出國報告（出國類別：進修）

赴荷蘭參加「海岸與港口結構」訓練課程報告

服務機關：交通部臺灣港務股份有限公司高雄港務分公司
姓名職稱：吳明東副工程師
派赴國家：荷蘭
出國期間：106 年 3 月 6 日至 106 年 3 月 24 日
報告日期：106 年 6 月 9 日

摘要：

一、「IHE Delft Institute for Water Education」(以下簡稱 IHE Delft)是世界上最大的國際性學士後水教育機構之一，設於荷蘭台夫特。IHE Delft 提供可授予學位的博士及碩士學程，並提供多樣的短期訓練課程。自 1957 年來，已有來自超過 160 個國家，14,500 名以上專業人士在該機構受過教育或訓練。

二、本次奉派參加 IHE Delft 之「海岸與港口結構」(Coastal and Port Structures)短期課程，主要介紹各種型式之防波堤、影響防波堤設計之各種邊界條件、防波堤斷面結構及尺寸之計算方法及水工試驗模型尺寸之計算等，各單元課程配合隨堂習題演練加強理解，最後參與課程的學員被要求依給定的條件，選擇一種防波堤型式，實際設計出防波堤斷面結構及決定水工試驗模型之尺寸，並須作成報告，於課堂中向教授及全體學員發表，接受詢問及挑戰。藉此，更可確保學員的學習成效。

三、本次為期三周的課程須完成上述的內容，雖然強度頗高，但經過本次課程後，確實對防波堤的設計能有進一步的了解，算是獲益良多。另外，與來自其他國家的夥伴合作，共同完成期末報告，更是難得的經驗。

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壹、目的

港務公司為培育人才，提供員工國外短期訓練課程，增進員工專業學能，並獲取國際交流經驗。

「IHE Delft Institute for Water Education」(以下簡稱 IHE Delft)是世界上最大的國際性學士後水教育機構之一，設於荷蘭台夫特。IHE Delft 提供可授予學位的博士及碩士學程，並提供多樣的短期訓練課程。自 1957 年來，已有來自超過 160 個國家，14,500 名以上專業人士在該機構受過教育或訓練。

本次奉派參加 IHE Delft 之「海岸與港口結構」(Coastal and Port Structures)課程，係為期三周之短期課程，主要介紹各種型式之防波堤及其設計方法。

防波堤是港埠工程中重要項目之一，本分公司進行中的工程計畫中，即涵蓋多個防波堤工程，未來隨著港口的發展，更是少不了防波堤工程。藉由本次課程的研習，期能增進防波堤相關的專業職能。

貳、課程內容

一、課程名稱:「海岸與港口結構」(Coastal and Port Structures)。

二、訓練機構: IHE Delft Institute for Water Education，位於荷蘭台夫特(Delft)，位置如下圖:

三、課程期間: 三周(106 年 3 月 6 日至 106 年 3 月 24 日)。

四、每日行程:

(一)106 年 3 月 6 日(星期一):

上午: 報到、防波堤設計。

下午: 防波堤設計。

(二)106 年 3 月 7 日(星期二):

上午: 防波堤設計。

下午: 自習。

(三)106 年 3 月 8 日(星期三):

小組討論。

(四)106 年 3 月 9 日(星期四):

上午: 自習。

下午: 防波堤設計。

(五)106 年 3 月 10 日(星期五):

小組討論。

(六)106 年 3 月 13 日(星期一):

上午: 自習。

下午: 防波堤設計。

(七)106 年 3 月 14 日(星期二):

上午: 防波堤設計。

下午: 自習。

(八)106 年 3 月 15 日(星期三):

小組討論。

(九)106 年 3 月 16 日(星期四):

上午: 防波堤設計。

下午: 防波堤設計、防波堤設計習作。

(十)106 年 3 月 17 日(星期五):

上午: 防波堤設計。

下午: 防波堤設計、防波堤設計習作。

(十一)106 年 3 月 20 日(星期一):

小組討論。

(十二)106 年 3 月 21 日(星期二):

上午: 自習。

下午: 防波堤設計、防波堤設計習作。

(十三)106 年 3 月 22 日(星期三):

上午:離岸工程。

下午:離岸工程。

(十四)106 年 3 月 23 日(星期四):

上午: 小組討論。

下午: 專題報告簡報。

(十五)106 年 3 月 24 日(星期五):

離校。

四、課程內容簡介:

本次三周之課程，主要介紹防波堤之設計，課程內容如下:

(一)防波堤之介紹:

甲、防波堤的功能:

i. 防波浪。
ii. 防海流。
iii. 避免航道淤積。
iv. 提供碼頭、船塢等之用。

乙、需求:

i. 配置。(依所要保護的範圍、標的，配置防坡堤)

ii. 滲透性。(依使用目的決定防波堤是否可允許水滲透)

iii. 堤頂高。(依可允許的越波量及波浪透過率決定堤頂高度)

iv. 可接近性。(依使用目的決定可否允許人、車等進入防波堤)

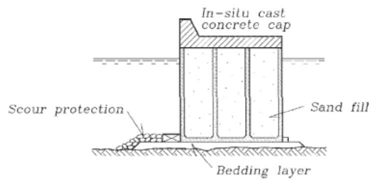
v. 堤背。(依使用需求決定堤背設計)

丙、防波堤型式:

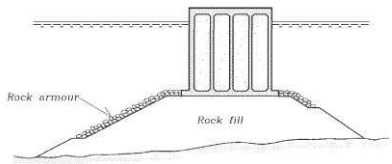
i. 抛石堤(rubble mound breakwater):又分為護坡為石塊或消波塊兩種。

ii.　　平台式拋石堤(berm breakwater)

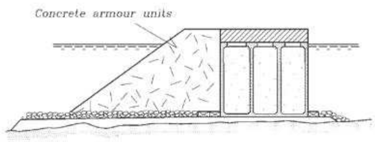
iii.　　直立式防波堤(vertical wall breakwater)-沉箱式(caisson):



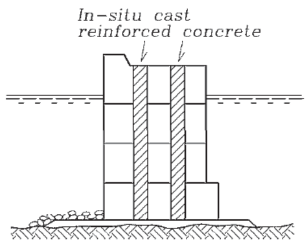
iv.　　直立式防波堤(vertical wall breakwater)-垂直複合式:



v.　　直立式防波堤(vertical wall breakwater)-水平複合式:



vi.　　直立式防波堤(vertical wall breakwater)-方塊式(block):



vii.　　潛堤(low-crested breakwater):

(二)防波堤設計之邊界條件

甲、土壤承載力:

由現地鑽探、測試求得。

乙、水深:

現地測量求得。

丙、潮位:

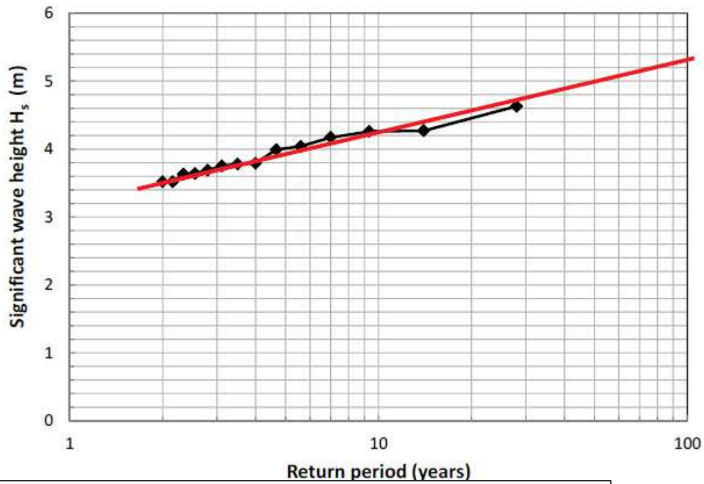
歷史測量紀錄分析求得。

丁、波浪:

i. 設計波:

由長期實測波浪資料，以統計分析方法，求得各迴歸期設計波。

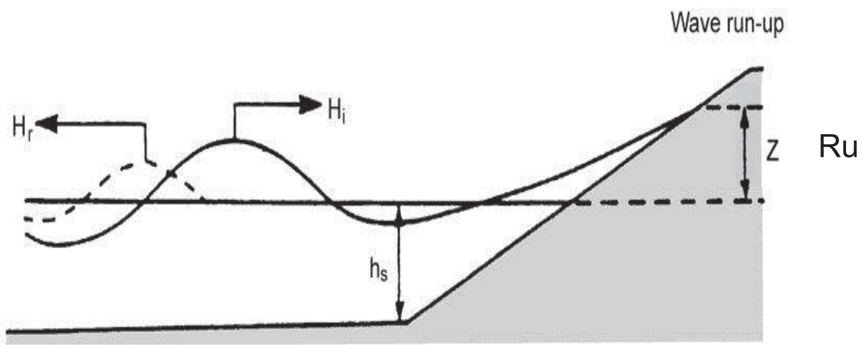
Date and time	H_s (m)
1-11-2012 2:00	4.63
10-12-2008 15:00	4.27
11-11-2012 11:00	4.26
8-12-1992 12:00	4.17
28-10-2012 12:00	4.04
11-2-2013 22:30	3.99
10-3-2010 0:00	3.79
31-1-2014 5:00	3.78
29-3-2009 16:00	3.75
16-5-2013 21:30	3.69
16-11-2002 15:00	3.64
23-12-2010 23:30	3.63
28-4-2009 2:30	3.52
25-3-2013 9:00	3.52



由長期實測波浪資料，統計分析求得各迴歸期設計波之範例

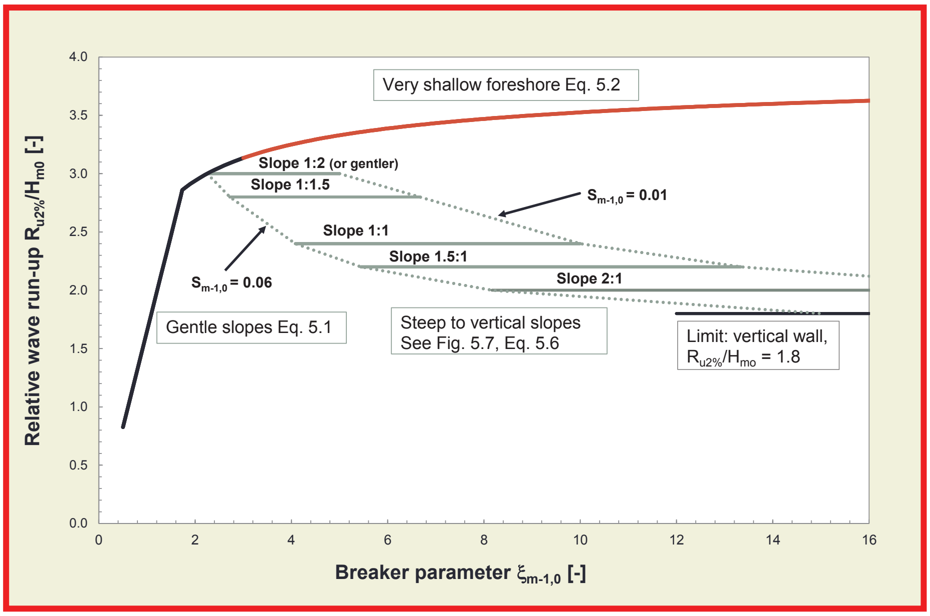
ii. 波的湧升(wave run-up):

定義如下圖:

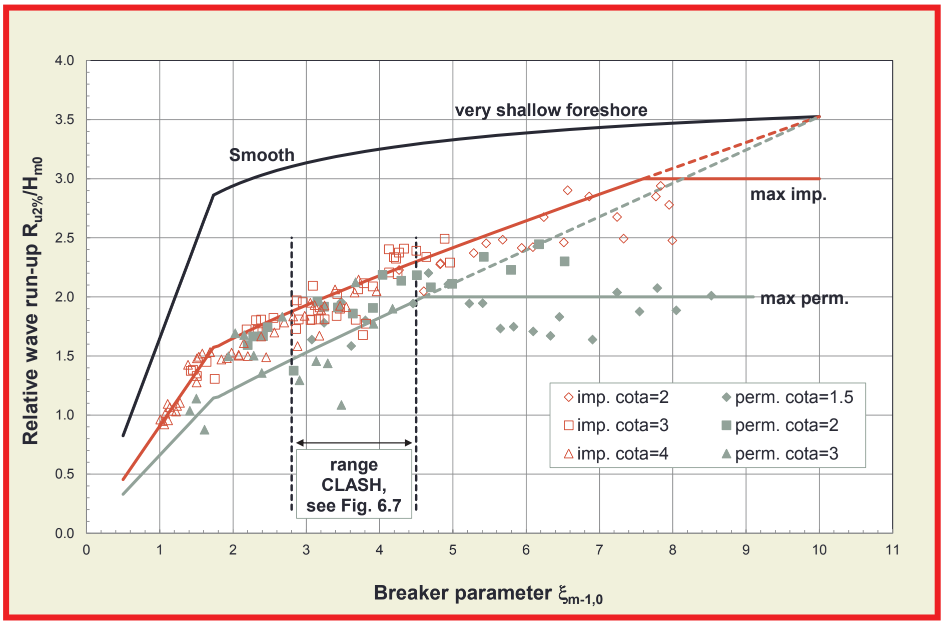


計算如下:

對平滑斜面(參見:EurOtop 2016):

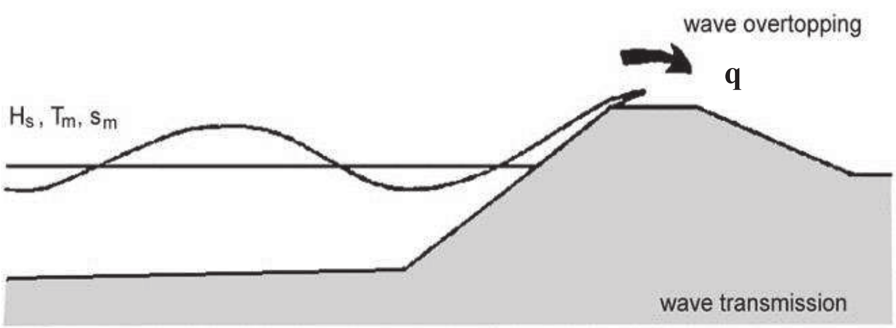


對石塊斜面(update EurOtop, data Van der Meer, 1988):



iii. 越波(wave overtopping):

定義如下:(波浪越過堤頂的量 q (m3/s per m or l/s per m))



計算如下:

越波量計算方法之一

No influencing foreshore:
$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.047 \cdot \exp\left[-\left(2.35\frac{R_C}{H_{m0}}\right)^{1.3}\right]$$

Influencing foreshore, non-impulsive:
$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.05\exp\left(-2.78\frac{R_c}{H_{m0}}\right)$$

Influencing foreshore, impulsive:
$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.011\left(\frac{H_{m0}}{hs_{m-1,0}}\right)^{0.5}\exp\left(-2.2\frac{R_c}{H_{m0}}\right) \quad \text{valid for } 0 < R_c/H_{m0} < 1.35$$

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.0014\left(\frac{H_{m0}}{hs_{m-1,0}}\right)^{0.5}\left(\frac{R_c}{H_{m0}}\right)^{-3} \quad \text{valid for } R_c/H_{m0} \geq 1.35$$

各種情形下越波量的限制:

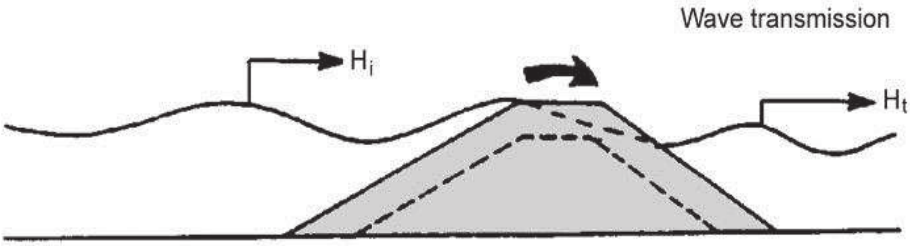
Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
Rubble mound breakwaters; H_{m0} > 5 m; no damage	1	2,000-3,000
Rubble mound breakwaters; H_{m0} > 5 m; rear side designed for wave overtopping	5-10	10,000-20,000
Grass covered crest and landward slope; closed grass cover; H_{m0} = 1-3 m	5	2,000-3,000
Grass covered crest and landward slope; maintained grass cover; H_{m0} = 1-3 m	1	1,000-2,000
Grass covered crest and landward slope; not maintained grass cover, open spots, moss, vegetable garden; H_{m0} = 0.5-3 m	0.1	500
Grass covered crest and landward slope; H_{m0} < 1 m	5-10	500
Grass covered crest and landward slope; H_{m0} < 0.3 m	No limit	No limit

Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
Significant damage or sinking of larger yachts; H_{m0} > 5 m	>10	>5,000 – 30,000
Significant damage or sinking of larger yachts; H_{m0} = 3-5 m	>20)	>5,000 – 30,000
Sinking small boats set 5-10 m from wall; H_{m0} = 3-5 m Damage to larger yachts	>5	>3,000-5,000
Safe for larger yachts; H_{m0} > 5 m	<5	<5,000
Safe for smaller boats set 5-10 m from wall; H_{m0} = 3-5 m	<1	<2,000
Building structure elements; H_{m0} = 1-3 m	≤1	<1,000
Damage to equipment set back 5-10m	≤1	<1,000

Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
People at structures with possible violent overtopping, mostly vertical structures	No acces for any predicted overtopping	No acces for any predicted overtopping
People at rubble mound breakwater crest and at dike crest. Clear view on the sea. H_{m0} = 3 m H_{m0} = 2 m H_{m0} = 1 m H_{m0} < 0.5 m	 0.3 1 10-20 No limit	 400 – 600 400 – 600 400 – 600 No limit
Cars on crest of a dike for dike inspection. H_{m0} = 3 m H_{m0} = 2 m H_{m0} = 1 m	 <5 10-20 <75	 1000-2000 1000-2000 1000-2000
Highways and roads, fast traffic	Close before debris in spray becomes dangerous	Close before debris in spray becomes dangerous
Railway tracks, slowly moving train	See cars on crest of a dike	See cars on crest of a dike

iv. 波的穿透(wave transmission):

定義如下:(穿透係數 Kt or Ct = Ht/Hi)



計算如下:

石塊斜面:

d'Angremond et al., 1996, for $B/H_s < 8$

$$K_t = -0.4\frac{R_c}{H_{si}} + 0.64\left(\frac{B}{H_{si}}\right)^{-0.31}\left(1 - e^{-0.5\xi_{op}}\right)$$

Lower boundary: $K_t = 0.075$ Upper boundary: $K_t = 0.8$

New formula for $B/H_s > 12$

$$K_t = -0.35\frac{R_c}{H_{si}} + 0.51\left(\frac{B}{H_{si}}\right)^{-0.65}\left(1 - e^{-0.41\xi_{op}}\right)$$

Lower boundary: $K_t = 0.05$ Upper boundary: $K_t = 0.93 - 0.006B/H_s$

Interpolation $8 < B/H_s < 12$

平滑斜面:

$$K_t = [-0.30\frac{R_c}{H_i} + 0.75(1-e^{-0.5\xi})](\cos\beta)^{2/3}$$

Minimum: $K_t = 0.075$

Maximum: $K_t = 0.8$

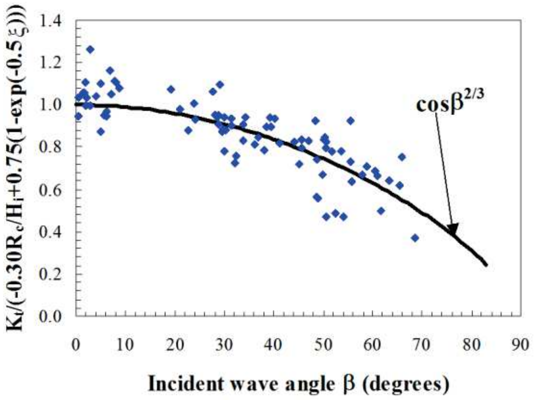
No influence of B!

Limitations:

$$1.0 < \xi < 3.0$$

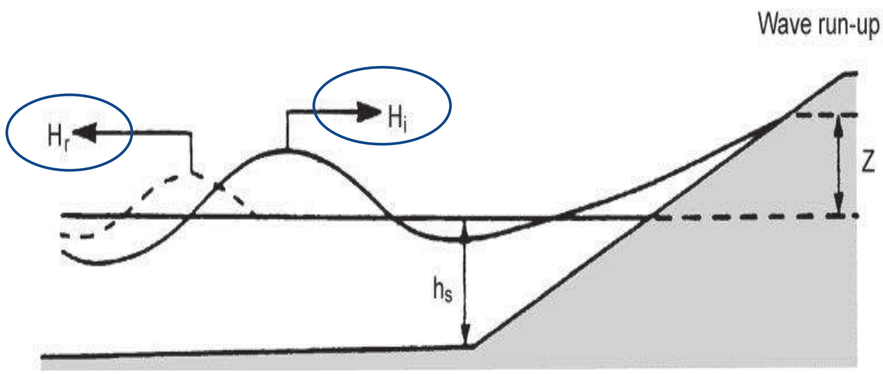
$$0^\circ \le \beta \le 70^\circ$$

$$1.0 \le \frac{B}{H_i} \le 4.0$$



v. 波的反射(wave reflection):

定義如下:(反射係數 Kr or Cr = Hr/Hi)

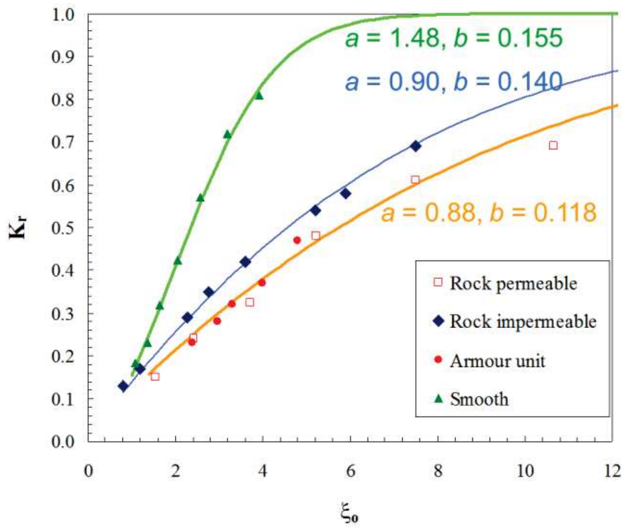


計算如下:

Zanuttigh and Van der Meer, 2006

$$K_r = \tanh(a \cdot \xi_o^b)$$

Vertical structure K_r = 0.9 – 1.0



(三)防波堤設計:

甲、拋石堤:

i. 石塊相關參數:

Nominal diameter D_{n50}

$D_{n50} = V^{1/3} = (M_{50}/\rho_r)^{1/3}$ = cubic size

$\rho_r = 2600 - 2700\ kg/m^3$ (rock)

$\rho_r = 2400\ kg/m^3$ (normal concrete)

Concrete units: D_n

Relative buoyant density: Δ

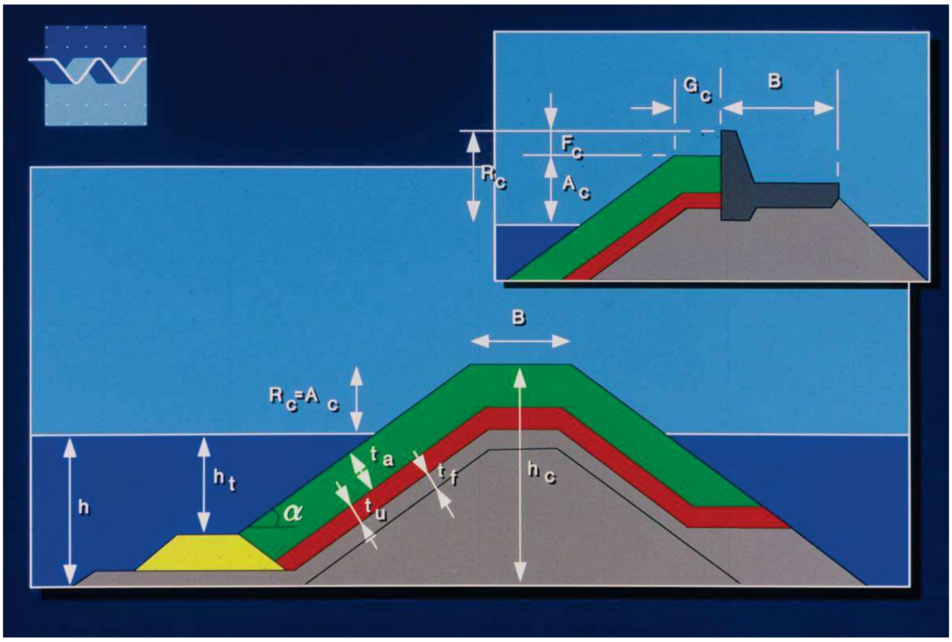
$\Delta = (\rho_r - \rho_w)/\ \rho_w$

$\Delta = 1.4 - 1.6$ in most situations

ii. 石塊分級:

Narrow grading		Wide grading		Very wide grading	
$D_{85}/D_{15} < 1.5$		$1.5 < D_{85}/D_{15} < 2.5$		$D_{85}/D_{15} > 2.5$	
Class	D_{85}/D_{15}	Class	D_{85}/D_{15}	Class	D_{85}/D_{15}
15 – 20 t	1.10	1 – 9 t	2.00	50- 1000 kg	2.71
10 – 15 t	1.14	1 – 6 t	1.82	20 -1000 kg	3.68
5 – 10 t	1.26	100 – 1000 kg	2.15	10 – 1000 kg	4.64
3 – 7 t	1.33	100 – 500kg	1.71	10 – 500 kg	3.68
1 – 3 t	1.44	10 – 80 kg	2.00	10 – 300 kg	3.10
00–1000 kg	1.49	10 – 60 kg	1.82	20 – 300 kg	2.46

iii. 斷面相關參數:



iv. 堤頂寬度與保護層厚度之計算:

Minimum crest width B_{min}:

$B_{min} = (3 \text{ to } 4)\ D_{n50}$

The thickness of layers:

$t_a = t_u = t_f = n\ k_t\ D_{n50}$

The number of units per m²:

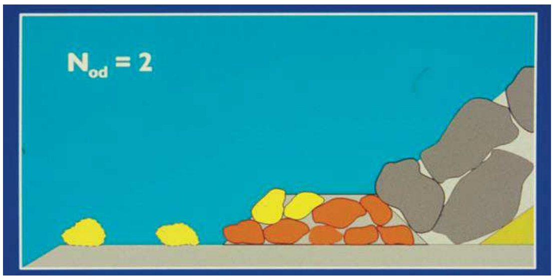
$N_a = n\ k_t\ (1 - n_v)/D_{n50}^2$

where:

t_a, t_u, t_f = thickness of armour, under layer or filter
n = number of layers
k_t = layer thickness coefficient
n_v = volumetric porosity
ϕ = packing density

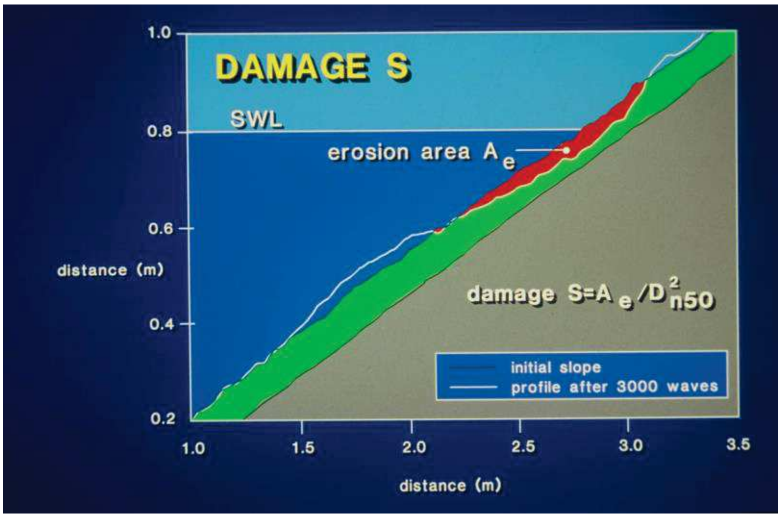
v. 破壞數:N_{od}

一定寬度內石塊實際被移動的個數。



vi. 破壞等級: Damage level S

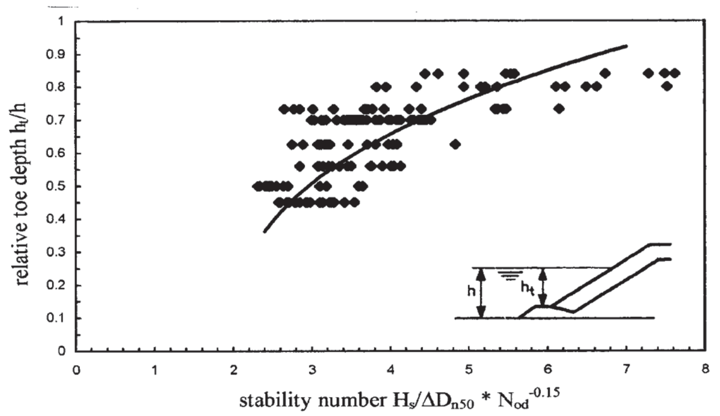
$S = Ae/D_{n50}^2$



vii.　安定數:

Stability number=Hs/ΔDn50，與波高級石塊大小有關。

Toe stability 可由下列關係式計算:



$$H_s/\Delta D_{n50} * N_{od}^{-0.15} = 2 + 6.2\,(h_t/h)^{2.7}$$

Application area;

$0.4 < h_t/h < 0.9$

$3 < h_t/D_{n50} < 25$

石塊護層 stability:

Van der Meer formulae (1988)

for plunging waves:

$$\frac{H_s}{\Delta D_{n50}} = 6.2\,P^{0.18}\left(\frac{S}{\sqrt{N}}\right)^{0.2}\xi_m^{-0.5}$$

and for surging waves:

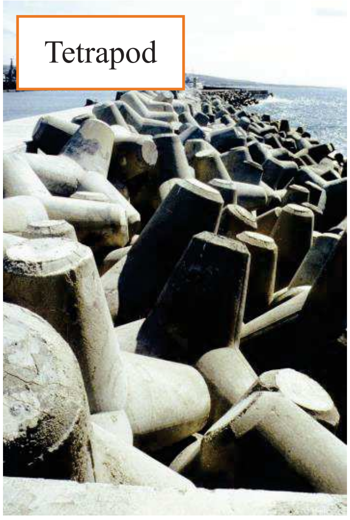
$$\frac{H_s}{\Delta D_{n50}} = 1.0\,P^{-0.13}\left(\frac{S}{\sqrt{N}}\right)^{0.2}\sqrt{\cot\alpha}\,\xi_m^{P}$$

$$\xi_{cr} = \left[6.2P^{0.31}\sqrt{tan\alpha}\right]^{\frac{1}{P+0.5}}$$

$\xi_m < \xi_{cr}$: plunging

$\xi_m \geq \xi_{cr}$: surging

viii. 水泥消波塊拋石堤之消波塊種類:



ix. 各種消波塊比較:

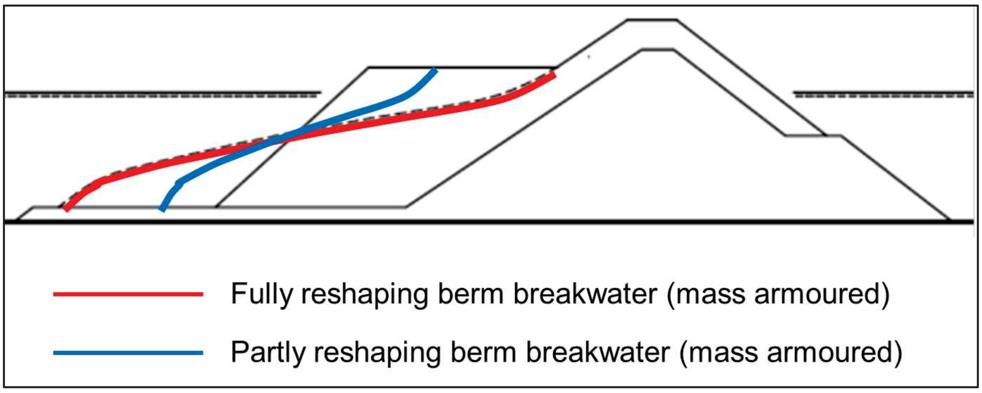
		Xbloc Accropde II			
	Accropode	Core-Loc	Tetrapod	Cube	Cube
number of layers	1	1	2	2	1
slope	1:4/3	1:4/3	1:1,5	1:1,5	1:1,5
K_D (breaking waves)	12	16	7	7	7
$H_s/\Delta D_n = N_s$	2.5	2.8	2.2	2.2	2.2
damage N_{od}	0	0	0,5	0,5	0
damage %	0	0	5	5	0
packing density ϕ	0,61	0,56	1,04	1,17	0,70
concrete per m^2 on slope	$0{,}182H_s$	$0{,}148H_s$	$0{,}350H_s$	$0{,}370H_s$	$0{,}236H_s$
relative volume of concrete	100%	81%	208%	220%	140%

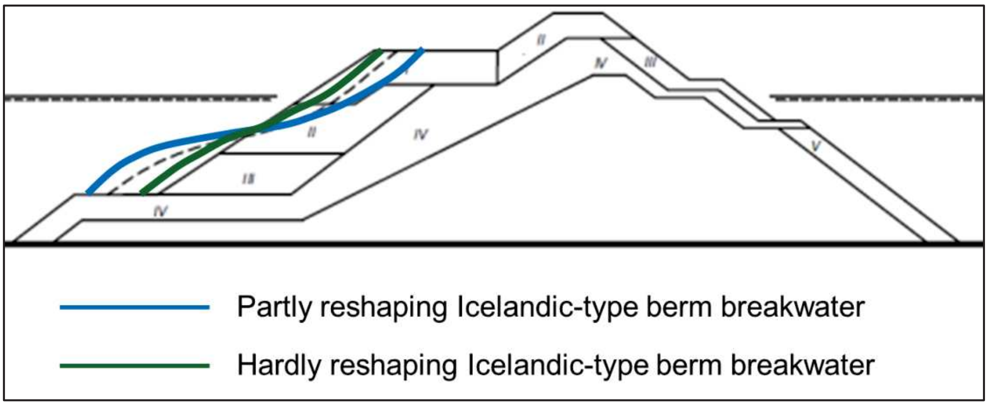
乙、平台式拋石堤(Berm breakwaters):

i. Berm breakwaters 分下列四種:

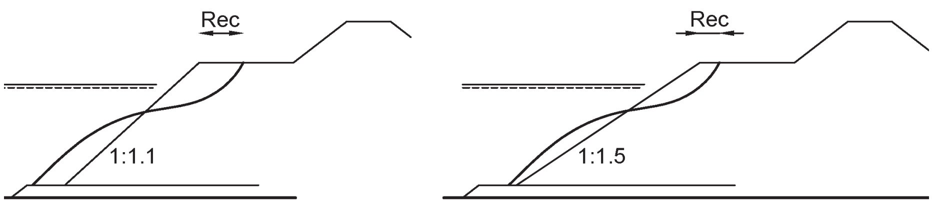
Breakwater	*Abbrevation*	$H_s/\Delta D_{n50}$	S_d	Rec/D_{n50}
Hardly reshaping berm breakwater (Icelandic-type)	**HR-IC**	1.7 - 2.0	2 - 8	0.5 - 2
Partly reshaping Icelandic-type berm breakwater	**PR-IC**	2.0 - 2.5	10 - 20	1 - 5
Partly reshaping mass armoured berm breakwater	**PR-MA**	2.0 - 2.5	10 - 20	1 - 5
Fully reshaping berm breakwater (mass armoured)	**FR-MA**	2.5 - 3.0	--	3 - 10

依可取得的石塊及期望的再塑形量來選擇。示意圖如下:





ii. Recession(Rec):堤面再塑形後減少的寬度，如下圖所示之 Rec。



Rec 計算公式如下:

$$Rec/D_{n50} = 1.6\ (H_s/\Delta D_{n50} - 1.0)^{2.5}$$

iii. 依允許的越波量計算堤頂高度，公式如下:

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.09 \cdot \exp\left[-\left(1.5\frac{R_c}{H_{m0} \cdot \gamma_{BB} \cdot \gamma_\beta}\right)^{1.3}\right]$$ **EurOtop (2016)**

with:

$\gamma_{BB} = 0.68 - 4.5s_{op} - 0.05B/H_{sD}$ for HR and PR

$\gamma_{BB} = 0.70 - 9.0s_{op}$ for FR

and B/H_{sD} is given by the design wave height.

iv. 授課教授自行開發一套以試算表(Excel)輔助設計的應用程式，將設計波高、週期、水深、潮位、允許的越波量等設計條件輸入後，即可產出防波堤的相關設計尺寸及畫出斷面圖，是相當實用的工具。示意如下:

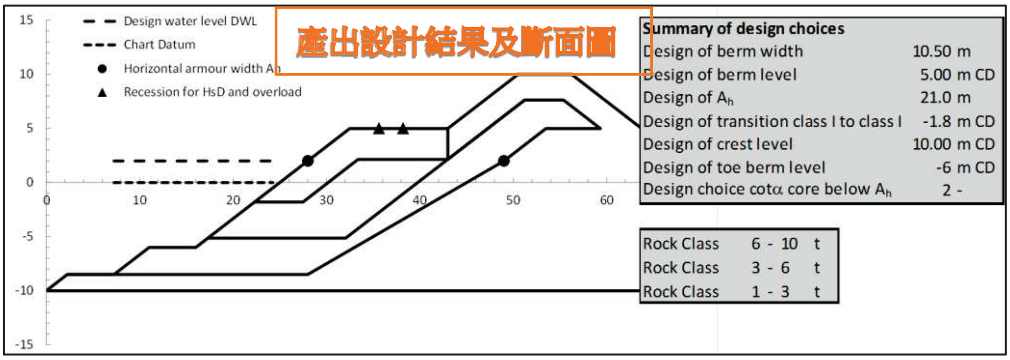
Conceptual design spreadsheet www.vdm-c.nl

輸入設計條件

General conditions		
Design wave height H_{sD}	3	m
Peak period T_p	9.8	s
Overload H_s	3.5	m
Design water level DWL	1	m CD
Lowest water level with H_{sD}	1	m CD
Lowest storm level	0	m CD
H_s at lowest storm level	3	m
Mean High Water Spring	1	m CD
Bottom level of foreshore at toe of structu	-9	m CD
Allowable overtopping q for H_{sD}	5	l/s per m
Allowable overtopping q for overload	20	l/s per m
Mass density water	1025	kg/m³
Mass density rock	2600	kg/m³
Choice of rock classes		
Rock Class I: minimum mass (0-15%)	1	t
Rock Class I: maximum mass (85-100%)	4	t
Rock Class II: minimum mass (0-15%)	0.2	t
Rock Class II: maximum mass (85-100%)	1	t
Rock Class III: M_{min} (leave open for MA)		t
Rock Class III: M_{max} (leave open for MA)		t

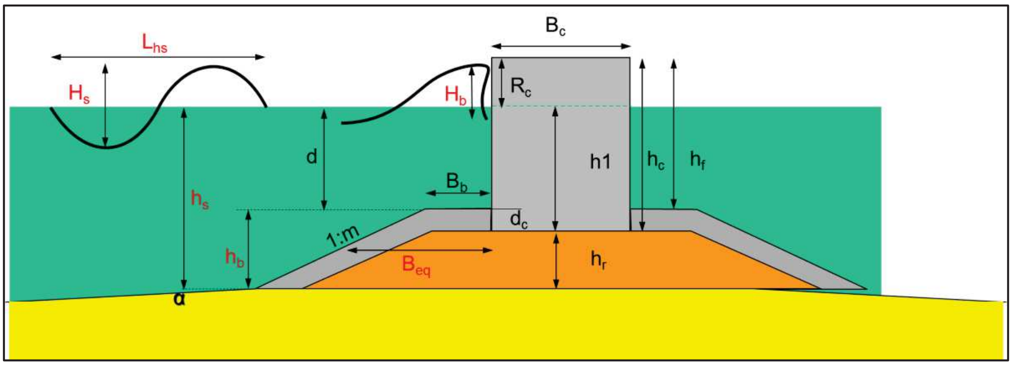
Outcome main parameters		
Wave steepness s_{op}	0.020	-
Relative mass density Δ	1.54	-
Median mass Class I M_{50}	2.5	t
Nominal diameter Class I D_{n50}	0.99	m
Stability number $H_{sD}/\Delta D_{n50}$	1.98	-
Type of berm breakwater	**Hardly reshaping**	
Number of rock classes for berm	2	
Basic recession for H_{sD} (no adaptation)	1.49	m
Recession for overload (no adaptation)	2.28	m
Nominal diameter Class II, D_{n50}	0.61	m
Nominal diameter Class III, D_{n50}	No Class	
Resiliency, berm width and level		
Wanted resiliency	20	%
Resulting Berm width B from resiliency	7.47	m
Minimum berm width B_{min} from geomet	2.96	m
Berm level $0.6H_{sD}$	2.8	m CD
Δw for waves during construction	1	m
MHWS plus Δw = working level	2	m CD
Minimum berm level from construction	3.97	m CD
Design choice of berm width	8.00	m
Design choice of berm level	4.00	m CD
Required horizontal armour width A_h	11.9	m
Design choice of A_h	12.0	m

Minimum transition level to class II		
For H_{sD} at lowest level	-0.2	m CD
For lowest level with according H_s	-1.2	m CD
Design choice of transition for IC (3 rock classes	-1.8	m CD
Transition lower class for MA (2 rock classes)	-1.8	m CD
Crest level ($\gamma_\beta = 1$)		
If no overtopping criteria, R_{cmin}	4.6	m CD
If no overtopping criteria, R_{cmax}	5.2	m CD
For given allowable overtopping, q, γ_{BB}	0.46	
Required crest level for design conditions	4.93	m CD
Required crest level for overload	4.87	m CD
Design choice of crest level	4.80	m CD
Check possibility of toe berm at level h_t		
Lowest possible toe level (two layers)	-6.27	m CD
Design conditions		
Allowable damage level for H_{sD}, N_{od}	2	-
Highest level of toe for H_{sD} with chosen N_{od}	-3.83	m CD
Check validity range h_t/D_{n50}	7.9	ok
Check validity range h_t/h	0.48	ok
Overload conditions		
Allowable damage level for overload, N_{od}	4	-
Highest level of toe for overload with chosen N	-4.12	m CD
Check validity range h_t/D_{n50}	8.3	ok
Check validity range h_t/h	0.51	ok
Design choice of toe berm level (0 if no berm)	0	m CD
Design choice cotα core below A_h	1.5	-

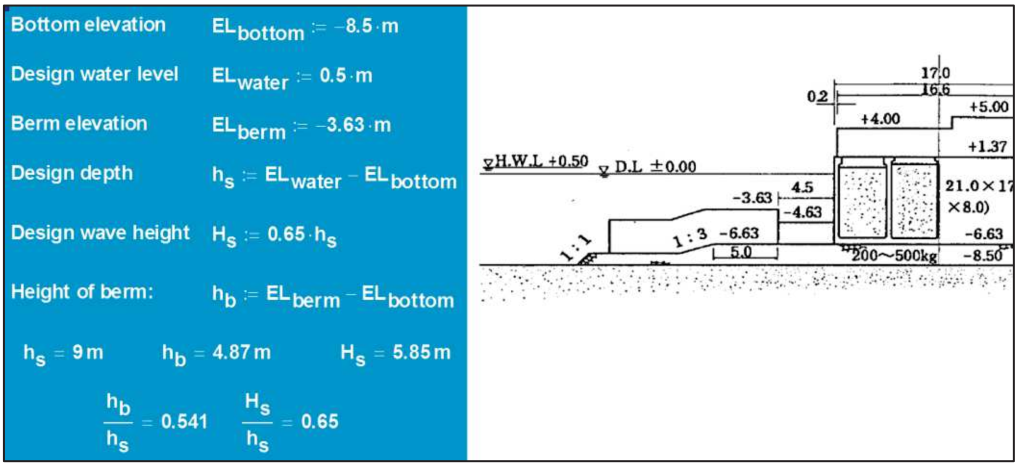


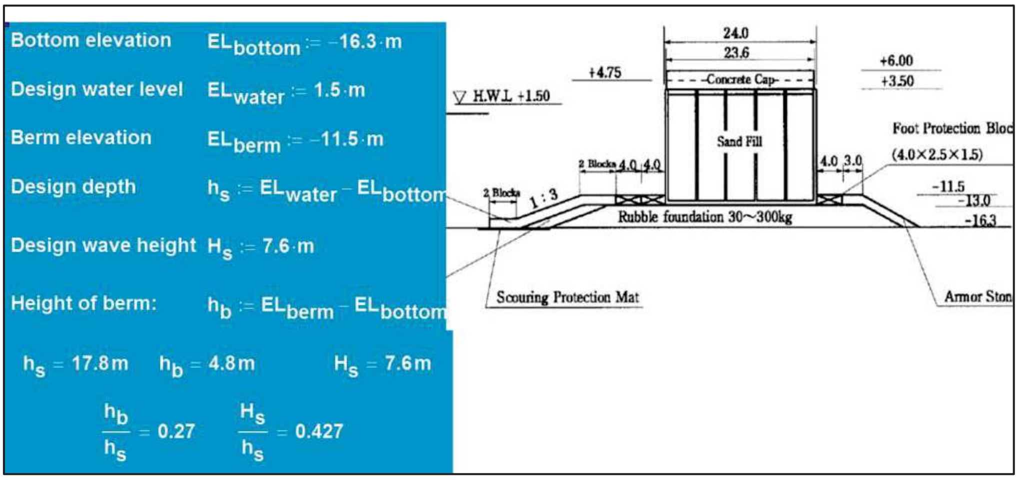
丙、直立式防波堤:

i. 幾何參數的定義(PROVERBS):

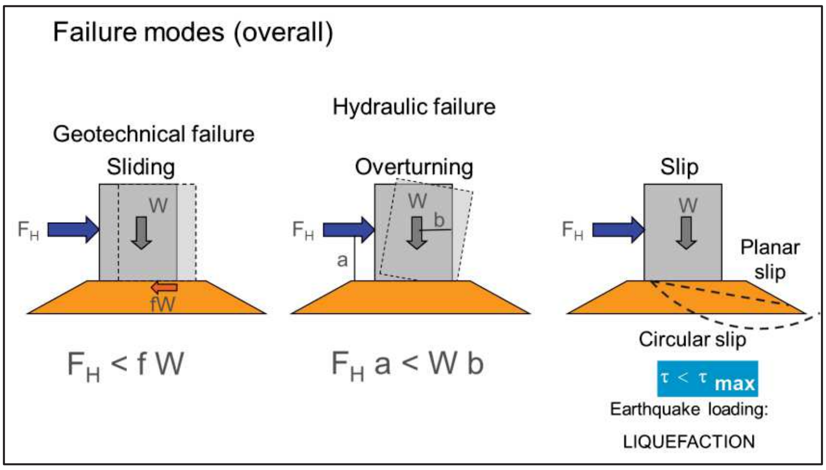


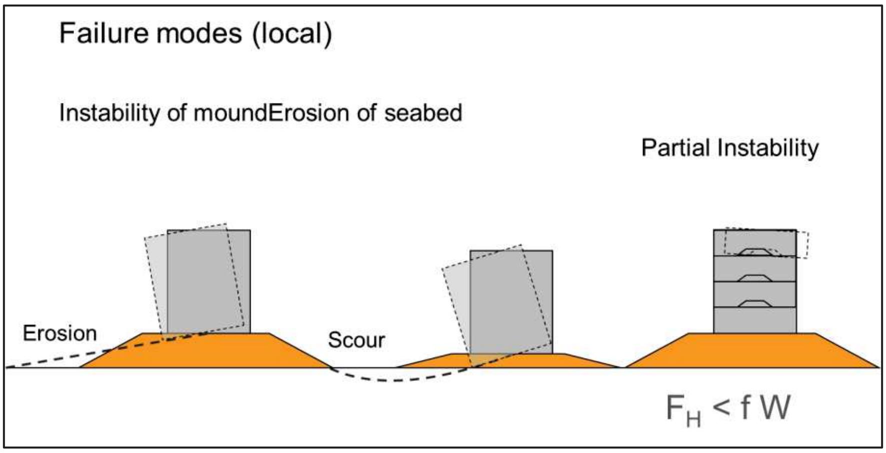
相關尺寸可參見下列設計範例:





ii. 破壞的模式:





iii.　　相關設計考量:

(四)波高 H_{max} for vertical structures = $1.8H_{1/3} = H_{1/250}$

(五)Goda (1984)建議:相對於 H_{max} 的週期等於 $T_{1/3}$。

(六)波長:

- Deep water: $L_0 := \frac{g \cdot T^2}{2 \cdot \pi}$
- Shallow water: solve iteratively $L = L_0 \cdot \tanh\left(\frac{2 \cdot \pi \cdot h}{L}\right)$
- Shallow water (5% accurate): $L_{approx}(d) := L_0 \cdot \sqrt{\tanh\left(\frac{2 \cdot \pi \cdot d}{L_0}\right)}$

● 設計水位需考量:

潮位(高潮及低潮)。

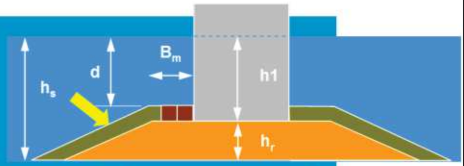
暴潮(結合高潮及低潮)。

海平面的上升。

● 堤基穩定性:

Stability Foundation Mound (The Rock Manual, eq. 5.189)
Based on model tests by Tanimoto and Takahashi.

$$k := \frac{2 \cdot \pi}{L_p}$$



$$\kappa_1 := \frac{2 \cdot k \cdot h_1}{\sinh(2 \cdot k \cdot h_1)}$$

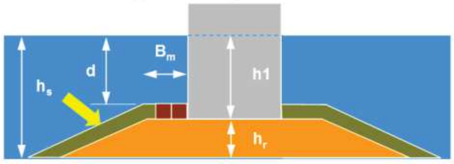
$$\kappa_2 := \max\left[0.45 \cdot (\sin(\beta))^2 \cdot (\cos(k \cdot B_m \cdot \cos(\beta)))^2, (\cos(\beta))^2 \cdot (\sin(k \cdot B_m \cdot \cos(\beta)))^2\right]$$

$$\kappa := \kappa_1 \cdot \kappa_2 \qquad a := \frac{1 - \kappa}{\kappa^{0.333}}$$

$$N_s := \max\left[1.8, 1.3 \cdot a \cdot \frac{h_1}{H_s} + 1.8 \cdot \exp\left[-1.5 \cdot a \cdot (1 - \kappa) \cdot \frac{h_1}{H_s}\right]\right]$$

$$W_{armour} := \rho_a \cdot g \cdot \left(\frac{H_s}{N_s \cdot \Delta}\right)^3 \qquad \Delta := \frac{\rho_a - \rho_w}{\rho_w}$$

Stability Foundation Mound (The Rock Manual, eq. 5.190)
Based on model tests by Madrigal and Valdés.



$$H_s/\Delta D_{n50} = (5.8\, h_1/h_s - 0.6)\, N_{od}^{0.19}$$

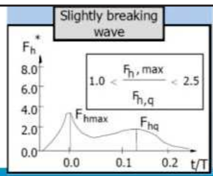
$$0.5 < h_1/h_s < 0.8$$
$$0.3 < B_m/h_s < 0.55$$

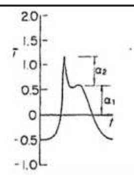
N_{od} = 0.5 almost no damage (1-3% of units displaced)
N_{od} = 2.0 acceptable damage (5-10% of units displaced)
N_{od} = 5.0 failure (20-30% of units displaced)

Do not forget!
Filter and underlayers
(Rubble mound breakwaters)

- 波力計算(Goda、Takahashi):

Quasi-static wave forces:





$$\eta := 0.75\cdot(1+\cos(\beta))\cdot H_{design}$$

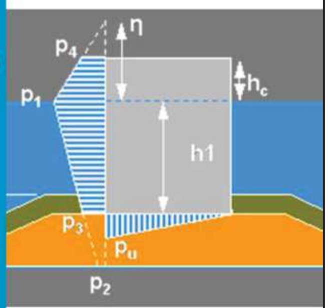
$$p_1 := \frac{1}{2}\cdot(1+\cos(\beta))\cdot\left[\alpha_1 + \alpha_2\cdot(\cos(\beta))^2\right]\cdot\rho_w\cdot g\cdot H_{design}$$

$$p_2 := \frac{p_1}{\cosh\left(\frac{2\cdot\pi\cdot h}{L}\right)}$$

$$p_3 := \alpha_3\cdot p_1$$

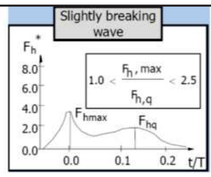
$$p_4 := \text{if}\left[\eta > h_c,\ p_1\cdot\left(1-\frac{h_c}{\eta}\right),\ 0\cdot Pa\right]$$

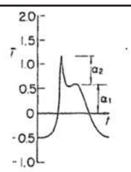
$$pu := 0.5\cdot(1+\cos(\beta))\cdot\alpha_1\cdot\alpha_3\cdot\rho_w\cdot g\cdot H_{design}$$



β = angle between wave direction and normal to breakwater axis

Quasi-static wave forces:



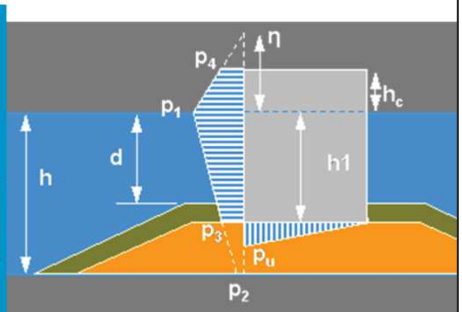


$$kh := \frac{2\cdot\pi\cdot h}{L}$$

$$\alpha_1 := 0.6 + 0.5\cdot\left(\frac{2\cdot kh}{\sinh(2\cdot kh)}\right)^2$$

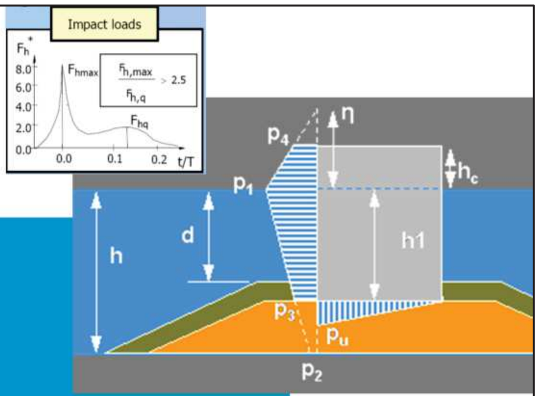
$$\alpha_2 := \min\left[\frac{h_{break}-d}{3\cdot h_{break}}\cdot\left(\frac{H_{design}}{d}\right)^2,\ \frac{2\cdot d}{H_{design}}\right]$$

$$\alpha_3 := 1 - \frac{h_1}{h_s}\cdot\left(1-\frac{1}{\cosh(kh)}\right)$$



Quasi-static wave forces:

Goda (extended) = Takahashi



$$\eta := 0.75\cdot(1+\cos(\beta))\cdot\lambda_1\cdot H_{design}$$

$$p_3 := \alpha_3\cdot p_1$$

$$p_4 := \mathrm{if}\left[\eta > h_c,\ p_1\cdot\left(1-\frac{h_c}{\eta}\right),\ 0\cdot Pa\right]$$

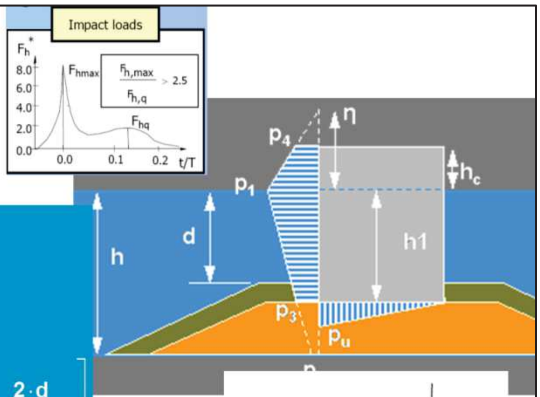
$$pu := 0.5\cdot(1+\cos(\beta))\cdot\lambda_3\cdot\alpha_1\cdot\alpha_3\cdot\rho_w\cdot g\cdot H_{design}$$

$$p_1 := \frac{1}{2}\cdot(1+\cos(\beta))\cdot\left[\lambda_1\cdot\alpha_1+\lambda_2\cdot\alpha_x\cdot(\cos(\beta))^2\right]\cdot\rho_w\cdot g\cdot H_{design}$$

β = angle between wave direction and normal to breakwater axis

Quasi-static wave forces:

Extended Goda = Takahashi



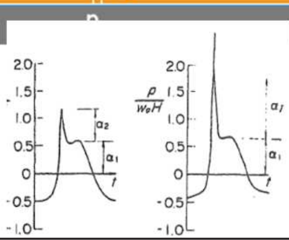
$$kh := \frac{2\cdot\pi\cdot h}{L}$$

$$\alpha_1 := 0.6+0.5\cdot\left(\frac{2\cdot kh}{\sinh(2\cdot kh)}\right)^2$$

$$\alpha_2 := \min\left[\frac{h_{break}-d}{3\cdot h_{break}}\cdot\left(\frac{H_{design}}{d}\right)^2,\ \frac{2\cdot d}{H_{design}}\right]$$

$$\alpha_x := \max(\alpha_2, \alpha_i)$$

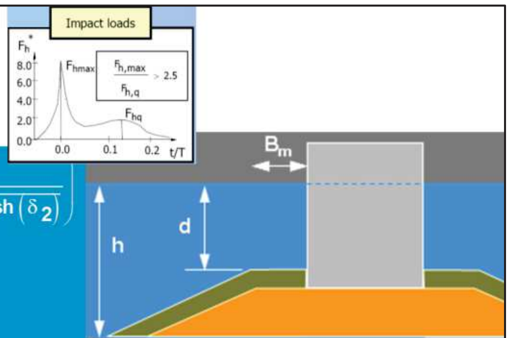
$$\alpha_3 := 1-\frac{h_1}{h_s}\cdot\left(1-\frac{1}{\cosh(kh)}\right)$$



Quasi-static wave forces:

Extended Goda = Takahashi

Impulsive Pressure Coefficient α_i



$$\alpha_{i1} := \mathrm{if}\left(\delta_2 \le 0,\ \frac{\cos(\delta_2)}{\cosh(\delta_1)},\ \frac{1}{\cosh(\delta_1)\cdot\sqrt{\cosh(\delta_2)}}\right)$$

$$\alpha_{i0} := \mathrm{if}\left(H \le 2\cdot d,\ \frac{H}{d},\ 2\right)$$

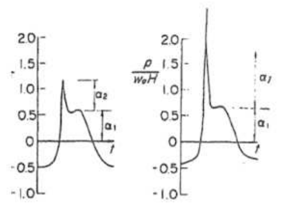
$$\alpha_i := \alpha_{i0}\cdot\alpha_{i1}$$

$$\delta_{11} := 0.93\cdot\left(\frac{B_m}{L}-0.12\right)+0.36\cdot\left(\frac{h-d}{h}-0.6\right)$$

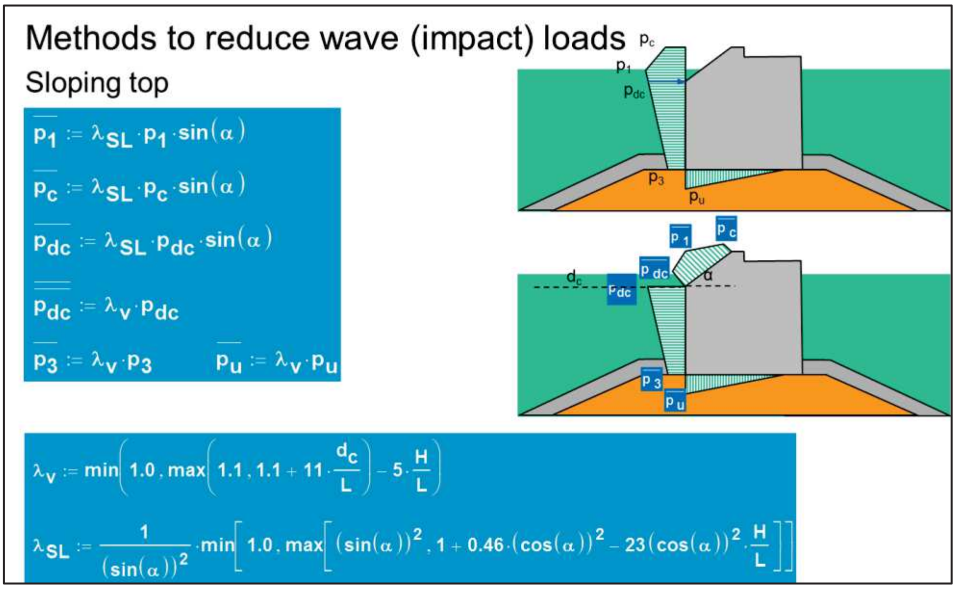
$$\delta_{22} := -0.36\cdot\left(\frac{B_m}{L}-0.12\right)+0.93\cdot\left(\frac{h-d}{h}-0.6\right)$$

$$\delta_1 := \mathrm{if}(\delta_{11} \le 0,\ 20\cdot\delta_{11},\ 15\cdot\delta_{11})$$

$$\delta_2 := \mathrm{if}(\delta_{22} \le 0,\ 4.9\cdot\delta_{11},\ 3\cdot\delta_{22})$$



減少波力衝擊的方法(Takahashi):



(四)模型試驗:

製作縮小尺寸之模型於波浪水槽中測試，驗證設計結果。

甲、模型比例因子之決定:

i. 比例因子盡可能取大值。

ii. 與試驗水槽之高度、可操作的水深、可產生之波高、試驗水槽之長度等有關。

乙、計算模型之尺寸:

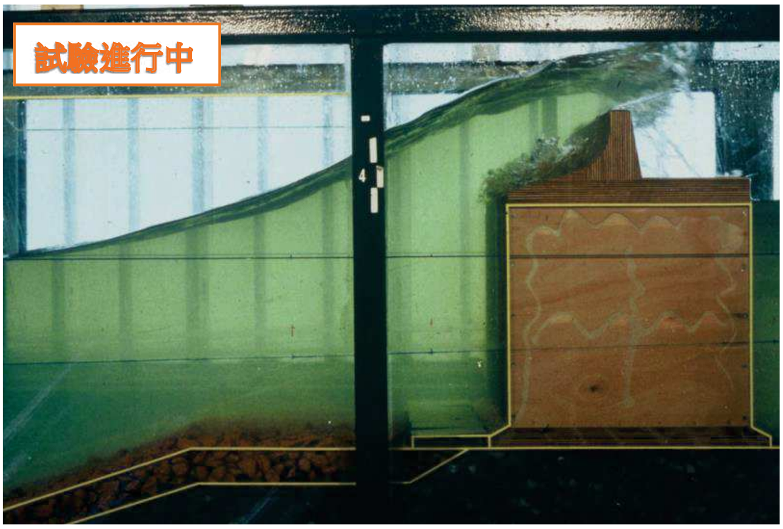
防波堤各設計尺寸乘以比例因子即可得到模型的尺寸。

丙、模型之製作與佈置:

依計算所得之模型尺寸製作模型，並將其佈置於試驗水槽中。

丁、試驗之安排:

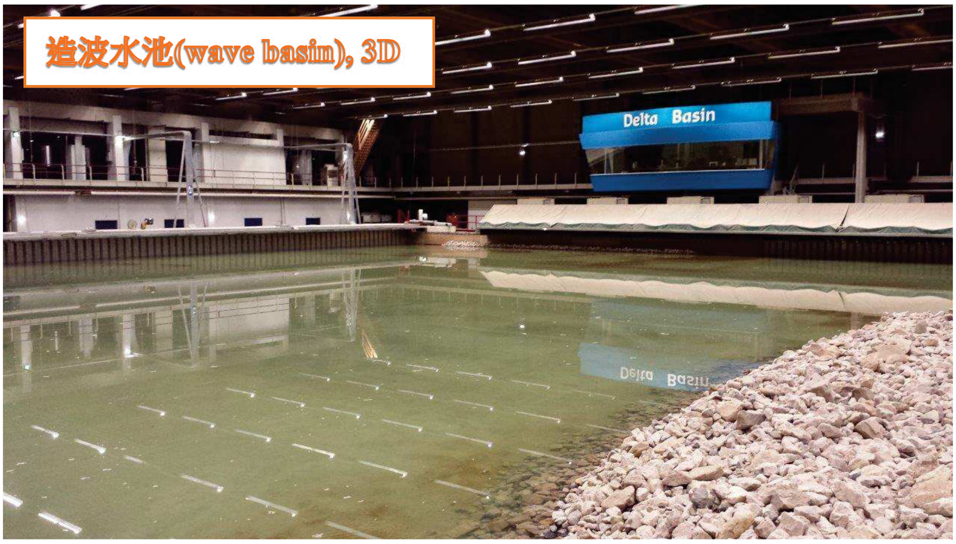
模型佈置完成後進行試驗。



戊、試驗設備之介紹:

i. 2D 波浪水槽(wave flume)

ii. 3D 造波水池(wave basin)



五.上課方式:

本課程屬該機構碩士學程課程之一，與碩士班學生一起上課。上課方式: 由授課教授講解課程內容，每單元結束後有習題演練，期末需完成一份專題報告(內容需包含: 依給定的基本資料及防波堤型式，如何決定設計邊界條件，如何設計出防波堤斷面尺寸，如何決定模型比例因子、計算模型尺寸並安排模型試驗)，並上台簡報及答辯。本次課程分組完成之期末專題報告如附件 1。

本次課程中，每節課程皆準時出席上課，且完成教授指定之習題及期末報告，並完成簡報，因此順利取得結業證書，如附件 2。

參、心得與建議

一、本次三周的課程，屬碩士學程正式學分課程，內容充實緊湊，上課方式亦相當嚴謹，除教授講解外，另有隨堂習題演練，期末並須分組撰寫專題報告，並上台簡報及答辯，算是強度頗高的課程，但也因增加學習成效。很高興本次能順利完成本課程，增進了個人專業上之知識。

二、課程中提供多種專業資料庫、設計手冊、相關參考資料等線上免費資源，有助於自行研讀或業務上有需求時之查詢。

三、本次課程獲得之教材及相關資訊，彙整後可供內部相關同仁參考。

四、與來自不同國家、不同文化背景的夥伴合作撰寫報告，學習溝通與協調、堅持與妥協、領導與被領導，是難得的經驗。

五、本次課程內容充實，惟無實際參訪行程。若能輔以實際工程或試驗室之參訪，相信更能增進對課程內容之理解。建議爾後挑選課程時，在可能的情況下，可考量此點。

附件1

Exercise Report

DESIGN OF BERM BREAKWATER – MASS ARMOURED TYPE

FOR

IHE-PORT

MARCH 2017

Mohammed Abdullah Al Ghailani

Tony Wu

Salim Al Breiki

INTRODUCTION

This report is part of an exercise given during short course programme on coastal and port structures at UNESCO-IHE institute held on March, 2017.

The exercise is to design the trunk-section of the eastern breakwater for IHE-Port, suggest scale and test programme and to give a presentation on the classroom.

IHE-Port shall be constructed about 3 km offshore on a more or less north-south situated coast and is open for waves from easterly directions. An entrance channel leads to the port. The breakwater connected to the shore by a pile founded bridge. The eastern part of the breakwater is located at a depth of -14 m CD.

Berths are foreseen behind the breakwater for export of minerals as well as a small container area directly behind the eastern breakwater. The port consists of two breakwaters, a 1300 m long eastern breakwater and a 900 m long southern breakwater. The most important function of the breakwaters is to provide a sufficient level of tranquility in the offshore port basin. The first 5 m of soil underneath the breakwater is soft silty material. Beyond that level, there is good quality sand present.

Based on given conditions and selected type of breakwater to be designed, the report content as follows.

1. Description of boundary conditions.
2. Design of cross-section of the trunk.
3. Physical model testing.

The group of participants on this short course and involved on this exercise have selected the berm breakwater MA type to design the cross-section. The berm breakwater is chosen because of the availability of the rock in nearby query. Moreover, the berm breakwater is easy for construction as it doesn't require big equipment.

1. Description of boundary conditions:

The main boundary conditions are illustrated in the following chart.

Design wave height H_{SD}= 5.4 m (for 100-years return period, Figure 1.)	Peak period T_P=12 s	Overload H_S=6.2 m (for 500-years return period, Figure 1.)
Seabed at toe -14 m CD	Density of water 1026 kg/m^3	Mass Density of Rock 2690 kg/m^3

The water level conditions:	
MSL	+0.65 m CD
MLWS	-0.2 m CD
MLW	+0.2 m CD
MHW	+1.2 m CD
MHWS	+1.8 m CD

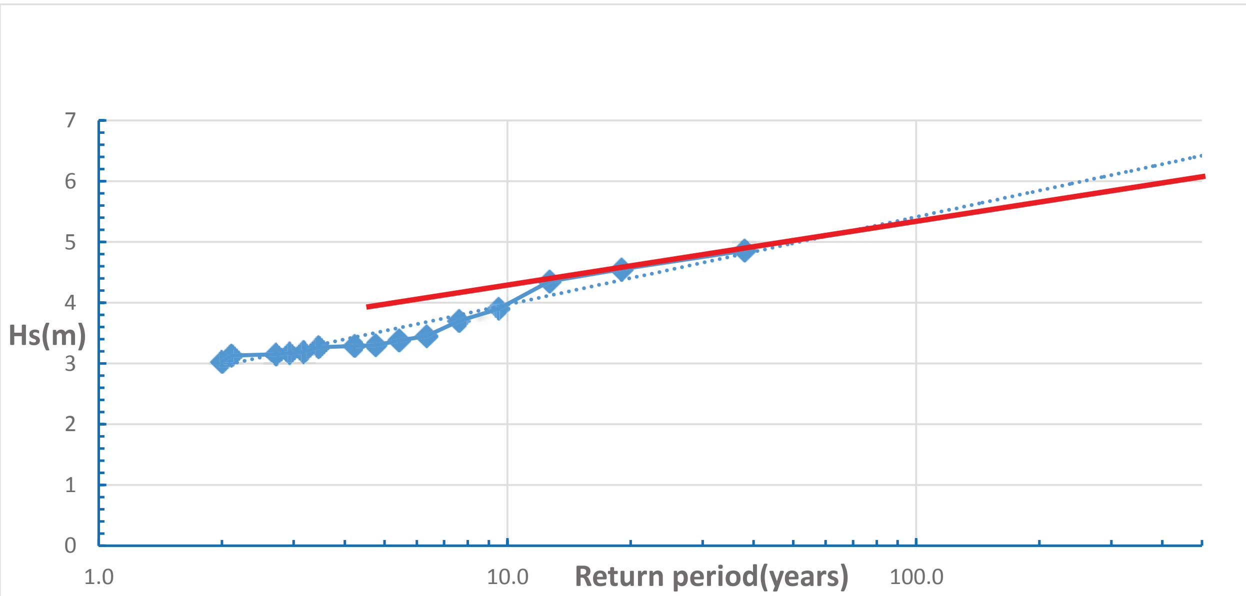


Figure 1. Wave return period curve for 500 years return period.

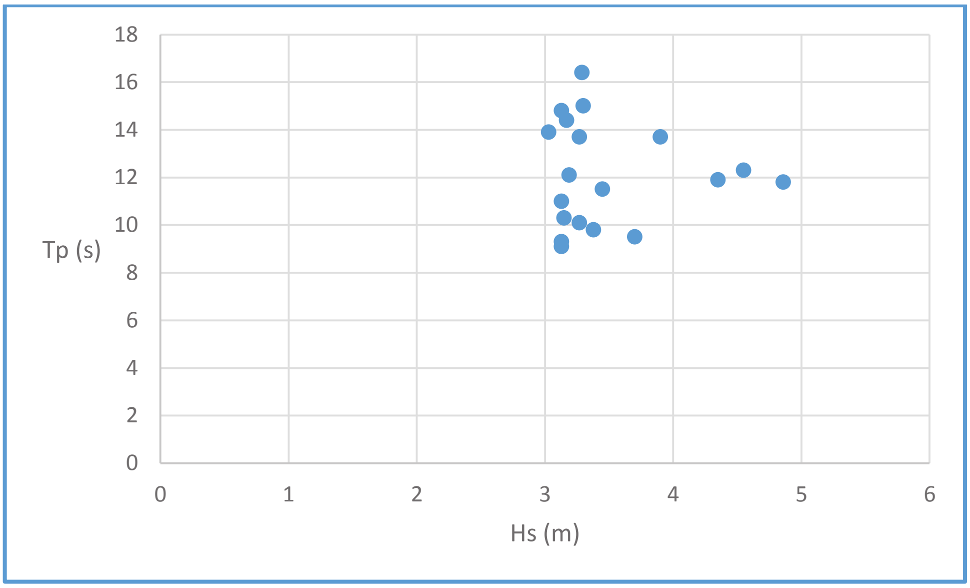


Figure 2. Wave height VS wave period.

Since the highest waves come with wave periods around 12 s (Figure 2.), Tp is determined as 12 s for the first design, then 10 s and 14 s are also considered for comparison.

Considering that there is no surge, the design water level could be just a little above the maximum tide, the design water level of 2.1 m CD is determined in this design. For construction work safety, there would be a Δw above MHWS. The Δw =0.5 m is chosen for this design because the overtopping is not greater than 1 l/s per m at this level.

The rock classes that selected for this design are class I (1-5 tone) and class II (5-10 tone). The reason of this selection is the availability of these rock classes in the nearby quarry.

The slope of the structure is designed to be 1:1.5

Table 1. Summary of design conditions:

Parameter	Value
DWL	+2.1 m CD
Allowable overtopping q for H_{SD} (100 years)	1 l/s per m
Allowable overtopping q for overload (500 years)	10 l/s per m
Wanted resiliency	35 %
cotα	1.5
Δw	0.5 m
Rock class I	5-10 t
Rock class II	1-5 t

2. Design of cross-section of the trunk:

To see how the wave period affects the design result, we use the design spreadsheet to calculate the crest level with conditions determined in Section 1., but in 3 different wave periods. From the results (shown in Table 2.), the crest level is dominated by H_{sD}, the wave period is relatively influenced the crest level as illustrated below. The conceptual design will be continued with an average wave period which is 12s.

Table 2. Crest level calculations.

Method used	Crest level (m)
Determined by $R_c/H_{sD}=1.2^*$	8.6
Determined by wave overtopping formulae** with Tp=10 s	1.04
Determined by wave overtopping formulae** with Tp=12 s	2.22
Determined by wave overtopping formulae** with Tp=14 s	2.93

*: Equation 5.6 in Design and Construction of Berm Breakwaters, by Van der Meer and Sigurdarson.

**: Equation 4.13 in Design and Construction of Berm Breakwaters, by Van der Meer and Sigurdarson.

Since the general design conditions are all determined, we can input them into the design spreadsheet, then the design parameters are calculated. According to these calculated parameters, we make some design choices. The input and outcome of the design spreadsheet are shown as Figure 3., Figure 4., Figure 5. and Figure 6.

General conditions		
Design wave height H_{sD}	5.4	m
Peak period T_p	12	s
Overload H_s	6.2	m
Design water level DWL	2.1	m CD
Lowest water level with H_{sD}	-0.2	m CD
Lowest storm level	-0.2	m CD
H_s at lowest storm level	5.2	m
Mean High Water Spring	1.8	m CD
Bottom level of foreshore at toe of structure	-14	m CD
Allowable overtopping q for H_{sD}	1	l/s per m
Allowable overtopping q for overload	10	l/s per m
Mass density water	1026	kg/m^3
Mass density rock	2690	kg/m^3
Choice of rock classes		
Rock Class I: minimum mass (0-15%)	5	t
Rock Class I: maximum mass (85-100%)	10	t
Rock Class II: minimum mass (0-15%)	1	t
Rock Class II: maximum mass (85-100%)	5	t
Rock Class III: M_{min} (leave open for MA)		t
Rock Class III: M_{max} (leave open for MA)		t

Figure 3. Input of the design spreadsheet.

Outcome main parameters		
Wave steepness s_{op}	0.024	-
Relative mass density Δ	1.62	-
Median mass Class I M_{50}	7.5	t
Nominal diameter Class I D_{n50}	1.41	m
Stability number $H_{sD}/\Delta D_{n50}$	2.37	-
Type of berm breakwater	**Partly reshaping**	
Number of rock classes for berm	2	
Basic recession for H_{sD} (no adaptation)	4.91	m
Recession for overload (no adaptation)	7.80	m
Nominal diameter Class II, D_{n50}	1.04	m
Nominal diameter Class III, D_{n50}	o Class III	
Resiliency, berm width and level		
Wanted resiliency	35	%
Resulting Berm width B from resiliency	14.02	m
Minimum berm width B_{min} from geometry	6.32	m
Berm level 0.6 H_{sD}	5.34	m CD
Δw for waves during construction	0.5	m
MHWS plus Δw = working level	2.3	m CD
Minimum berm level from construction	5.11	m CD
Design choice of berm width	14.00	m
Design choice of berm level	6.00	m CD
Required horizontal armour width A_h	25.5	m
Design choice of A_h	26.0	m

Minimum transition level to class II			
For H_{sD} at lowest level	-2.4	m CD	
For lowest level with according H_s	-2.28	m CD	
Design choice of transition for IC (3 rock classes)	-3.32	m CD	
Transition lower class for MA (2 rock classes)	-3.32	m CD	
Crest level (γ_β = 1)			
If no overtopping criteria, $R_{c\,min}$	8.6	m CD	
If no overtopping criteria, $R_{c\,max}$	9.7	m CD	
For given allowable overtopping, q, γ_{BB}	0.01		
Required crest level for design conditions	2.22	m CD	
Required crest level for overload	2.21	m CD	
Design choice of crest level	10.00	m CD	
Check possibility of toe berm at level h_t			
Lowest possible toe level (two layers)	-10.43	m CD	
Design conditions			
Allowable damage level for H_{sD}, N_{od}	2	-	
Highest level of toe for H_{sD} with chosen N_{od}	-6.93	m CD	
Check validity range h_t/D_{n50}	6.5		ok
Check validity range h_t/h	0.49		ok
Overload conditions			
Allowable damage level for overload, N_{od}	4	-	
Highest level of toe for overload with chosen N_{od}	-7.21	m CD	
Check validity range h_t/D_{n50}	6.8		ok
Check validity range h_t/h	0.51		ok
Design choice of toe berm level (0 if no berm)	0	m CD	
Design choice cotα core below A_h	1.5	-	

Figure 4. Outcome of the design spreadsheet.

Summary of design choices		
Design of berm width	14.00	m
Design of berm level	6.00	m CD
Design of A_h	26.0	m
Design of transition class I to class II	-3.3	m CD
Design of crest level	10.00	m CD
Design of toe berm level	0.00	m CD
Design choice cotα core below A_h	1.50	-

Rock Class I	5	-	10	t
Rock Class II	1	-	5	t
Rock Class II	0	-	0	t

Figure 5. Summary of design choices.

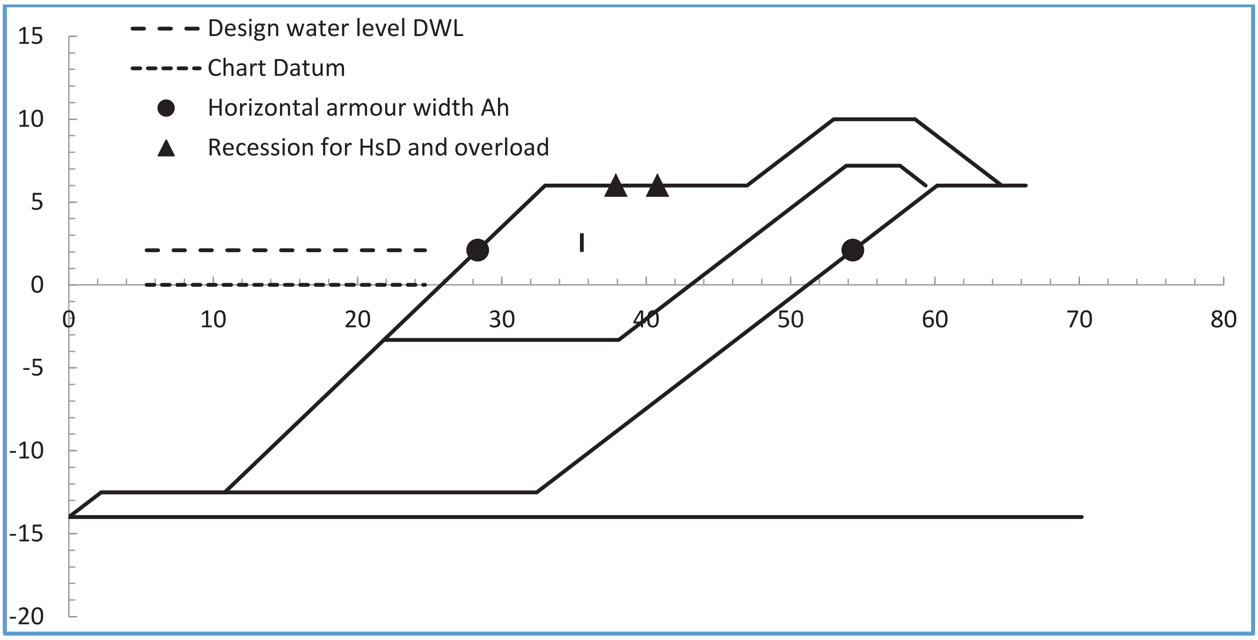


Figure 6. Cross-section from design spreadsheet.

Finally, concluding with design conditions, calculated parameters and made design choices, the berm breakwater cross-section for this design was drawn as Figure 7.

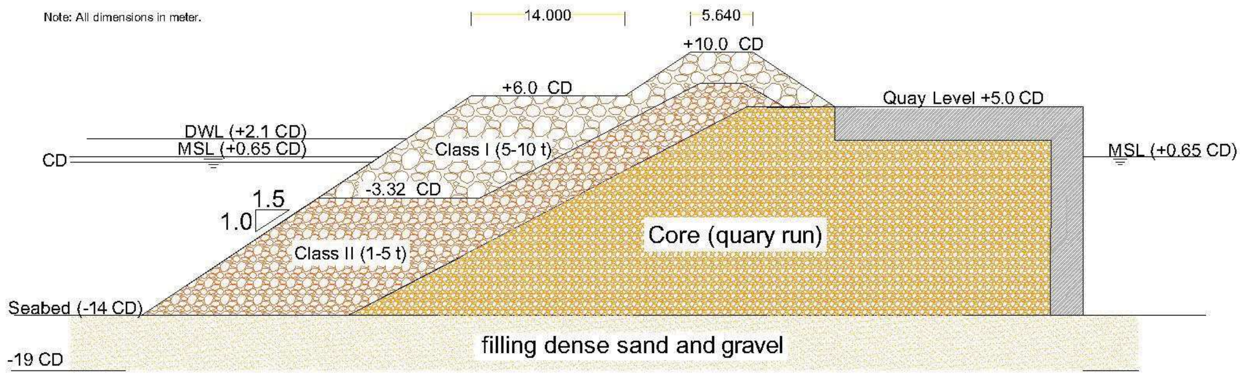


Figure 7. Final cross-section of the structure for the designed berm breakwater.

Concerning the bearing capacity of the seabed underneath the structure, the pressure of the structure on the seabed soil is estimated to be 400 kPa, it is far greater than the bearing capacity of the existing soil (soft silty material). In this case, soil improvement is required. One could directly put rocks onto the soft soil and let rocks settle down, but this would cost more rocks. Besides, one could dredge the soft soil, then directly put rocks to fill (also cost more rocks) or fill by sands. Since the construction site is near the harbor, assume the dredger is available. In this condition, dredging the soft soil and filling by sands could be an economical solution.

3. Physical model testing:

Prototype:

Hs overload= 6.2m

Hs design= 5.4m

M (class I)= 5-10 tone

M (class II)= 1-5 tone

Tp= 12s

Model:

Scaling factor Hs for the overload design / Hs that the model can generate

6.2/0.25 = 24.8, (*Hs of the model is the limiting factor*)

Then the scaling factor that will be used is 25

Hs overload= 6.2/25= 0.248m

Hs design= 5.4/25= 0.216m

M (class I)= 0.032 – 0.64 kg

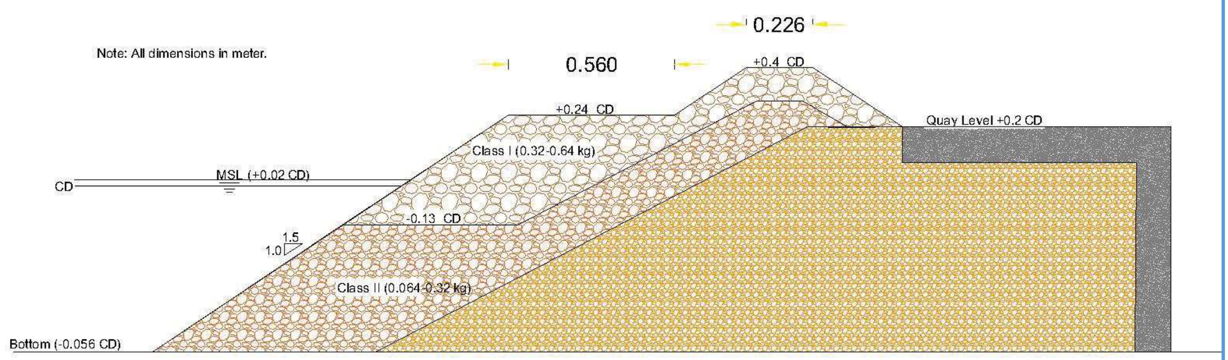
M (class II)= 0.064 – 0.032 kg

Tp= 2.4s

	Prototype	**Model**
Hs overload	6.2m	0.248m
Hs design	5.4m	0.216m
M (class I)	5-10 tone	0.032 – 0.64 kg
M (class II)	1-5 tone	0.064 – 0.032 kg
Tp	12	2.4s

The overtopping will be measured by putting a box behind the model and weight the overtopped water.

Cross-section of Model Testing:



Testing Program

The suggested testing program will be as following:

- Model with significant Hs=5.4m for Tp=10s, 12s, 14s
- Model with overload Hs=6.2m for Tp=10s, 12s, 14s

Conclusion

In this report, a design for berm breakwater has been made. The design has been made based on the boundary conditions of the significant wave height recorded and period. A cross-section for the breakwater has been made. In addition, improvement techniques have been suggested to treat the soft soil in the location. In order to verify the design, a model test has been made to be conducted in a flume. The test will give more confident on the design of this berm breakwater.

附件2

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CERTIFICATE

Short Course
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This is to certify that

Wu Ming Tung

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has followed and successfully completed the short course
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from 06 March 2017 – 24 March 2017.

E.L. Ploeger, MSc
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Delft, the Netherlands, 24 March 2017