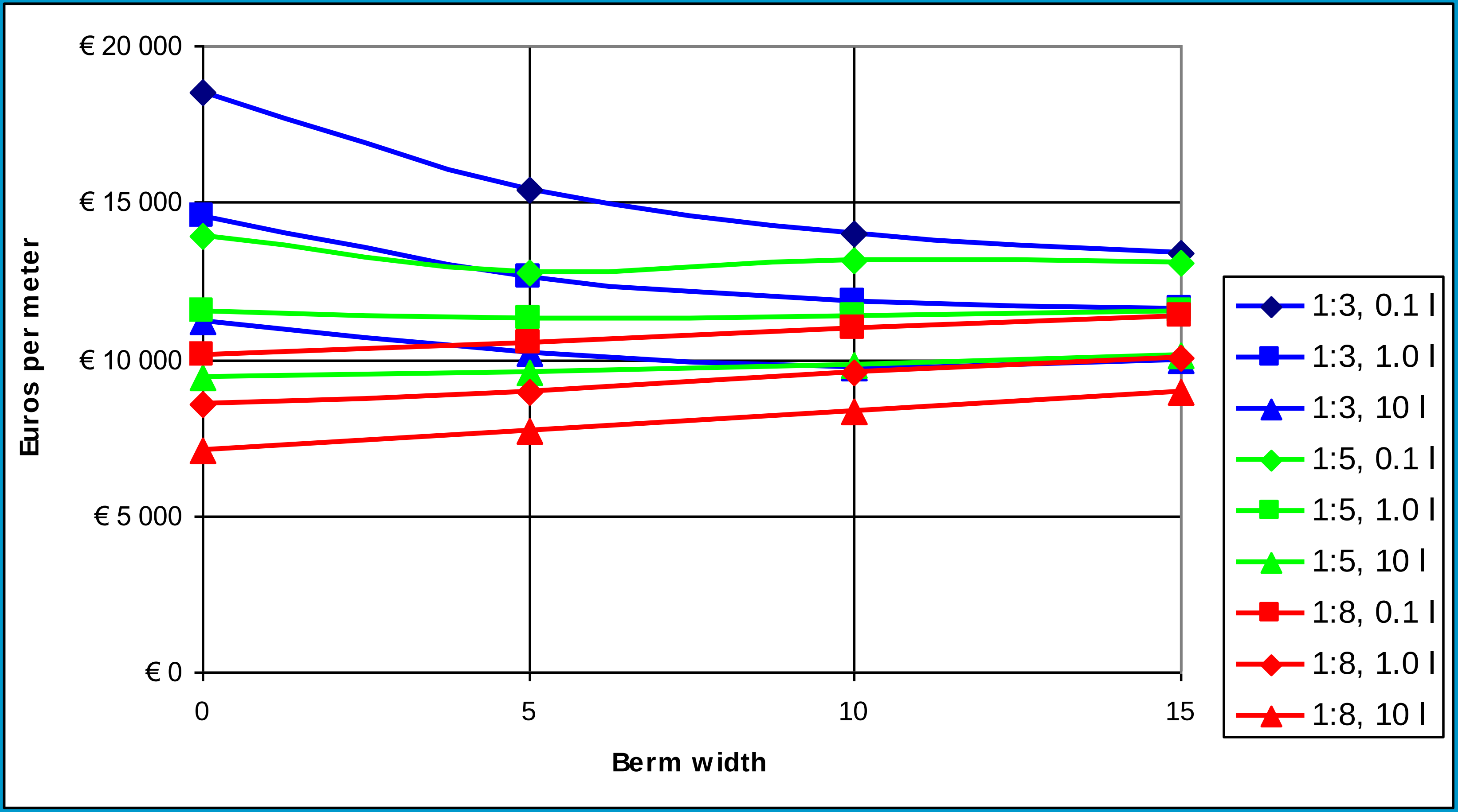
summary of all the costs

realise that for 10 l/s the cost of the inner slope will be higher than for a 0.1 l/s slope !!

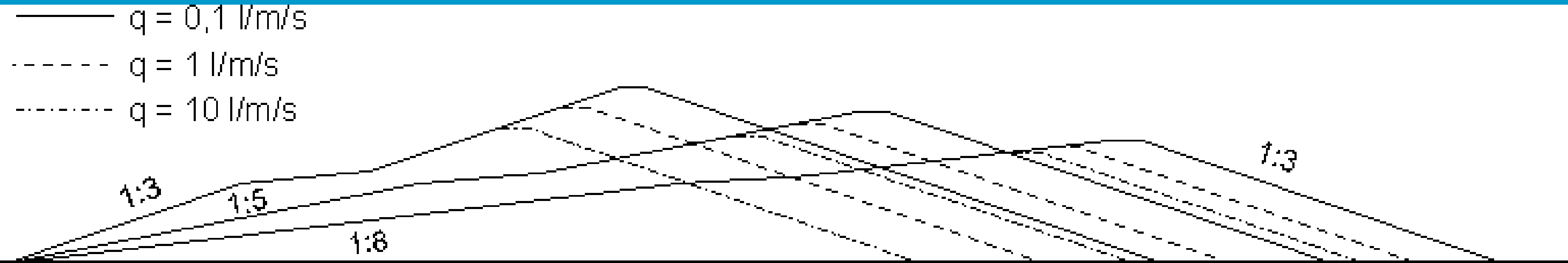
berm and slope optimization (8)



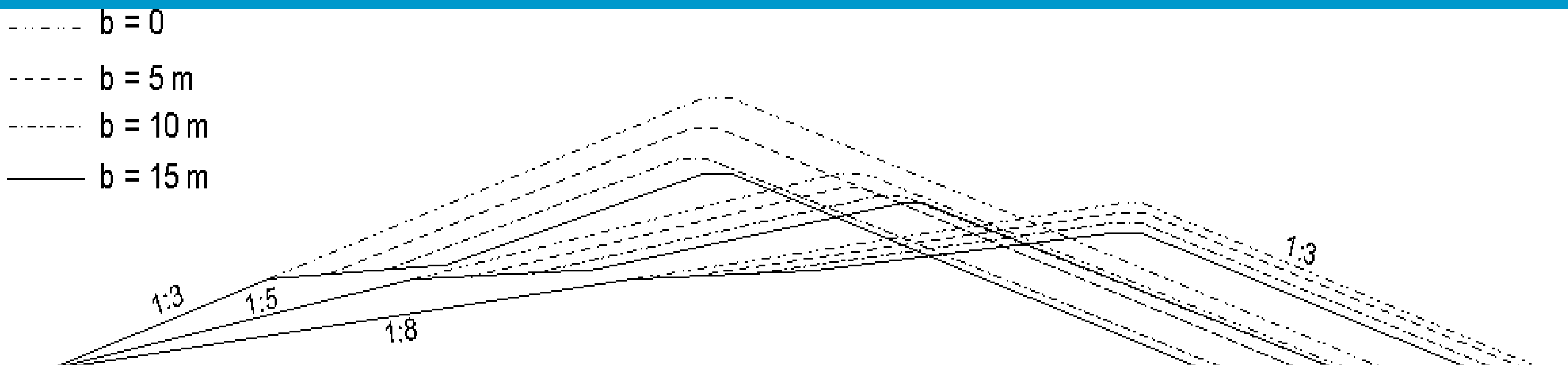
berm and slope optimization (9)



berm and slope optimization (8)



given berm width 10 m



given discharge 0.1 l/s

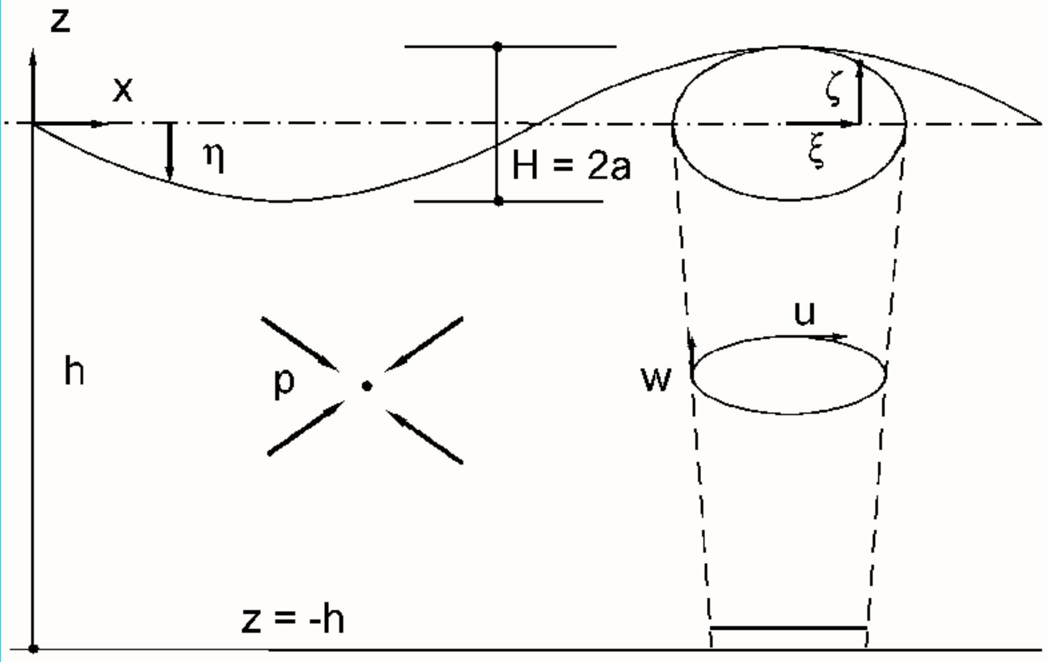
Simulator for highschools

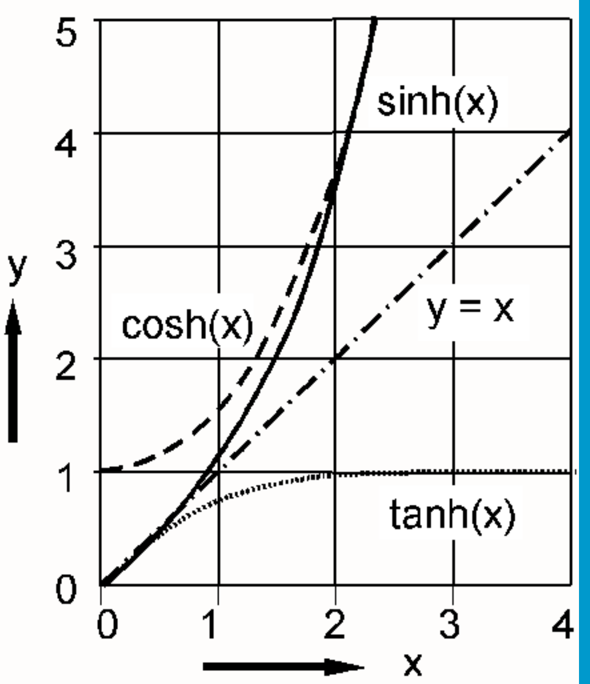
- user interface
- spreadsheet

recapitulation of short wave theory

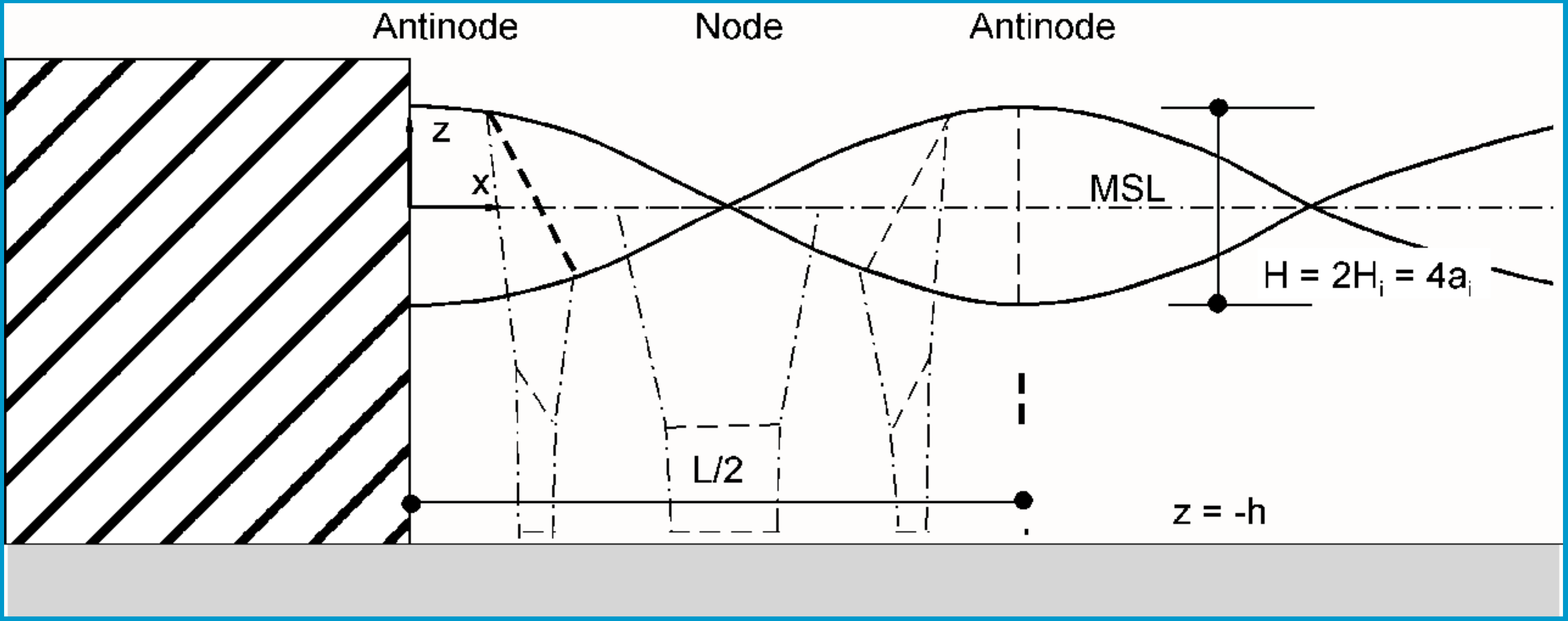
June 3, 2012

definitions and behaviour of hyperbolic functions





standing wave



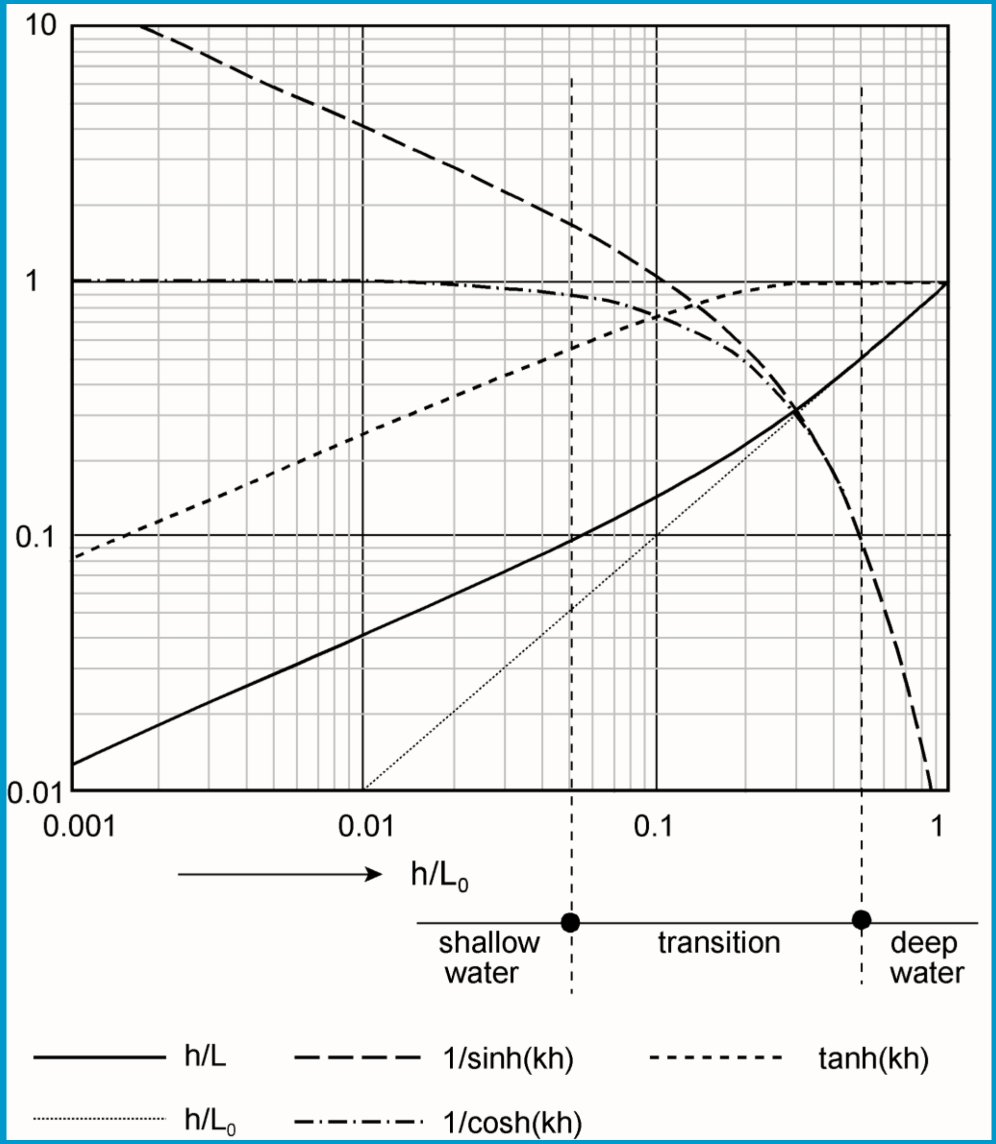
linear wave theory

basic equations

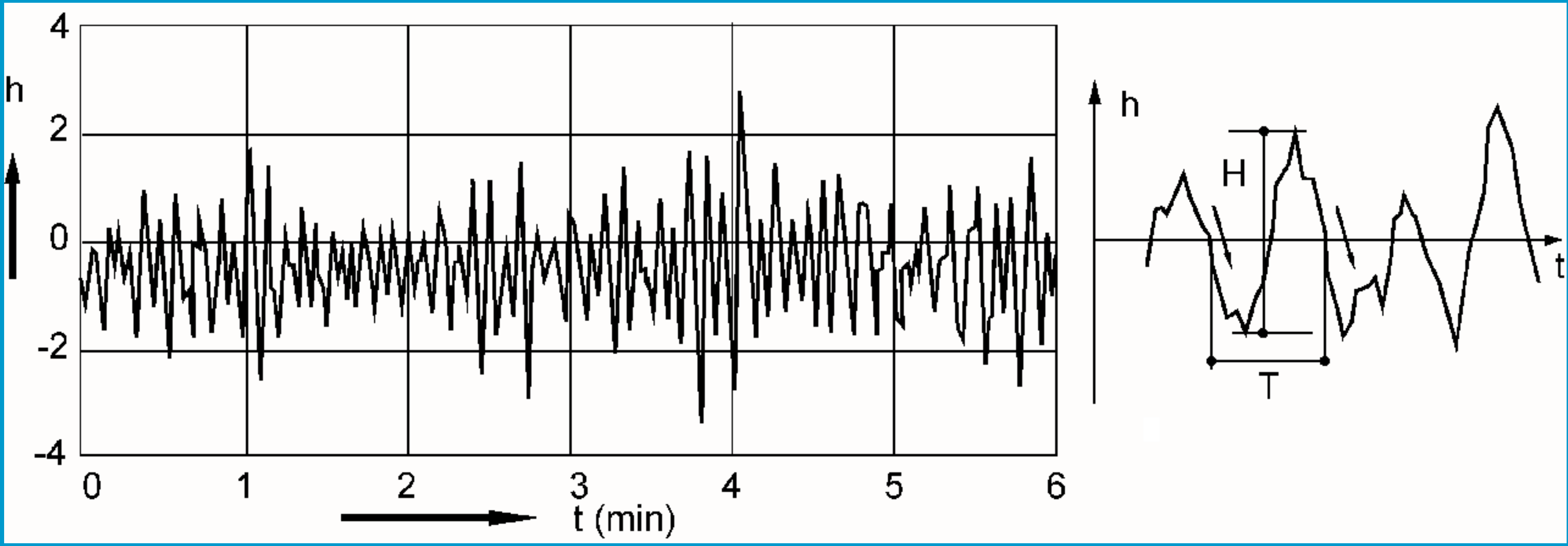
Relative depth	Shallow Water $\frac{h}{L} < \frac{1}{20}$	Transitional water depth $\frac{1}{20} < \frac{h}{L} < \frac{1}{2}$	Deep Water $\frac{h}{L} > \frac{1}{2}$
Wave Celerity	$c = \frac{L}{T} = \sqrt{gh}$	$c = \frac{L}{T} = \frac{gT}{2\pi}\tanh kh$	$c = c_0 = \frac{L}{T} = \frac{gT}{2\pi}$
Wave Length	$L = T\sqrt{gh}$	$L = \frac{gT^2}{2\pi}\tanh kh$	$L = L_0 = \frac{gT^2}{2\pi}$
Group Velocity	$c_g = c = \sqrt{gh}$	$c_g = nc = \frac{1}{2}\left(1 + \frac{2kh}{\sinh 2kh}\right)c$	$c_g = \frac{1}{2}c_0 = \frac{gT}{4\pi}$
Energy Flux (per m width)	$F = Ec_g = \frac{1}{2}\rho g a^2\sqrt{gh}$	$F = Ec_g = \frac{1}{2}\rho g a^2 nc$	$F = \frac{T}{8\pi}\rho g^2 a^2$
Particle velocity Horizontal Vertical	$u = a\sqrt{\frac{g}{h}}\sin\theta$ $w = \omega a\left(1 + \frac{z}{h}\right)\cos\theta$	$u = \omega a\frac{\cosh k(h+z)}{\sinh kh}\sin\theta$ $w = \omega a\frac{\sinh k(h+z)}{\sinh kh}\cos\theta$	$u = \omega a e^{kz}\sin\theta$ $w = \omega a e^{kz}\sin\theta$ $\xi = -a e^{kz}\cos\theta$
Particle displacement Horizontal Vertical	$\xi = -\frac{a}{\omega}\sqrt{\frac{g}{h}}\cos\theta$ $p = -\rho g z + \rho g a\sin\theta$	$\xi = -a\frac{\cosh k(h+z)}{\sinh kh}\cos\theta$ $\zeta = a\frac{\sinh k(h+z)}{\sinh kh}\sin\theta$	$\zeta = a e^{kz}\sin\theta$
Subsurface pressure		$p = -\rho g z + \rho g a\frac{\cosh k(h+z)}{\cosh kh}\sin\theta$	

$$a = \frac{H}{2} \qquad \omega = \frac{2\pi}{T} \qquad k = \frac{2\pi}{L} \qquad \theta = \omega t - kx$$

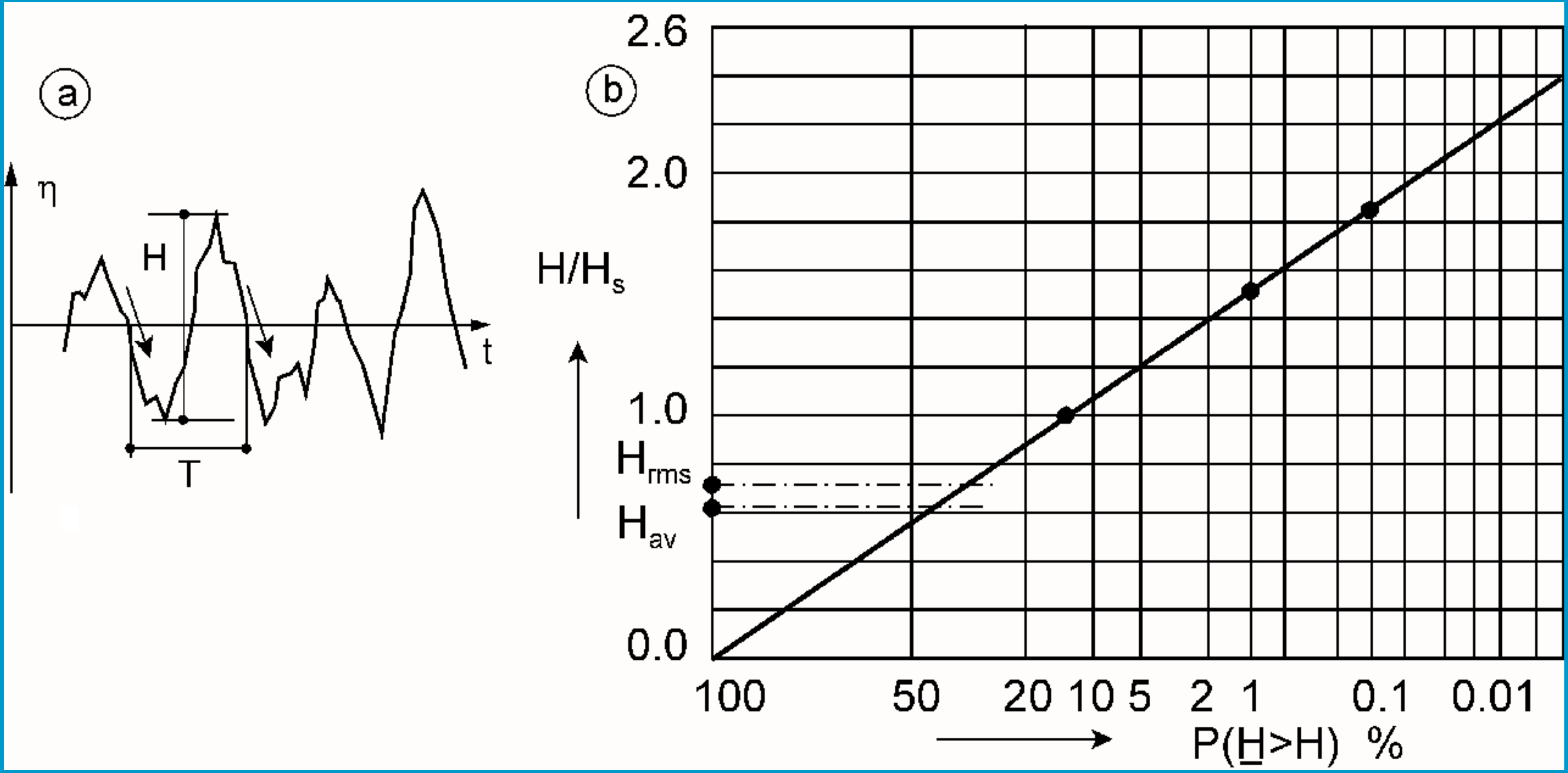
parameters in linear wave theory



definition of H and T



wave definitions and wave height distribution

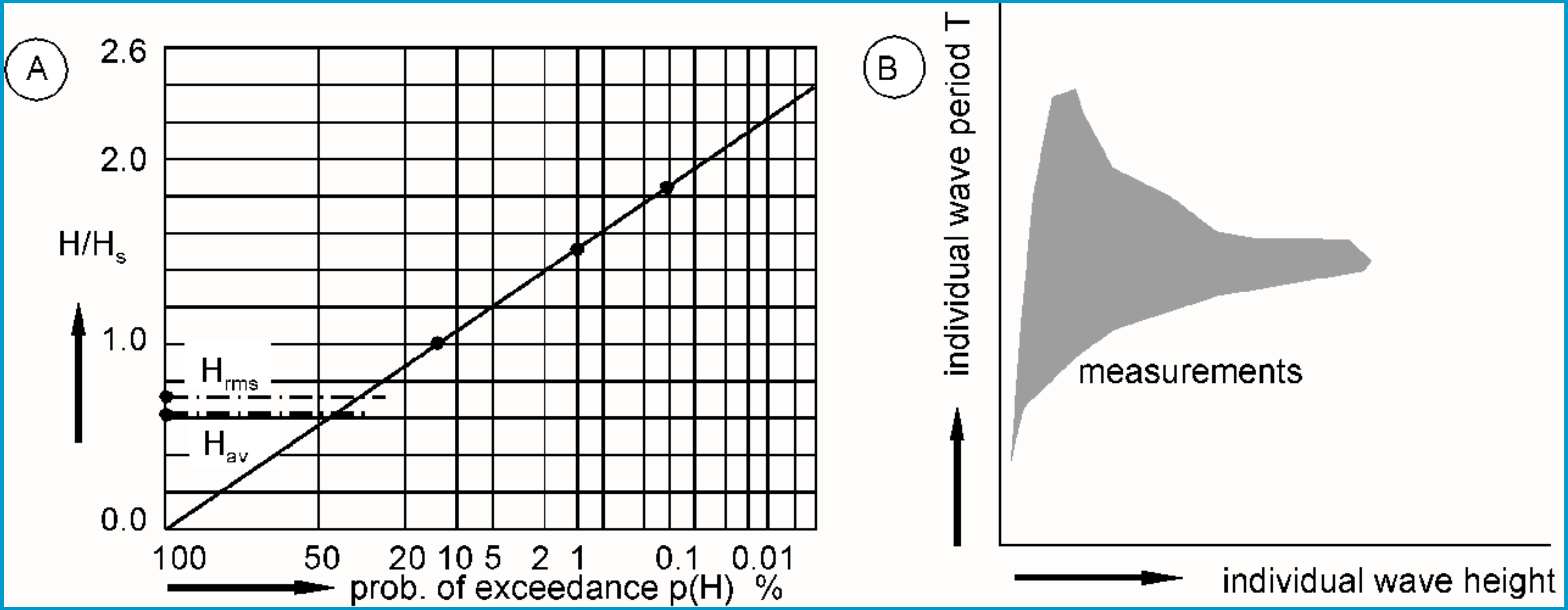


Rayleigh distribution

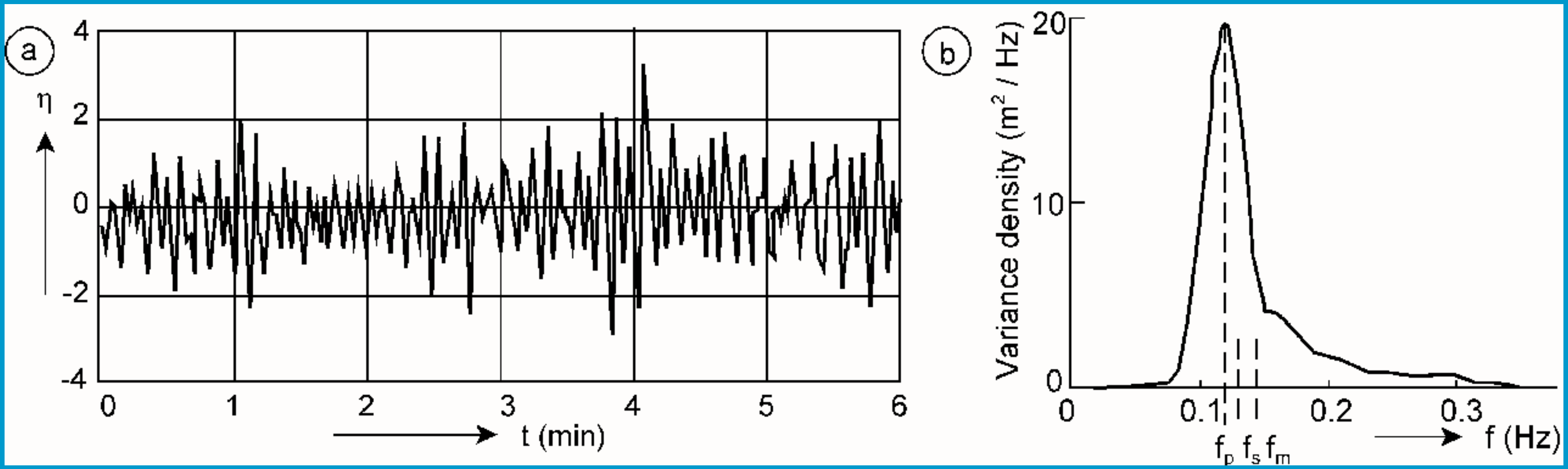
$$P\{\underline{H} > H\} = \exp\left[-\left(\frac{H}{H_{rms}}\right)^2\right] = \exp\left[-2\left(\frac{H}{H_s}\right)^2\right]$$

$$H_s \equiv H_{visual} \equiv H_{1/3} \equiv H_{13.5\%} \equiv H_{m0} \approx 4\sqrt{m_0}$$

wave height and wave period



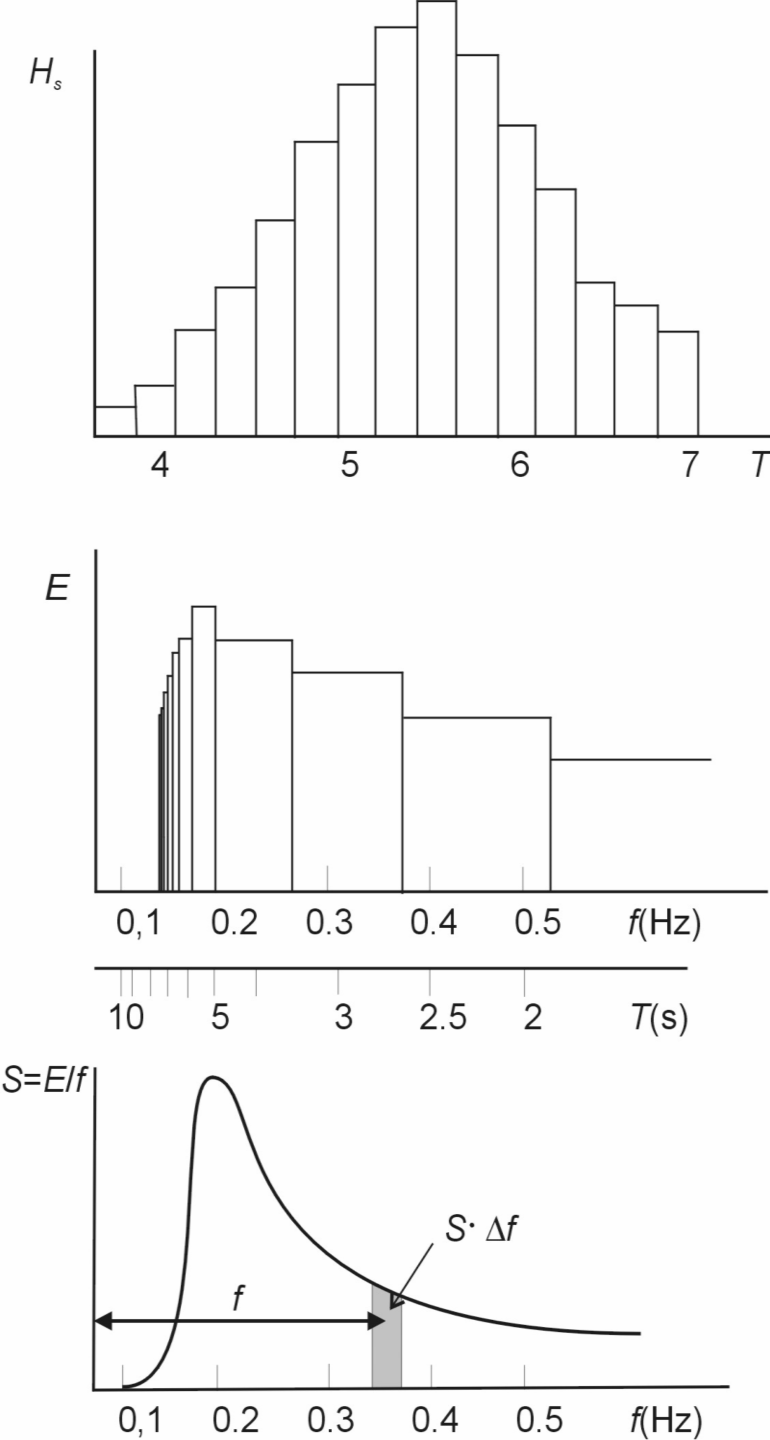
wave registration in the North Sea



Spectral moments: m_0 = surface of energy density spectrum

m_{-1} = first negative moment of spectrum

$T_{m-1,0} = m_{-1}/m_0$ = spectral wave period $\approx 0.9\ T_p$

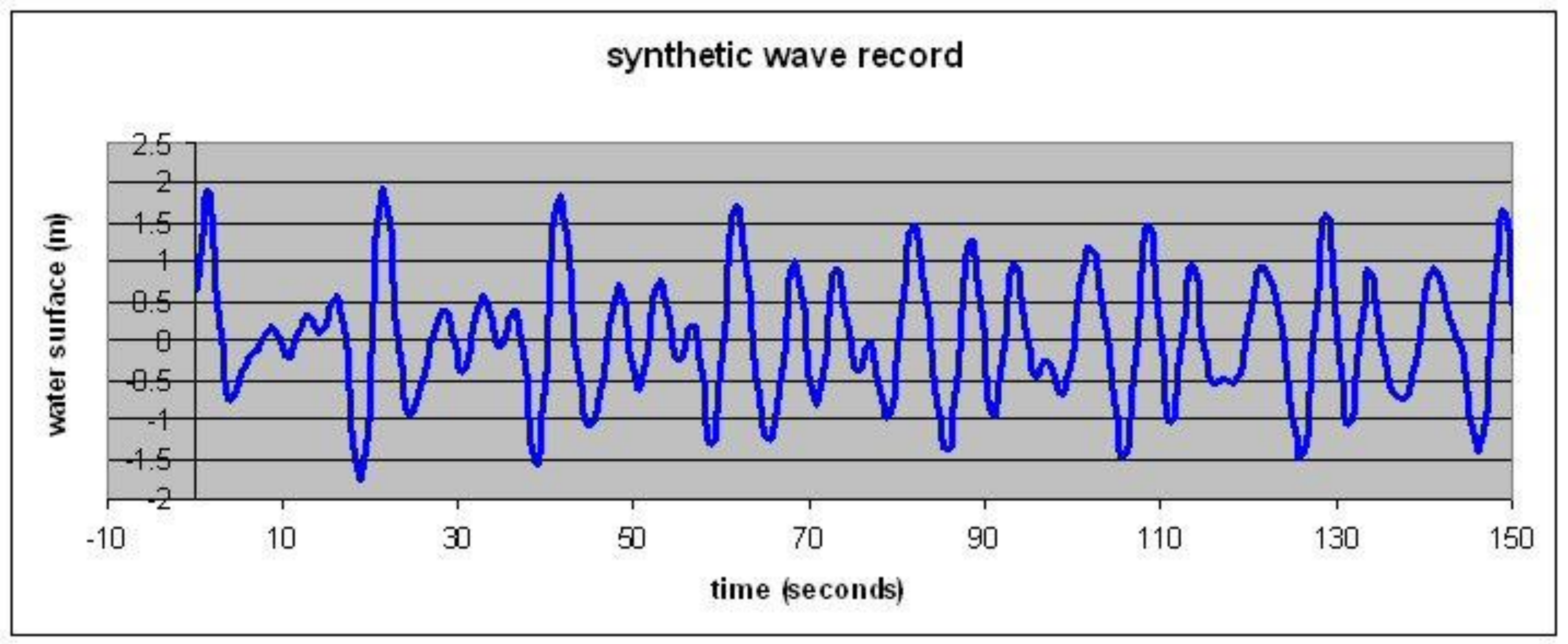


Spectra

$$m_n = \int_0^\infty f^n S(f)\,df$$

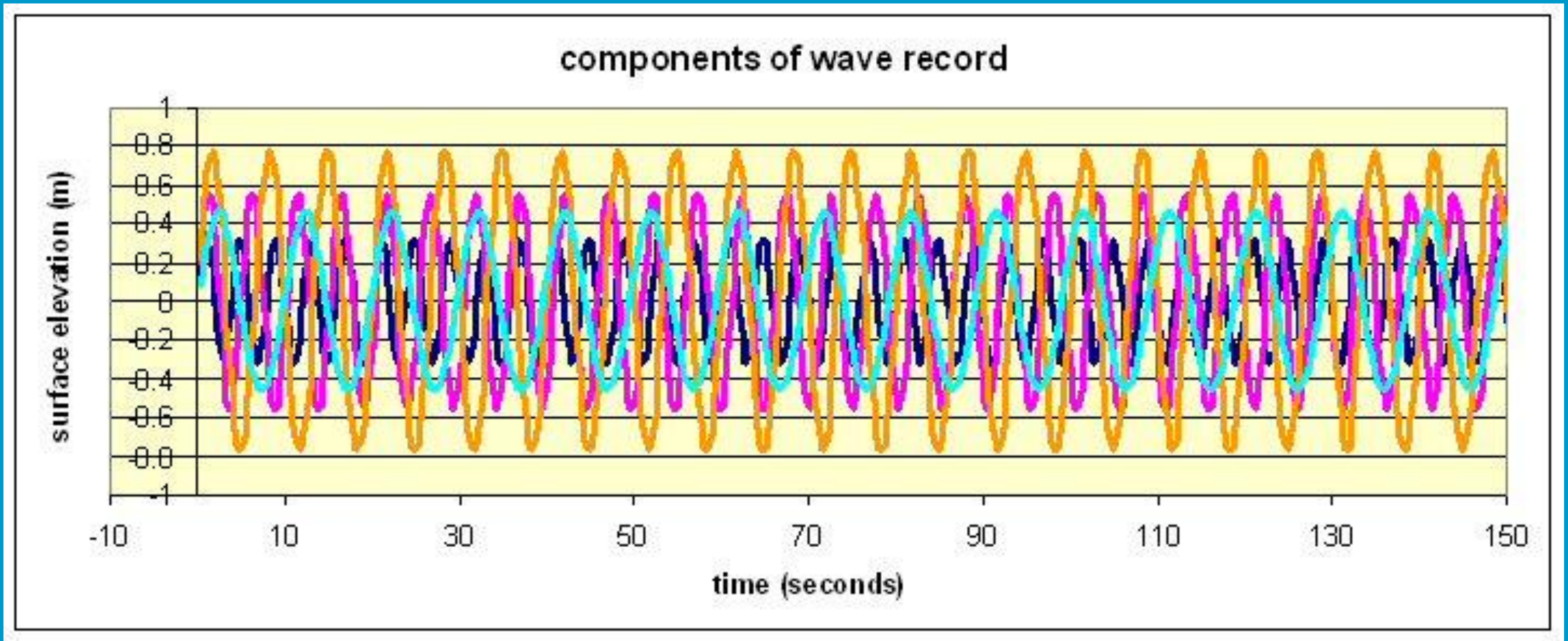
TU Delft

Example wave record



28 waves, H_s = "13% wave", H_s= wave nr 4, $H_s \approx 3.8$
28 waves in 150 seconds, so $T_m = 5.3$ s

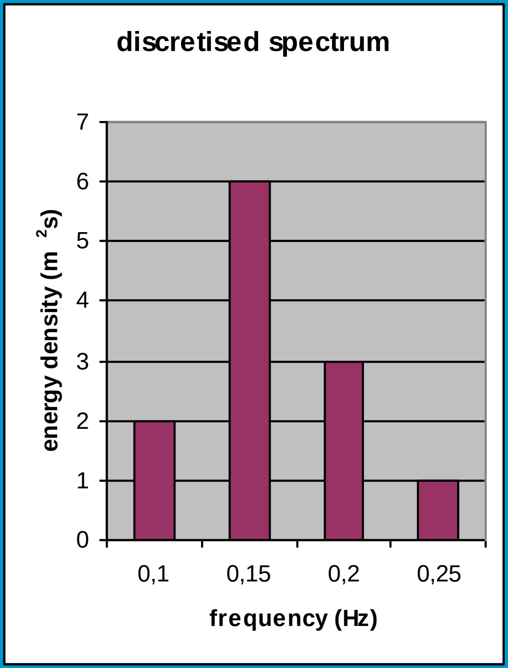
composition of the record

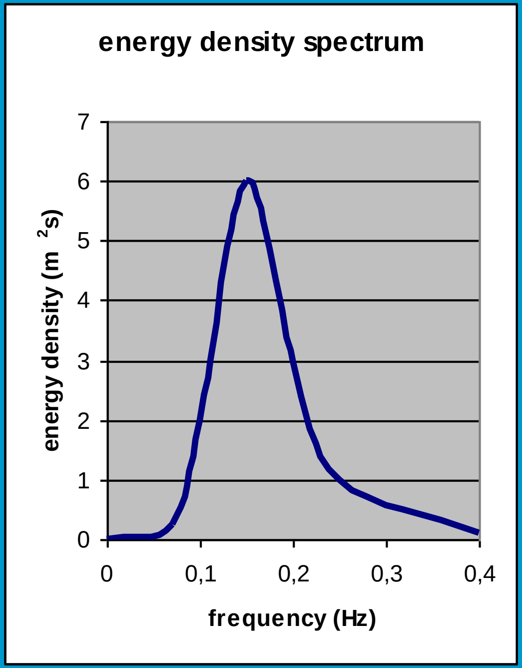


$H_1 = 0.63$ m	$T_1 = 4$ sec	
$H_2 = 1.80$ m	$T_2 = 5$ sec	
$H_3 = 1.55$ m	$T_3 = 6.67$ sec	
$H_4 = 0.90$ m	$T_4 = 10$ sec	$T_m = 5.3$ sec

Spectrum

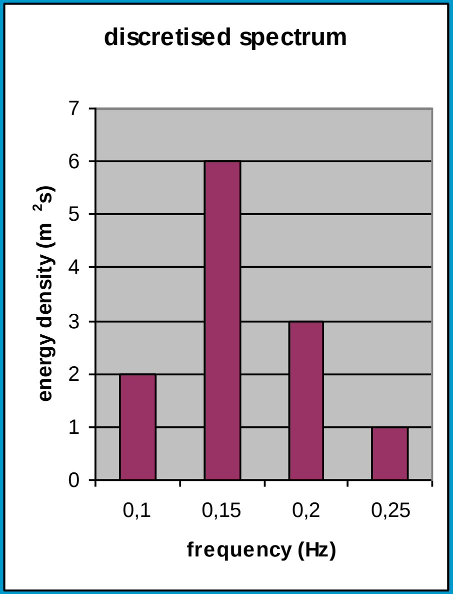
$$\frac{1}{2}a^2 = S \cdot \Delta f$$





$$H = \sqrt{8S \cdot \Delta f} \qquad S = \frac{H^2}{8\Delta f} = \frac{1.55^2}{8 \cdot 0.05} = 6\ [m^2 s]$$

Calculation of m_0

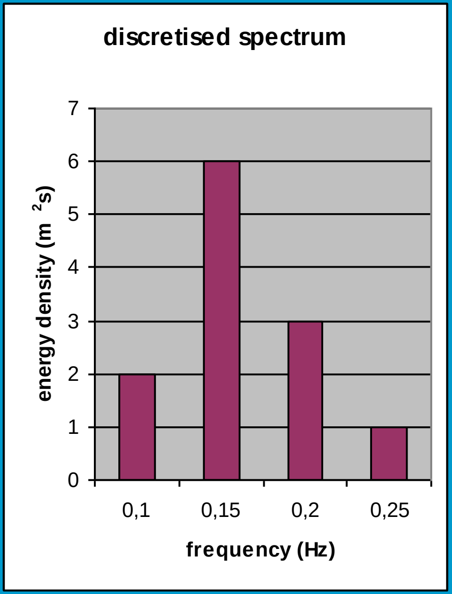


0.05*2	0.10
0.05*6	0.30
0.05*3	0.15
0.05*1	0.05
	0.60

$$4\sqrt{m_0} = 3.1\,m$$

$$m_n = \int_0^\infty f^n S(f)\,df$$

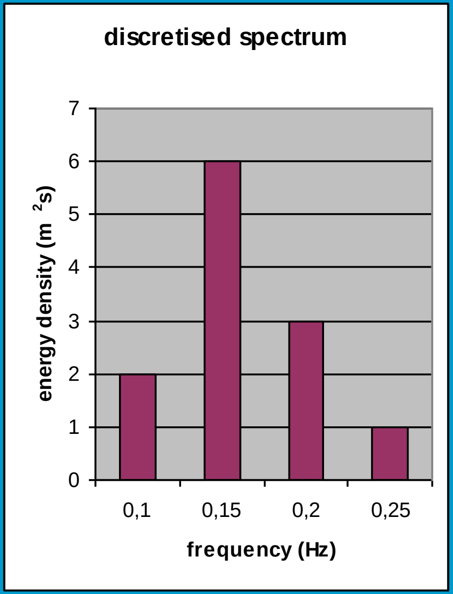
Calculation of m_1



dist * SΔf	
0.10*0.10	0.010
0.15*0.30	0.045
0.20*0.15	0.030
0.25*0.05	0.013
	0.098

$$m_n = \int_0^\infty f^n S(f)\, df$$

Calculation of m_2

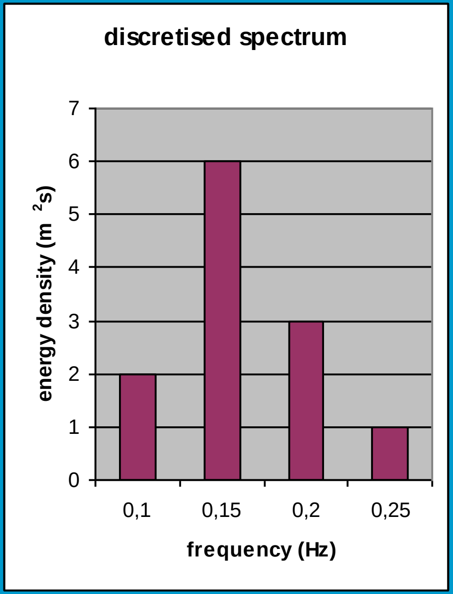


dist² * SΔf	
$0.10^2*0.10$	$1.00\ 10^{-3}$
$0.15^2*0.30$	$6.75\ 10^{-3}$
$0.20^2*0.15$	$6.00\ 10^{-3}$
$0.25^2*0.05$	$3.12\ 10^{-3}$
	$1.69\ 10^{-3}$

$$T = \sqrt{\frac{m_0}{m_2}} = 10\sqrt{\frac{0.60}{1.69}} = 5.69\,\text{sec}$$

$$m_n = \int_0^{\infty} f^n S(f)\,df$$

Calculation of m_{-1}



1/dist * SΔf	
1/0.10*0.10	1.0
1/0.15*0.30	2.0
1/0.20*0.15	0.75
1/0.25*0.05	0.20
	3.95

$$T_{m-1,0} = \frac{m_{-1}}{m_0} = \frac{3.95}{0.60} = 6.58 \text{ sec}$$

$$m_n = \int_0^\infty f^n S(f)\,df$$

Overview

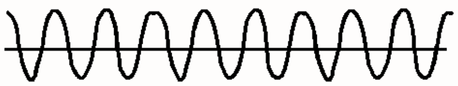
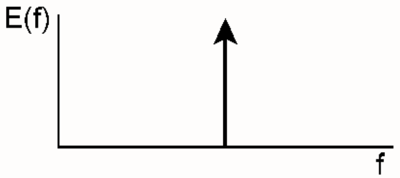
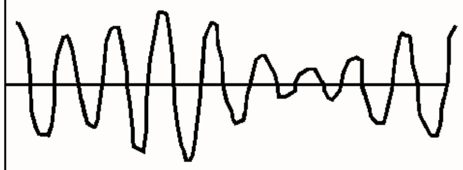
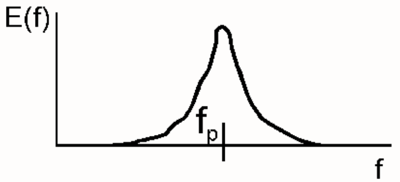
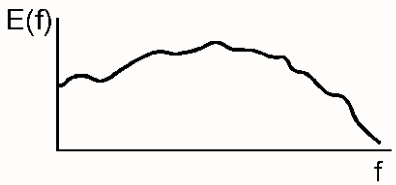
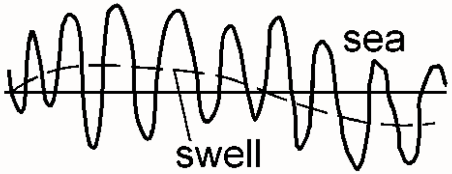
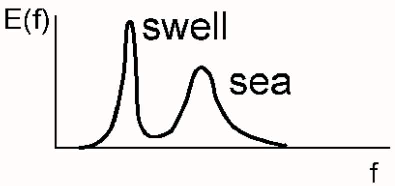
- H_{m0} = 3.1 m
(1.55+1.10+0.90+0.63=4.18)
- T_{m0} = 5.69 sec
- $T_{m-1,0}$ = 6.58 sec
- T_{peak} = 6.67 sec
- $\frac{T_{m-1,0}}{T_{m0}} = \frac{6.58}{5.69} = 1.16$
- T_m = 5.35 sec
- $\frac{T_{m0}}{T_m} = \frac{5.69}{5.35} = 1.06$

Usual assumptions:
$T_{m0} = T_p$
$T_{1/3} = T_m$

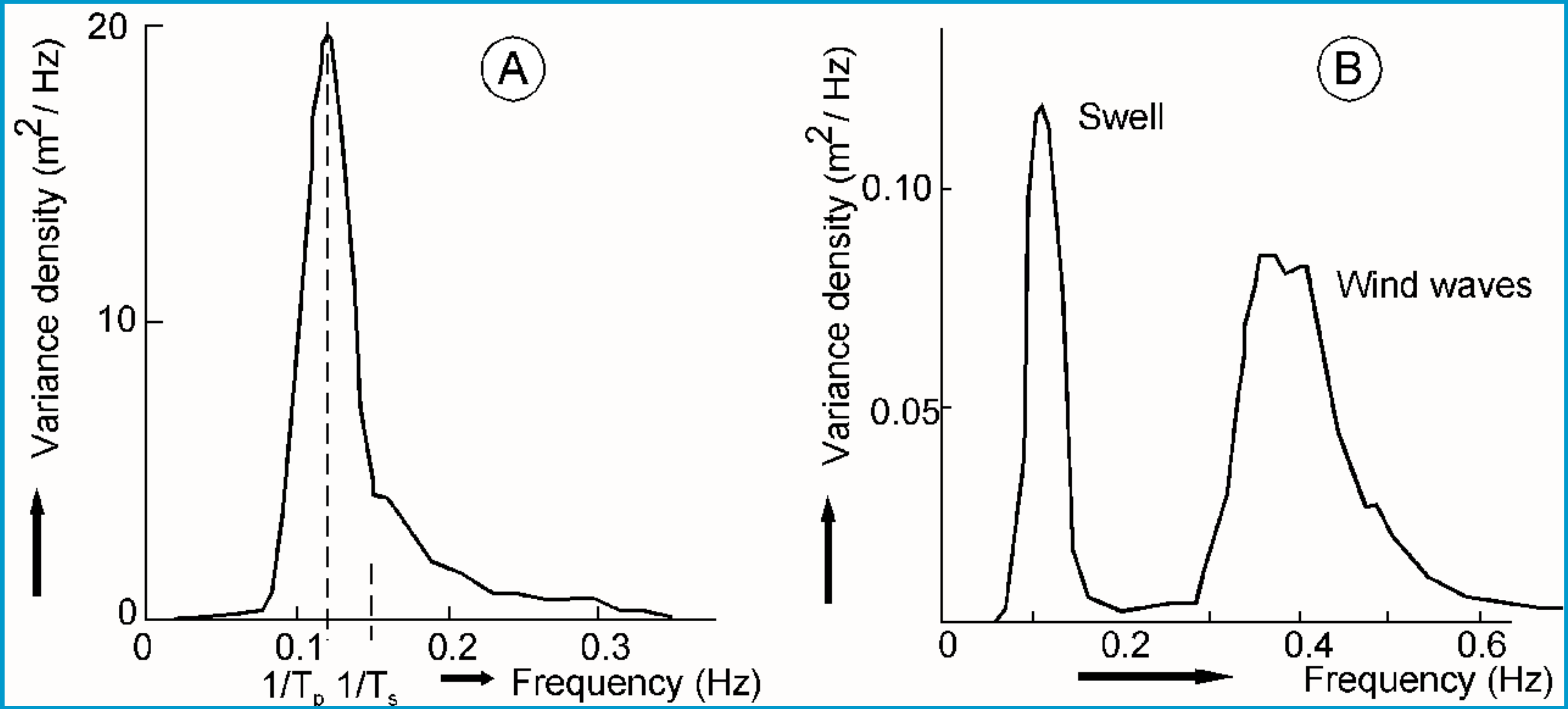
For standard spectra:

Goda:	$T_p=1.1\ T_{1/3}$
PM:	$T_p=1.15\ T_{1/3}$
Jonswap:	$T_p=1.07\ T_{1/3}$
TAW (vdMeer):	$T_p=1.1T_{m-1,0}$
Old Test (vdMeer):	$T_p=1.04\ T_{m-1,0}$
Also:	$T_{m-1,0}=1.064T_{1/3}$

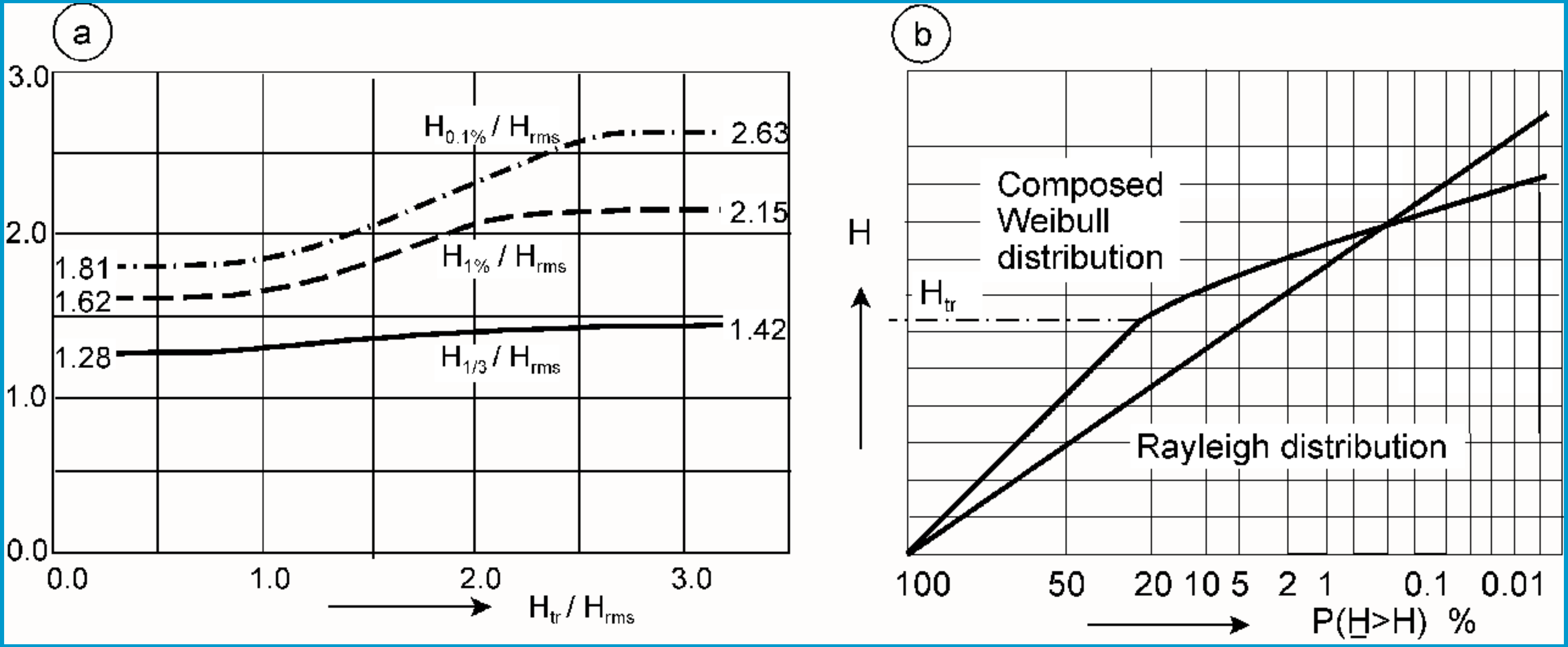
spectrum types

	Record	Spectrum
sine		
wind wave		
noise		
sea and swell		

two types of spectra



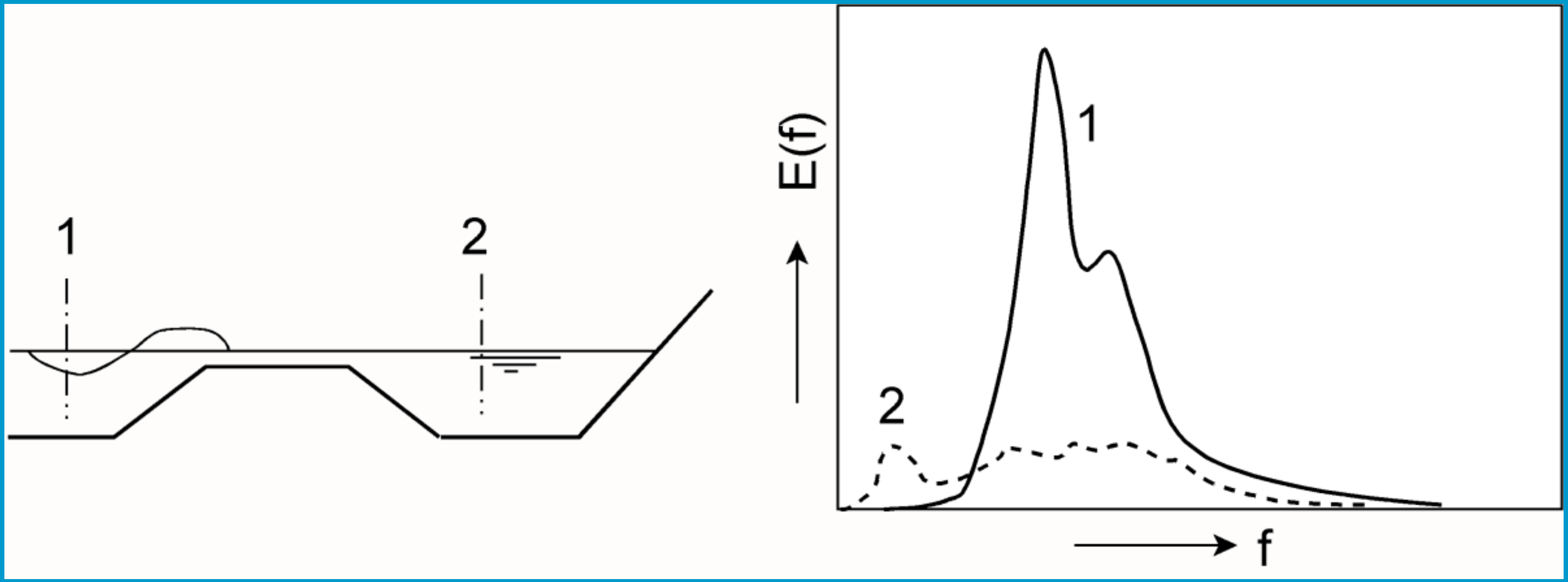
wave height distribution in shallow water



shallow water distributions

$$\Pr\{\underline{H} \leq H\} = \begin{cases} F_1(H) = 1 - \exp\left[-\left(\dfrac{H}{H_1}\right)^2\right] & H \leq H_{tr} \\ F_2(H) = 1 - \exp\left[-\left(\dfrac{H}{H_2}\right)^{3.6}\right] & H > H_{tr} \end{cases}$$

wave spectra across shallow bar

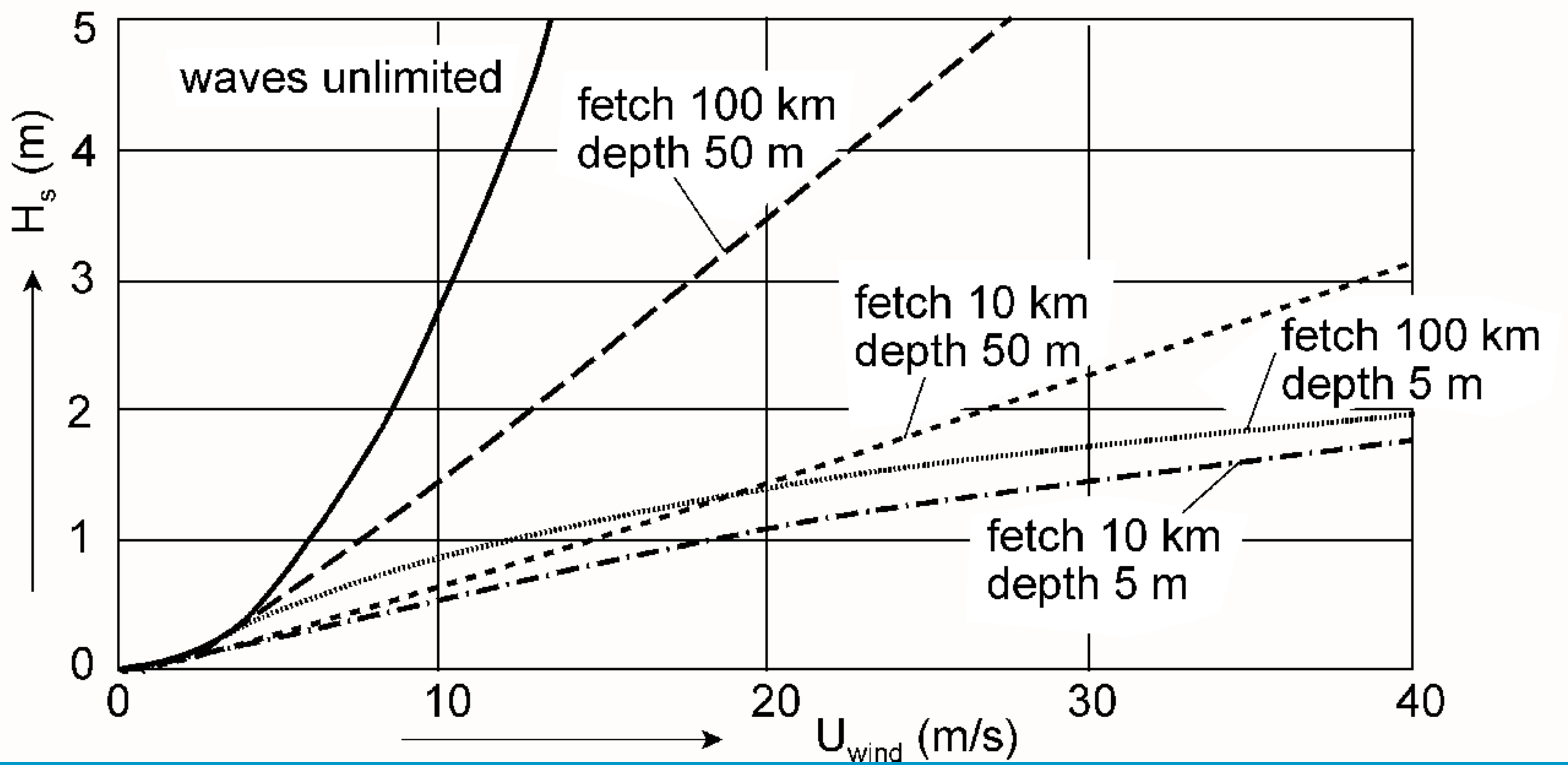


wave generation

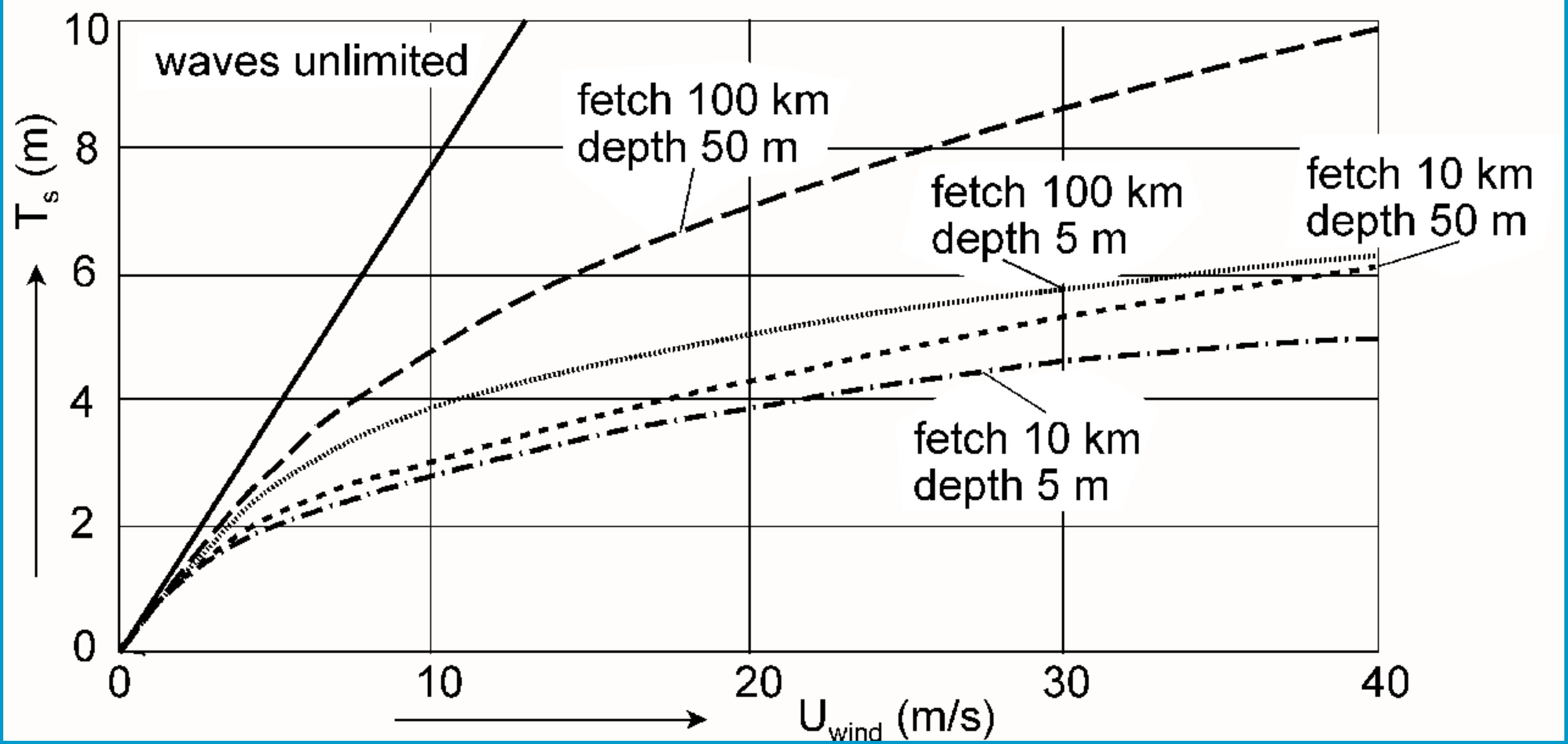
$$\frac{gH_s}{u_w^2} = 0.283 \tanh\left[0.578\left(\frac{gh}{u_w^2}\right)^{0.75}\right] \tanh\left[\frac{0.0125\left(\frac{gF}{u_w^2}\right)^{0.42}}{\tanh\left[0.578\left(\frac{gh}{u_w^2}\right)^{0.75}\right]}\right]$$

$$\frac{gT_s}{2\pi u_w} = 1.20 \tanh\left[0.833\left(\frac{gh}{u_w^2}\right)^{0.375}\right] \tanh\left[\frac{0.077\left(\frac{gF}{u_w^2}\right)^{0.25}}{\tanh\left[0.833\left(\frac{gh}{u_w^2}\right)^{0.375}\right]}\right]$$

wave height as function of wind, depth and fetch



wave period as function of wind, depth and fetch



wave height as function of duration, fetch and wind velocity