

## Design of low-crested (submerged) structures – an overview –

Krystian W. Pilarczyk, Rijkswaterstaat, Road and Hydraulic Engineering Division,  
P.O. Box 5044, 2600 GA Delft, the Netherlands; [k.w.pilarczyk@dww.rws.minvenw.nl](mailto:k.w.pilarczyk@dww.rws.minvenw.nl)

### 1. Introduction

Wave climate, in combination with currents, tides and storm surges, is the main cause of coastal erosion problems. Various coastal structures can be applied to solve, or at least, to reduce these problems. They can provide direct protection (breakwaters, seawalls, dikes) or indirect protection (offshore breakwaters of various designs), thus reducing the hydraulic load on the coast (Figure 1).

Low crested and submerged structures (LCS) such as detached breakwaters and artificial reefs are becoming very common coastal protection measures (used alone or in combination with artificial sand nourishment). Their purpose is to reduce the hydraulic loading to a required level that maintains the dynamic equilibrium of the shoreline. To attain this goal, they are designed to allow the transmission of a certain amount of wave energy over the structure by overtopping and also some transmission through the porous structure (exposed breakwaters) or wave breaking and energy dissipation on shallow crest (submerged structures).

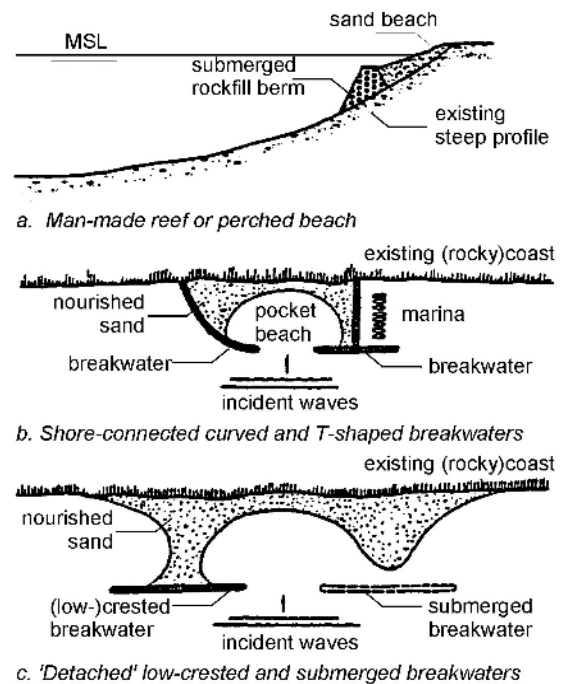


Figure 1. Examples of low-crested structures

Owing to aesthetic requirements, low freeboards are usually preferred (freeboard around SWL or below). However, in tidal environments and when frequent storm surges occur these become less effective if designed as narrow-crested structures. This is also the reason why broad-crested submerged breakwaters (also called-, artificial reefs) became popular, especially in Japan (Figure 2, Yoshioka et al., 1993). However, broad-crested structures are much more expensive than narrow-crested ones and their use should be supported by proper cost-benefit-studies. The development of alternative materials and systems, for example, the use of sand-filled geotubes as a core of such structures, can effectively reduce the cost (Pilarczyk, 1996, 1999).

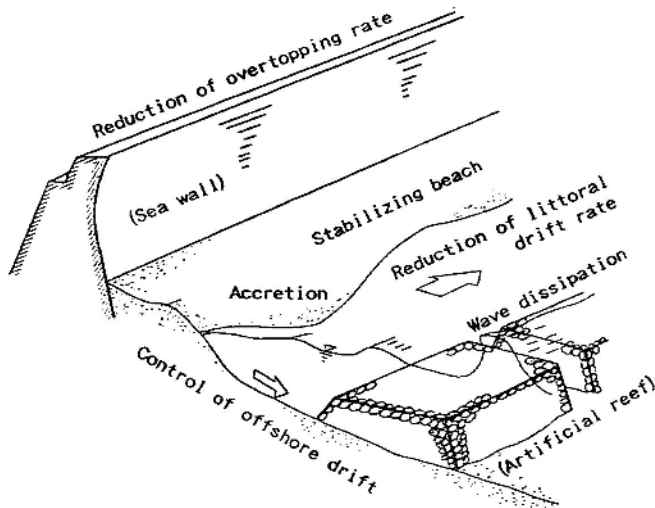


Figure 2. Objectives of artificial reefs (Yoshioka et al., 1993)

This paper provides an overview of literature and design tools relating to or used in the design of low-crested and submerged structures. Special attention is paid to Japanese literature (design guidelines and experience) which is less known outside Japan. Some recent examples of low-crested structures (artificial reefs) and alternative designs are also presented.

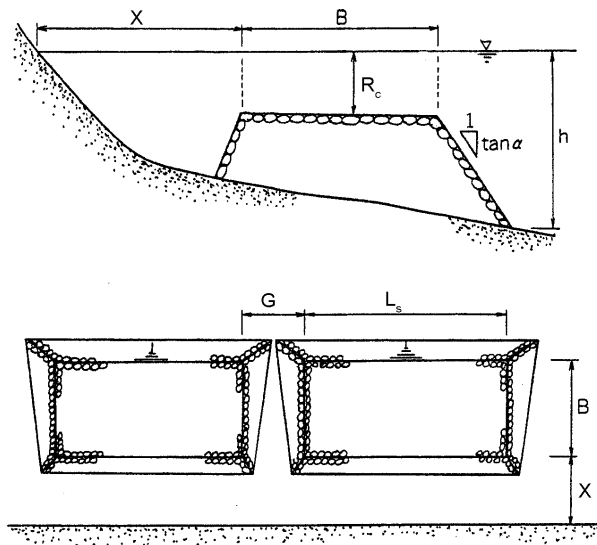


Figure 3. Definitions for submerged structures

The following design aspects for exposed and submerged structures are treated in more detail:

- transmission characteristics (including some prototype data)
- functional design (lay-out and rules)
- stability of rock and geosystems

Usually, offshore breakwaters, and especially, the low-crested submerged structures, provide environmentally friendly coastal solutions. However, high construction cost and the difficulty of predicting the response of the beach are the two main disadvantages that inhibit use of offshore breakwaters. It should be noted that the low-crested structures could be used not only for shoreline control but also to reduce wave loading on the coastal structures (including dunes) and properties.

For shoreline control the final morphological response will result from the time-averaged (i.e. annual average) transmissivity. However, to simulate this in the designing process, for example, in numerical simulation, it is necessary to know the variation in the transmission coefficient for various submergence conditions. Usually when there is need for reduction in wave attack on structures and properties the wave reduction during extreme conditions (storm surges) is of interest (reduction of wave pressure, run up and/or overtopping). In both cases the effectiveness of the measures taken will depend on their capability to reduce the waves.

While considerable research has been done on shoreline response to exposed offshore breakwaters, very little qualitative work has been done on the effect of submerged offshore reefs, particularly outside the laboratory (Black&Mead, 1999). Therefore, the main purpose of this paper is to provide information on wave transmission for low-crested structures and to refer the reader to recent literature.

## 2. Wave transmission over the low-crested structures

Shoreline response to an offshore breakwater is controlled by at least 14 variables (Hanson and Kraus, 1989, 1990, 1991), of which eight are considered primary; (1) distance offshore; (2) length of the structure; (3) transmission characteristics of the structure; (4) beach slope and/or depth at the structure (controlled in part by the sand grain size); (5) mean wave height; (6) mean wave period; (7) orientation angle of the structure; and (8) predominant wave direction. For segmented detached breakwaters and artificial reefs, the gap between segments becomes another primary variable.

The efficiency of submerged structures (reefs) and the resulting shoreline response mainly depends on transmission characteristics and the layout of the structure. A number of engineering procedures to estimate combined wave transmission through a breakwater and wave overtopping are available, but still not very reliable (Tanaka, 1976, Ahrens, 1987, Uda, 1988, Van der Meer, 1990, d'Angremond-vdMeer-de Jong, 1996, Seabrook et al, 1998, etc). The new approach to the definition of transmission over and through the structure can be found in (Wamsley & Ahrens (2003).

### 2.1. Wave transmission in scale models; definitions and results

The transmission coefficient,  $K_t$ , defined as the ratio of the height directly shoreward of the breakwater to the height directly seaward of the breakwater, has the range  $0 < K_t < 1$ , for which a value of 0 implies no transmission (high, impermeable), and a value of 1 implies complete transmission (no breakwater). Factors that control wave transmission include crest height and width, structure slope, core and armour material (permeability and roughness), tidal and design level, wave height and period.

As wave transmission increases, diffraction effects decrease, thus decreasing the size of a salient through direct attack by the transmitted waves and weakening the diffraction-current moving sediment into the shadow zone (Hanson and Kraus, 1991). It is obvious that the design rules for submerged

structures should include a transmission coefficient as an essential governing parameter. Some of the methods to determine the transmission over the submerged structures will be discussed below.

The first complete set of transmission characteristics for exposed and submerged breakwaters/reefs were presented by Tanaka (1976) and Uda (1988), this being within the scope of the preparation of Japanese Manual on Artificial Reefs (Yoshioka et al., 1993). These graphs are based on tests with regular waves and expressed in deep water wave parameters. It is useful to include these graphs because they present the general tendency of variation of transmission within a wide range of conditions. The graphs show that wave steepness has also influence on transmission.

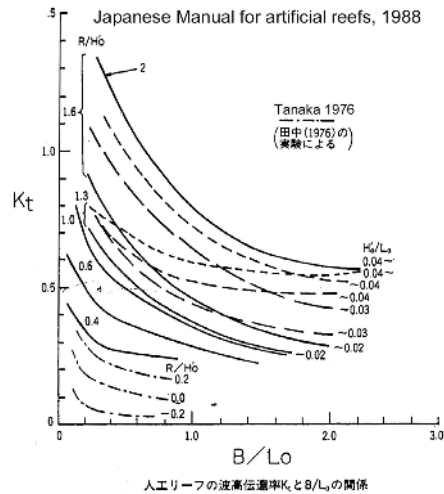
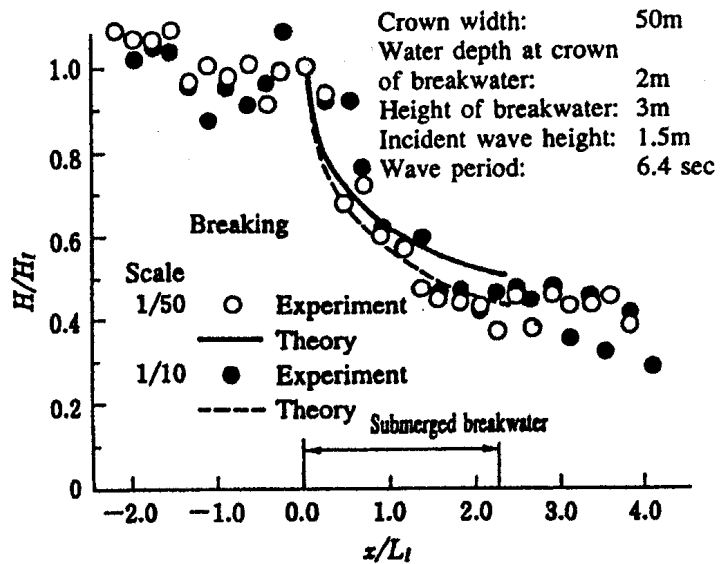
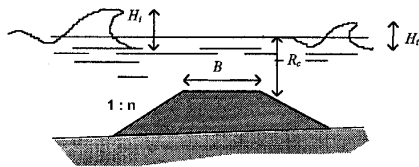


Figure 4. Transmission characteristics for artificial reefs (Uda, 1988)



Note: in the Figure 5,  $L_i = T(g R_c)^{0.5} \sim 28\text{m}$ , but  $B$  (op x-axis) is the width at the bottom  $\sim 66\text{m}$  (crest + slopes =  $50 + 16$ ), therefore  $B/L_i$  is about 2.3.

Figure 5. Wave reduction caused by a submerged structure (Sawaragi, 1995);  $R_c/H_i = 2/1.5 = 1.33$ ,  $B/L = 1.77$  for the crest and 2.3 for the bottom width)

An interesting investigation into the effect of wave breaking and wave transformation on the artificial reef was performed at Osaka University (Sawaragi, 1992, 1995). An example of the results is presented in Figure 5, where both, experimental and analytical results are presented. It was found that wave transformation initiated by submerged breakwaters could be predicted analytically by using the expression proposed by Sawaragi et al. (1989), even in the case where the forced wave breaking takes place on the breakwater. These results agree closely with the test results of Delft Hydraulics presented in Table 1.

#### Physical modelling of wave transmission by submerged breakwaters for AmWaj Island (Delft Hydraulics, 2002)

The Amwaj Islands Development Project in Bahrain involves a new island on the existing coral reef (Fowler et al., 2002). To protect the waterfront developments on this island from wave attack, a scheme that uses submerged breakwaters has been planned. These, should also function as the anchor for a sandy (-artificial-) beach, preventing the sand from being washed out into the sea. The main technical aspects studied in a physical model were the amount of wave transmission over the breakwaters (important for the beach stability analysis) and the stability of the armour layer on the breakwaters. The results of wave transmission tests are summarized in Table 1. In this table  $H_{si}$  is the incoming wave height at the toe of the breakwater. In most tests this wave height was nearly equal to that generated at the foreshore. Only a few points with lower water level conditions showed a slight reduction ( $<5\%$ ) in wave height in front of the breakwater, indicating that the effect of shoaling on the 1/50 foreshore was minimal. In front of the breakwater local depth is  $h$ .

The results of wave transmission are presented as  $K_t=H_{st}/H_{si}$  as function of the relative freeboard ( $R_c/H_{si}$ ) and the relative crest length ( $B/L$ ). The wavelength  $L$  was determined on the crest  $L_c= T_p(gR_c)^{0.5}$ , at the toe,  $L_h= T_p(gR_c)^{0.5}$ , and at deepwater  $L_o=gT^2/2\pi$ . By using these three definitions of  $L$  it is possible to make a comparison with various current presentations in the literature. In the table the calculated values of  $K_t$  applying the formula of d'Angremond-vdMeer-de Jong (1996) based on  $L_c$ -definition (instead of  $L_o$ ) and using two numerical coefficients ( $C=0.64$  for permeable- and  $C=0.80$  for impermeable- structures) are also presented. The agreement between the measured and calculated results is relatively good (Figure 6b).

Table 1 Results of transmission tests and calculated values (acc. to d'Angremond-vdMeer-deJong, '96)

Test	B(m)	h(m)	H <sub>si</sub> (m)	T <sub>p</sub> (s)	R <sub>c</sub> (m)	L <sub>c</sub> (m)	L <sub>h</sub> (m)	R <sub>c</sub> /H <sub>si</sub>	B/L <sub>c</sub>	B/L <sub>h</sub>	B/L <sub>o</sub>	K <sub>t</sub>	K <sub>t(C1-C2)</sub>
101	50	5.8	2.37	7.98	-2.7	41.0	58.4	-1.14	1.22	0.86	0.50	0.56	0.58-0.61
102	50	4.7	2.50	7.98	-1.6	31.7	53.2	-0.64	1.56	0.94	0.50	0.36	0.37-0.39
103	40	5.8	2.49	7.97	-2.7	40.9	58.3	-1.08	0.98	0.69	0.40	0.58	0.57-0.60
104	40	4.7	2.54	8.02	-1.6	31.9	53.5	-0.63	1.26	0.75	0.40	0.37	0.37-0.40
105	30	5.8	2.48	7.91	-2.7	40.6	57.8	-1.09	0.74	0.52	0.31	0.62	0.60-0.62
106	30	4.7	2.34	8.01	-1.6	31.8	53.4	-0.68	0.94	0.56	0.30	0.46	0.41-0.44
107	20	5.8	2.45	8.01	-2.7	41.2	58.6	-1.10	0.49	0.34	0.20	0.68	0.60-0.65
108	20	4.7	2.33	8.00	-1.6	31.8	53.3	-0.69	0.63	0.38	0.20	0.53	0.42-0.46
201	50	5.8	2.51	7.95	-2.0	35.3	58.1	-0.80	1.42	0.86	0.51	0.42	0.44-0.46
202	50	4.7	2.50	8.03	-0.9	23.9	53.6	-0.36	2.09	0.93	0.50	0.24	0.24-0.27
203	40	5.8	2.49	8.01	-2.0	35.5	58.6	-0.80	1.13	0.68	0.40	0.46	0.45-0.48
204	40	4.7	2.54	8.02	-0.9	23.9	53.5	-0.35	1.67	0.75	0.40	0.28	0.25-0.28
205	30	5.8	2.48	7.91	-2.0	35.1	57.8	-0.81	0.86	0.52	0.31	0.49	0.46-0.49
206	30	4.7	2.34	8.01	-0.9	23.9	53.4	-0.39	1.26	0.56	0.30	0.33	0.27-0.30
207	20	5.8	2.45	8.01	-2.0	35.5	58.6	-0.82	0.56	0.34	0.20	0.56	0.48-0.52
208	20	4.7	2.33	8.00	-0.9	23.9	53.3	-0.39	0.84	0.38	0.20	0.40	0.29-0.32
301	40	5.8	1.84	7.19	-2.0	31.8	51.7	-1.09	1.26	0.77	0.50	0.53	0.56-0.59
302	40	5.8	2.43	8.03	-2.0	35.6	58.8	-0.82	1.12	0.68	0.40	0.47	0.45-0.49
303	40	4.7	1.85	7.23	-0.9	21.6	47.7	-0.49	1.86	0.84	0.49	0.29	0.32-0.35
304	40	4.7	2.32	7.97	-0.9	23.8	53.1	-0.39	1.68	0.75	0.40	0.31	0.26-0.29

Note: Toplayer:  $D_{n50}=0.62m$  for tests 101-206 and  $0.50m$  for tests 301-304; the core consists of sand-filled geotubes;  $K_{t(C1-C2)}$ : calculated with Formula (d'Angremond-vdMeer-de Jong, 1966) with coef.  $C_1=0.64$  and  $C_2=0.80$ , respectively, and  $L_c$  (instead of  $L_o$ ); seaward slope 1 on 3, bottom slope 1 on 50 (see also Figure 6a).

The original formula of d'Angremond&Van der Meer &de Jong (1996) for exposed and submerged structures reads:

$$K_t = -0.4R_c/H_i + (B/H_i)^{-0.31} [1 - \exp(-0.5\xi)] C \quad \text{and} \quad \xi = \tan\alpha / (H_i/L_o)^{0.5} \quad (1)$$

Application of Seabrook&Hall formula (1998):

$$K_t = 1 - \{ \exp[-0.65(R_c/H_i) - 1.09(H_i/B)] + 0.047[BR_c/(L D_{n50})] - 0.067[R_c H_i / (B D_{n50})] \} \quad (2)$$

where  $D_{n50}$ = equivalent stone diameter,  $L$ = wavelength, provides less agreement (the calculated values are 0.15 to 0.20 lower than measured ones).

Example of application in AmWaj project is shown in Figure 6a.

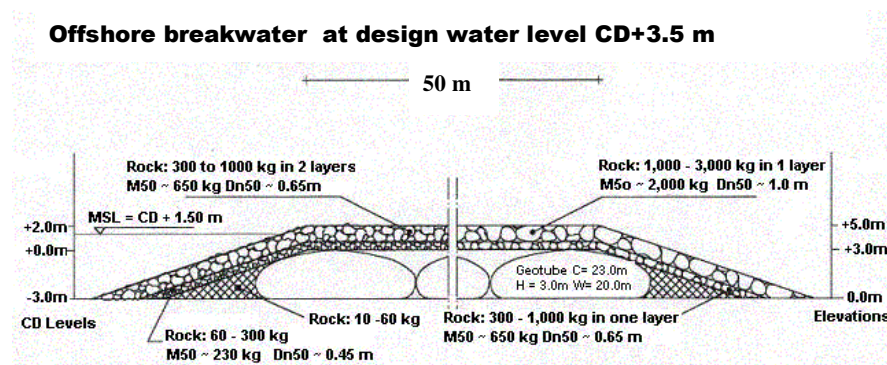


Figure 6a. Example of offshore reef breakwater for AmWaj project

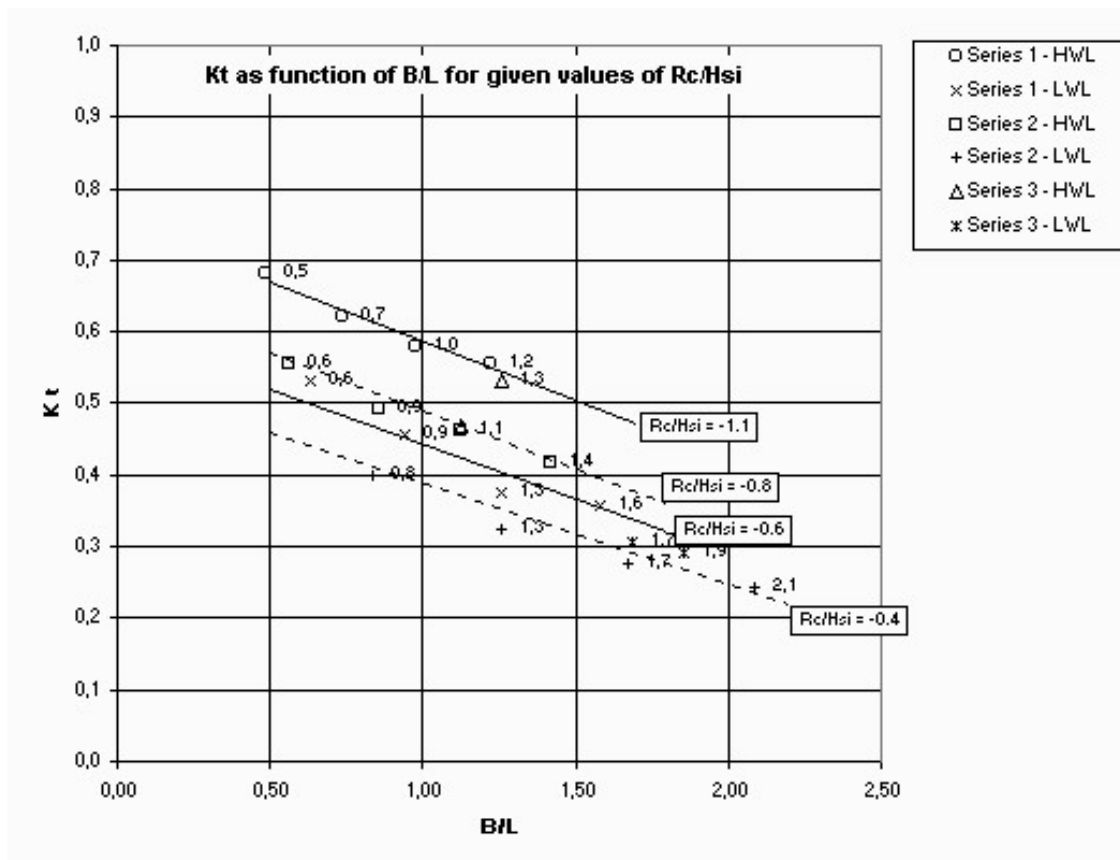
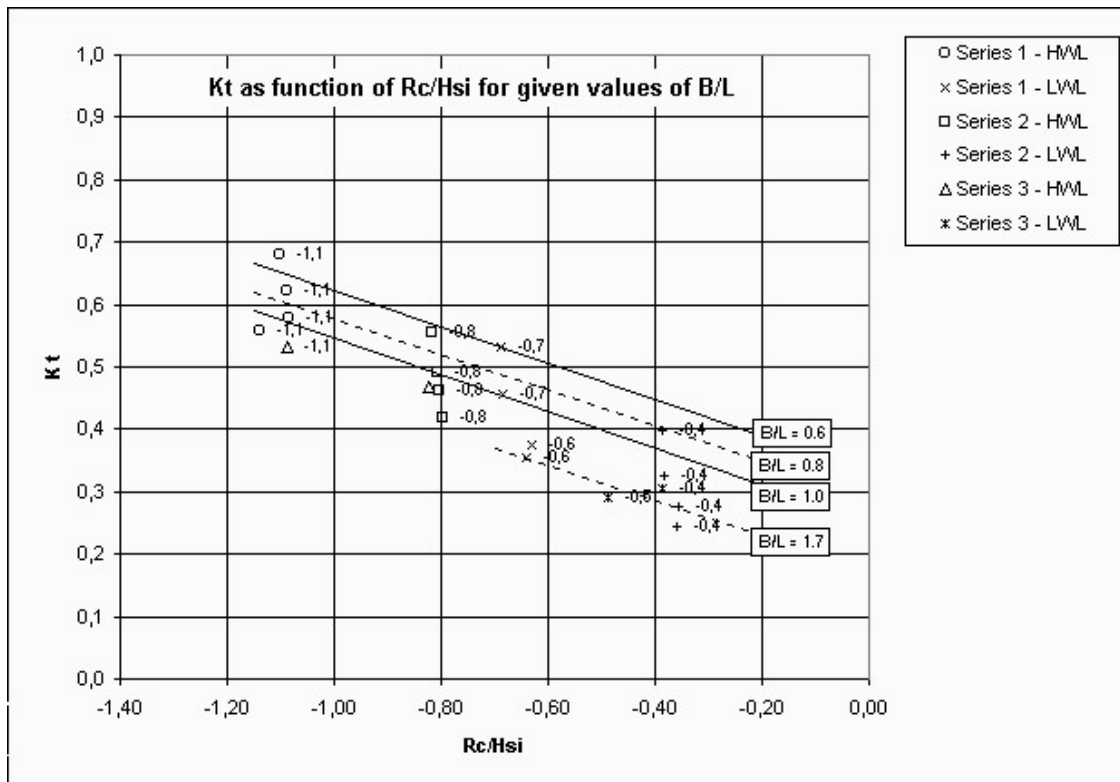
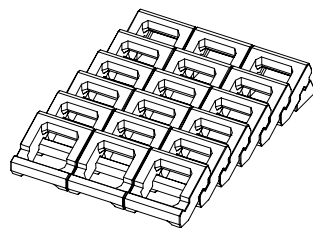


Figure 6b. Transmission results of model investigation for reef structures (Delft Hydraulics, 2002)

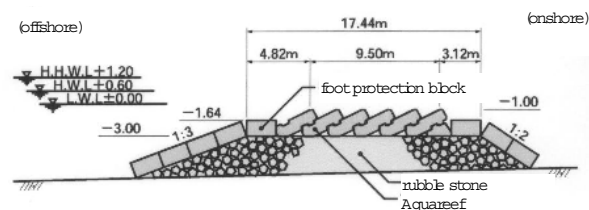
*Model tests Aquareef; Tetra Co, Japan (Hirose et al., 2002)*

In recent years in Japan much more attention has been paid to environmental aspects of coastal protection (Nakayama, 1993). This has resulted in the development of more friendly artificial reefs creating better conditions for the marine environment. An example of such a structure is Aquareef, which is protected by Aqua blocks (Figure 7). The first developments were reported by Asakawa and Hamaguchi in 1991 in a paper in which the transmission characteristics with regular waves were presented. More detailed descriptions of the functional and technical design of these reefs can be found in (Hirose et al., 2002). Development of this block and reef structure was supported recently by an extensive model investigation (with random waves) related to transmissivity and stability aspects. Both aspects were tested in a wide range of wave and submergence conditions, as is evident from the transmission graphs in Figure 7. This figure shows the relation between the wave height transmission coefficient  $H_t/H_{1/3}$  and the relative wave length  $B/L_{1/3}$ , where  $H_t$  is the transmitted wave height recorded on the landward side,  $H_{1/3}$  and  $L_{1/3}$  are the significant wave height and wavelength at the toe of the rubble mound, and  $B$  is the crown width of the units. A number of these reefs have already been constructed and some experience of their functioning has been gained.

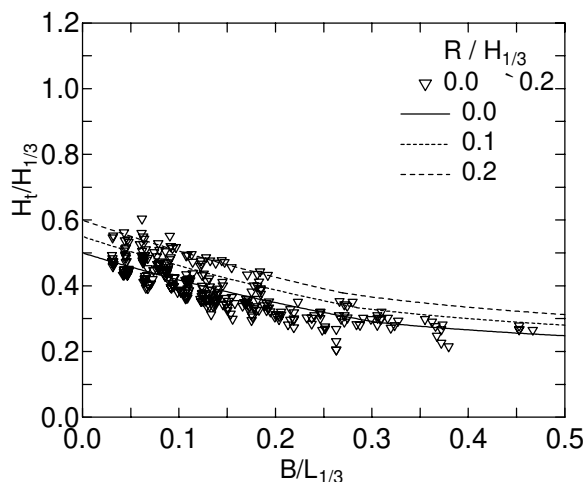
Note: there is a good agreement between these data and those of Delft Hydraulics (see Table 1); small deviations can be explained by differences in surface roughness and the permeability of the core.



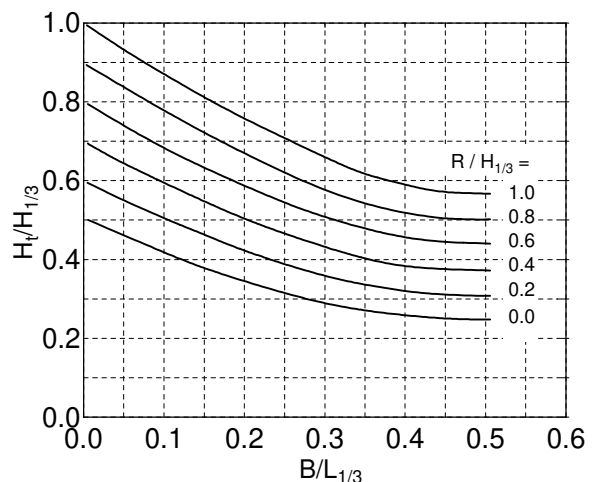
Aqua blocks



Example of the cross section of the reef constructed at Onishika beach



Transmission results for water levels close to the crest



General transmission characteristic

Figure 7. Wave transmission characteristics; example of measured data, general transmission characteristic and of the cross section of the artificial reef constructed at Onishika beach (Hirose et al., PIANC 2002);  $B$ =crest width,  $R$ =submergence (freeboard below SWL)

*2.2. Prototype measurements - examples from Japan*

The construction of detached breakwaters and, especially, artificial reefs (= submerged breakwaters with broad crest) is very popular and advanced in Japan. Their application had already started in the 70-'s, supported by extensive model studies. The design techniques were gradually improved by using the results of a large number of prototype measurements and by monitoring completed projects. Because of the limited space only some results are presented and no comments are given. More information and also the results of morphological responses can be found in the original papers.

In general, the Japanese structures are placed closer to the shore than the distance observed in U.S. and European projects, often resulting in tombolos, generally undesirable for the more common coastal projects, except for pocket beach design. The Japanese design procedure can be found in Uda (1988) and Yoshioka (1993).

Yugawara, Japan (Ohnaka and Yoshizwa, 1994, Aono and Cruz, 1996)

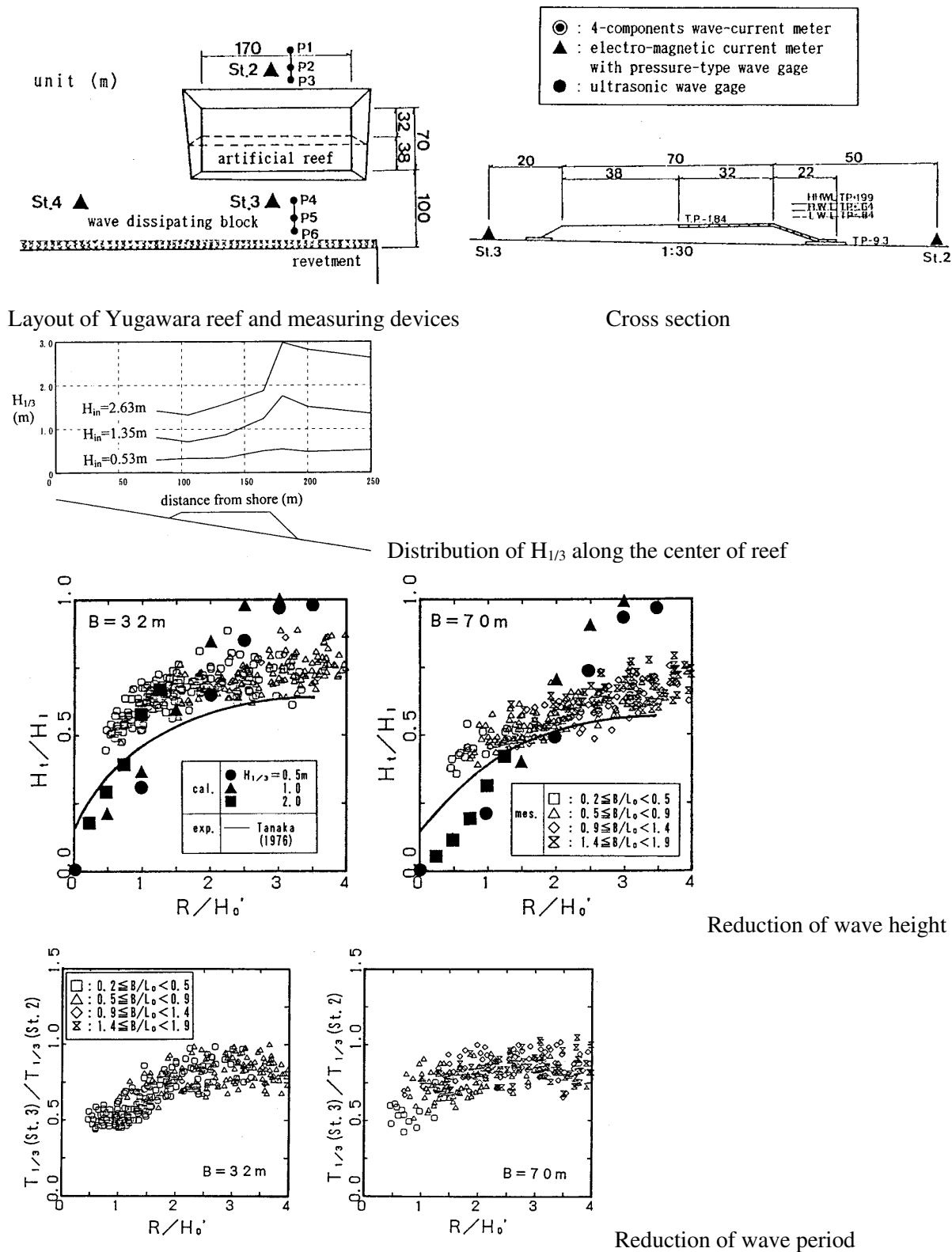
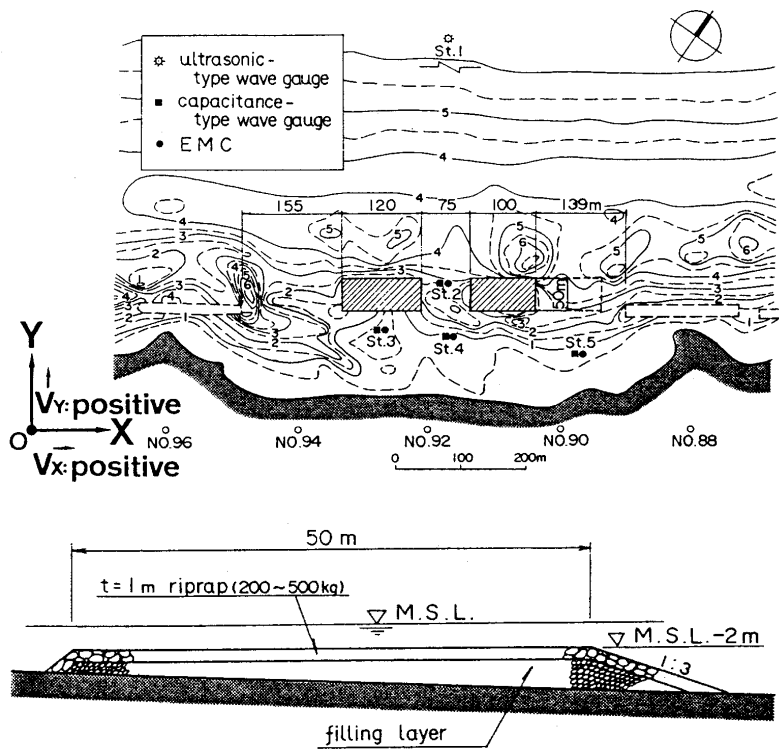
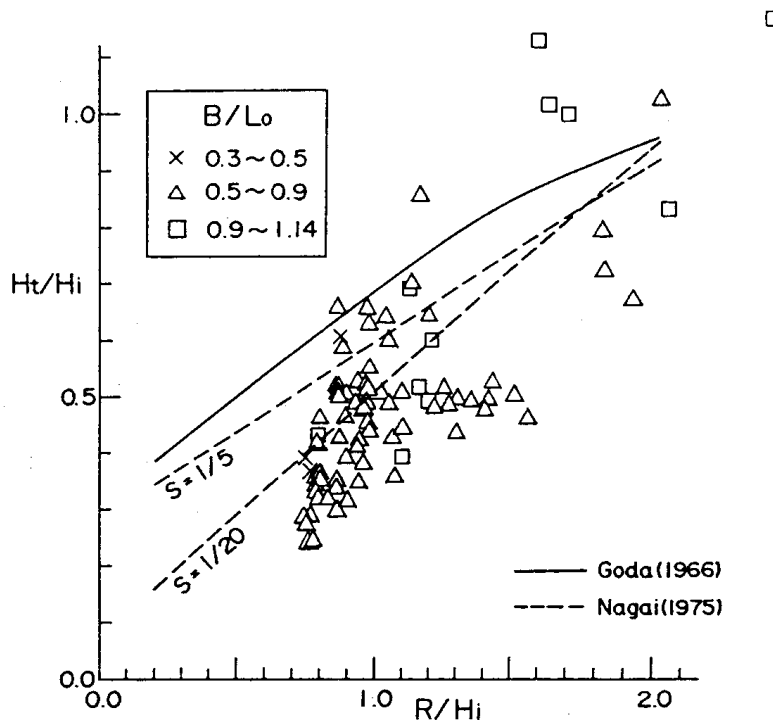


Figure 8. Prototype measurements for Yugawara reef, Japan

Niigata Reef (1), prototype measurements (Hamaguchi et al., 1991)



- Situation and measuring points

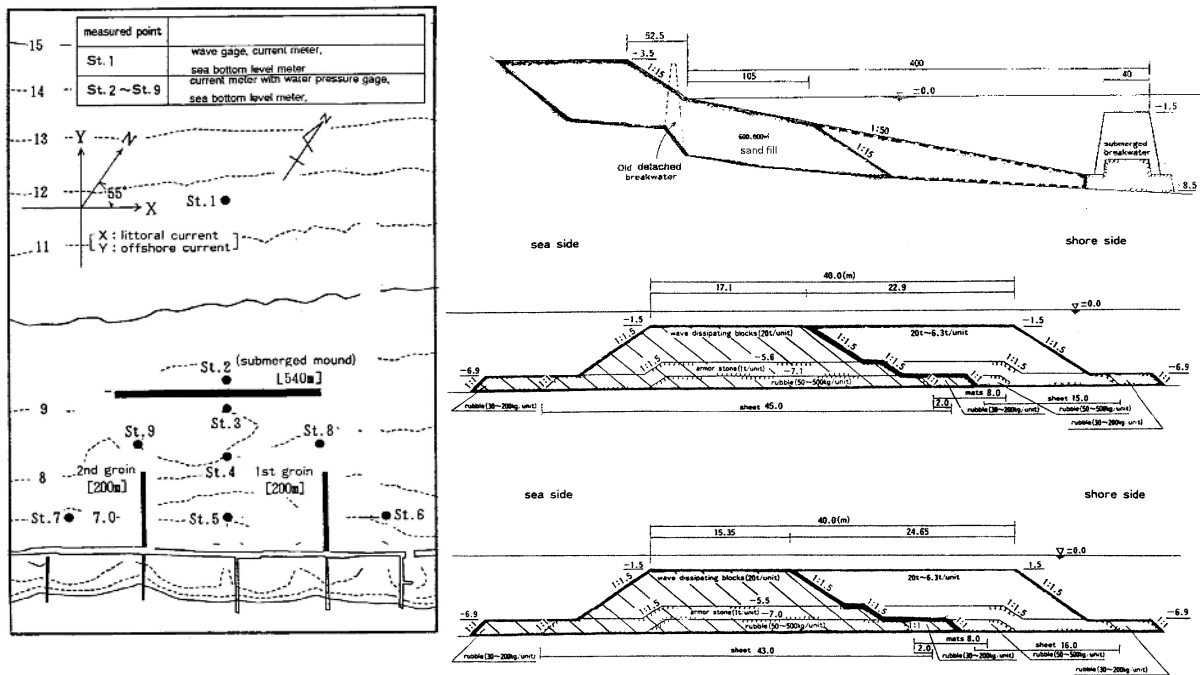


Relation between wave transmission coefficient  $H_t/H_i$  and the relative crest depth  $R/H_i$  of the reef

Figure 9. Prototype measurements for Niigata reef (1), Japan





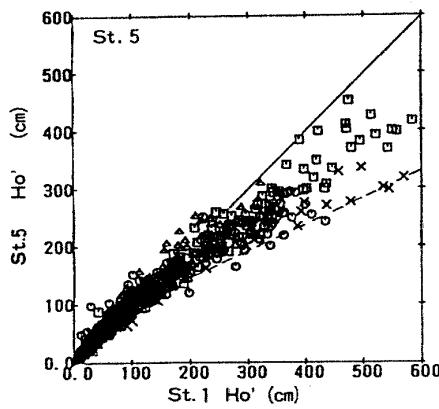
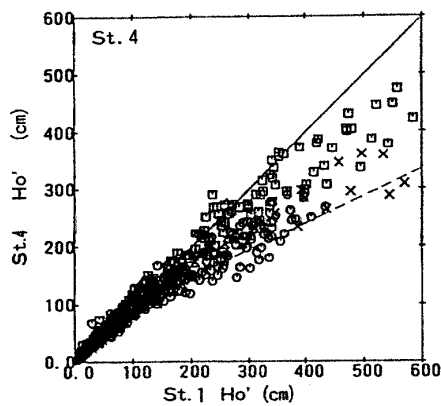
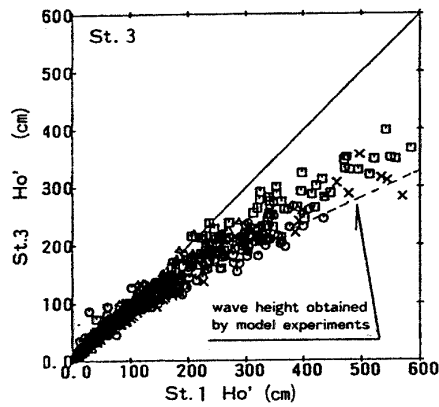
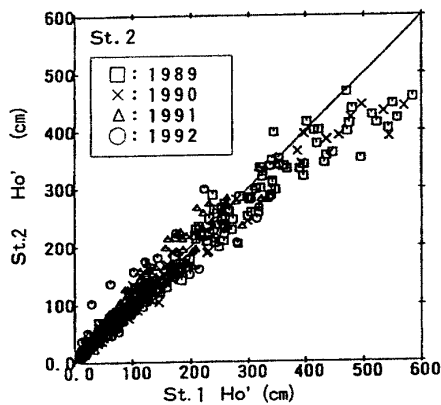
Niigata Reef (2), prototype measurements (Funakoshi et al., 1994)



- Situation and measuring points

- Cross-sections

 provisional cross section  
 complete



- Wave height correlations (transmission)

Figure 10. Prototype measurements for Niigata reef (2), Japan

### 3. Layout and morphological response

Most commonly an offshore obstruction, such as a reef or island, will cause the shoreline in its lee to protrude in a smooth fashion, forming a salient or a tombolo. This occurs because the reef reduces the wave height in its lee and thereby reduces the capacity of the waves to transport sand. Consequently, sediment moved by longshore currents and waves builds up in the lee of the reef (Black, 2001). The level of protection is governed by the size and offshore position of the reef, so the size of the salient or tombolo varies in accordance with reef dimensions. Of course, one can expect this kind morphological change only if the sediment is available (from natural sources or as sand nourishment).

The examples of simple geometrical empirical criteria for the lay-out and shoreline response of the detached, exposed (emerged) breakwaters are given below (i.e., Harris & Herbich, 1986, Dally & Pope, 1986, etc.):

$$\text{- for tombolo formation: } L_s/X > (1.0 \text{ to } 1.5) \quad (3a)$$

$$\text{- for salient formation: } L_s/X = (0.5 \text{ to } 1.0) \quad (3b)$$

$$\text{- for salients where there are multiple breakwaters: } G X/L_s^2 > 0.5 \quad (3c)$$

Where  $L_s$  is the length of a breakwater and  $X$  is the distance to the shore,  $G$  is the gap width (see Figure 3), and the transmission coefficient  $K_t$  is defined for annual wave conditions.

A more complete review of these criteria can be found in US Corps, 1993 and Pilarczyk&Zeidler (1996). These geometrical criteria do not include the transmission; however, the transmission coefficients  $K_t$  for exposed breakwaters are usually in the range 0.1 to 0.3.

To include the effect of submergence (transmission) Pilarczyk proposes, at least as a first approximation, adding the factor  $(1-K_t)$  to the existing rules. Then the rules for low-crested breakwaters can be modified to (for example):

$$\text{Tombolo: } L_s/X > (1.0 \text{ to } 1.5)/(1-K_t) \quad \text{or} \quad X/L_s < (2/3 \text{ to } 1) (1-K_t), \quad \text{or} \quad X/(1-K_t) < (2/3 \text{ to } 1) L_s \quad (4a)$$

$$\text{Salient: } L_s/X < 1/(1-K_t) \quad \text{or} \quad X/L_s > (1-K_t), \quad \text{or} \quad X/(1-K_t) > L_s \quad (4b)$$

$$\text{For salients where there are multiple breakwaters: } G X/L_s^2 > 0.5(1-K_t) \quad (4c)$$

The gap width is usually  $L \leq G \leq 0.8 L_s$ , where  $L$  is the wavelength at the structure defined as:  $L = T (g h)^{0.5}$ ;  $T$  = wave period,  $h$  = local depth at the breakwater.

One of the first properly documented attempts to obtain criteria for detached breakwaters including transmissivity was made by Hanson and Krause (1989,1990), see Figure 11. Based on numerical simulations (Genesis model) and some limited verification from existing prototype data, they developed the following criteria for a single detached breakwater:

$$\text{- for a salient: } L_s/L \leq 48 (1 - K_t) H_o/h \quad (5a)$$

$$\text{- for a tombolo: } L_s/L \leq 11 (1 - K_t) H_o/h \quad (5b)$$

Where  $L_s$  = length of the structure segment (breakwater),  $X = n h$  = distance from the original shoreline ( $n$ = bottom gradient),  $h$  = depth at the breakwater,  $H_o$  = deepwater wave height,  $L$  = wave length at the breakwater.

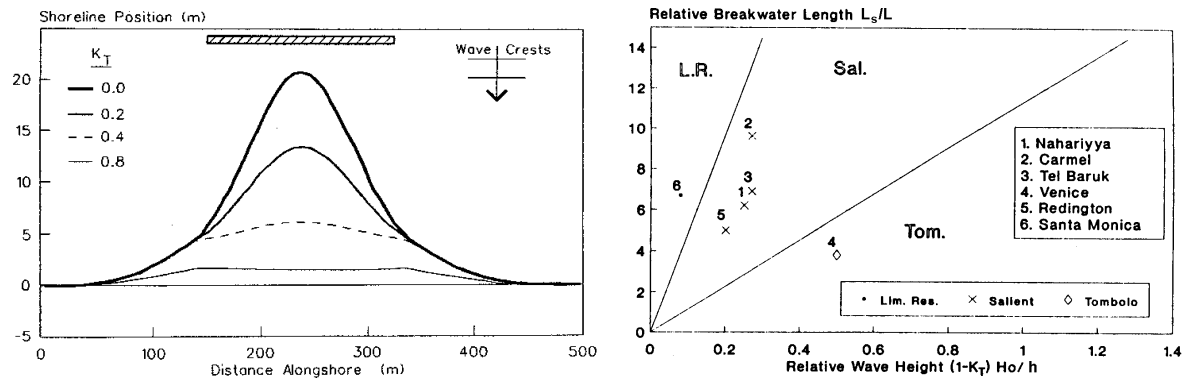


Figure 11. Numerical example of shoreline response as a function of transmission and verification of proposed criteria according to Hanson & Kraus, 1990.

These criteria can be used as preliminary design criteria for distinguishing shoreline response to a single, transmissive detached breakwater. However, the range of verification data is too small to permit the validity of this approach to be assessed for submerged breakwaters. Actually, a similar approach is used for the submerged breakwaters within the scope of the European project DELOS (Jimenez and Sanches-Arcilla, 2002). In general, it can be stated that numerical models (i.e., Genesis, Delft 2D-3D, Mike 21, etc.) can already be treated as useful design tools for the simulation of morphological shore response to the presence of offshore structures. Examples can be found in (Hanson & Krause, 1989, 1991, Groenewoud et al. 1996, Bos et al., 1996, Larson et al., 1997, Zyserman et al., 1999).

As mentioned above, while considerable research has been done on shoreline response to exposed offshore breakwaters, very little qualitative work has been done on the effect of submerged offshore reefs, particularly outside the laboratory. Thus, within the Artificial Reefs Program (Black & Mead, 1999) ([www.asrltd.co.nz](http://www.asrltd.co.nz)), Andrews (1997) examined aerial photographs seeking cases of shoreline adjustment to offshore reefs and islands. All relevant shoreline features in New Zealand and eastern Australia were scanned and digitized, providing 123 different cases. A range of other statistics, particularly reef and island geometry, was also obtained. Some of these results are repeated below.

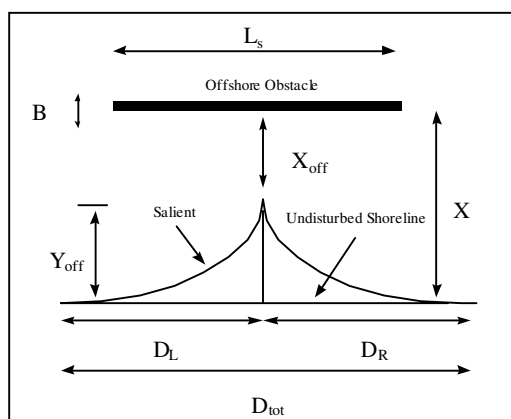


Figure a) Definitions

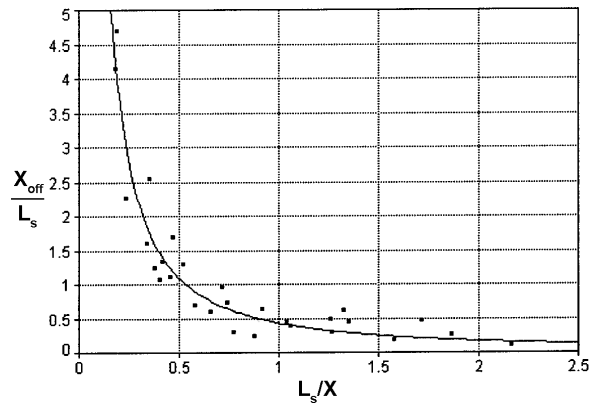


Figure b) Example of salient relation for reefs

Figure 12:  $X_{off}/L_s$  versus  $L_s/X$  for submerged offshore reefs, where  $X_{off}$  is the distance of the tip of the salient from the offshore reef,  $L_s$  is the longshore dimension of the reef and  $X$  is the distance of the reef from the undisturbed shoreline.

*Tombolo and Salient limiting parameters.* To examine the effects of wave transmission on limiting parameters, data for reefs and islands were considered separately. The data indicated that tombolo formation behind islands occurs with  $L_s/X$  ratios of 0.65 and higher and salients form when  $L_s/X$  is

less than 1.0. Therefore, for islands the  $L_s/X$  ratios determining the division between salients and tombolos are similar to those from previously presented breakwater research. Similarly, data resulting from offshore reefs indicate that tombolo formation occurs at  $L_s/X$  ratios of 0.6 and higher, and salients most commonly form when  $L_s/X$  is less than 2. The data suggests that variation in wave transmission (from zero for offshore islands through to variable transmission for offshore reefs) allows a broader range of tombolo and salient limiting parameters. Thus, a reef that allows a large proportion of wave energy to pass over the obstacle can be (or must be) positioned closer to the shoreline than an emergent feature.

Thus, from natural reefs and islands the following general limiting parameters were identified:

$$\text{Islands:} \quad \text{Tombolos form when } \frac{L_s}{X} > 0.65 \quad \text{Salients form when } \frac{L_s}{X} < 1.0 \quad (6a,b)$$

$$\text{Reefs:} \quad \text{Tombolos form when } \frac{L_s}{X} > 0.60 \quad \text{Salients form when } \frac{L_s}{X} < 2.0 \quad (7a,b)$$

Non-depositional conditions for both shoreline formations occur when  $L_s/X < \approx 0.1$ .

Andrews discovered that the size of salients (including length, offshore amplitude and shape) behind submerged reefs was predictable. For example, Fig. 12b shows that the distance between the tip of the salient and the offshore reef ( $X_{off}$ ) can be predicted from the longshore dimension of the offshore reef ( $L_s$ ) and its distance from the undisturbed shoreline ( $X$ ). The relationship defined by the data is not totally consistent with previous studies of offshore breakwaters. More detailed information, especially on coastal response, the geometry of salients, and comparison with literature can be found in Black&Andrews (2001) and on the website [www.asrltd.co.nz](http://www.asrltd.co.nz), where some examples from real projects are also presented.

To investigate the effects of wave transmission on salient amplitude, salient data of various types was analyzed separately for reefs and islands. Island data exhibits a power-curve relationship:

$$\frac{X_{off}}{L_s} = 0.40 \left( \frac{L_s}{X} \right)^{-1.52} \quad (\text{islands only}) \quad (8)$$

Reef data (Figure 11b) presents a power-curve relationship:

$$\frac{X_{off}}{L_s} = 0.50 \left( \frac{L_s}{X} \right)^{-1.27} \quad (\text{reefs only}) \quad (9)$$

From Equations 8 and 9 islands and reefs can be seen to have similar curve shapes, but the magnitudes and responses are different. Hsu and Silvester (1990) presented a similar relationship for single emergent breakwaters based on literature data (physical models, numerical models and some prototype data):

$$\frac{X}{B} = 0.68 \left( \frac{B}{S} \right)^{-1.22} \quad (\text{emergent breakwaters, Hsu\&Silvester}) \quad (10)$$

Comparison of the equations of Andrews with that of Hsu and Silvester suggests that the equations derived from natural conditions predict larger salient amplitude. Natural salients are assumed to be in equilibrium as their forms are a result of average wave hydrodynamics over long time periods, and they include all inputs (known and unknown) that shape and form salient formations. A number of other factors such as scale effects in the laboratory tests, insufficient directional spread, variability in natural cross-shore bathymetry, sediment grain sizes or tidal ranges may explain the difference.

It should be mentioned that recently Ming and Chiew (2000) published a paper on shoreline changes behind an exposed detached breakwater where the limit between tombolo and salient formation is defined at  $X/L_s = 0.8$  (salient  $X/L_s > 0.8$ ). They also provide the equation for the plan area of sand deposition ( $A$ ), namely the area enclosed by the initial shoreline and the shoreward equilibrium shoreline (the shoreline refers to the still water line):

$$\frac{A}{X^2} = -0.348 + 0.043 \frac{X}{L_s} + 0.711 \frac{L_s}{X} \quad (11)$$

It is also worth noting that Black and Mead (2001) have introduced a new concept of coastal protection by applying wave rotation due to the presence of submerged structures. Wave rotation targets the cause of the erosion, i.e. longshore wave-driven currents. Offshore, submerged structures are oriented to rotate waves so that the longshore current (and sediment transport) is reduced inshore. The realigned wave angle at the breaking point (in harmony with the alignment of the beach) results in reduced longshore flows and sediment accretion in the lee of the rotating reef.

The choice of the layout of submerged breakwaters can also be affected by the current patterns around the breakwaters. The Japanese Manual (1988) provides information on various current patterns for submerged reefs (Yoshioka et al., 1993). The principle schematisations are shown in Figure 13.

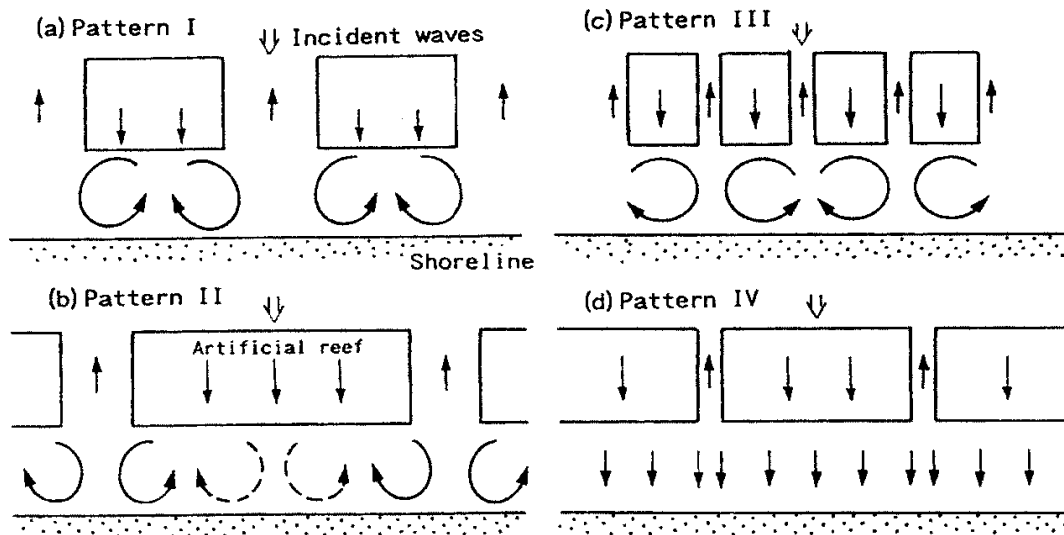


Figure 13. Flow pattern created by various spacing of breakwaters acc. to Japanese Manual (1988)

#### Reef Balls as alternative reefs

The relatively new innovative coastal solution is to use artificial reef structures called “Reef Balls” as submerged breakwaters, providing both wave attenuation for shoreline erosion abatement, and artificial reef structures for habitat enhancement. An example of this technology using patented Reef Ball™ is shown in Figure 14.



Figure 14. Individual Reef Ball™ Unit

Reef Balls are mound-shaped concrete artificial reef modules that mimic natural coral heads (Barber, 1999). The modules have holes of many different sizes in them to provide habitat for many types of marine life. They are engineered to be simple to make and deploy and are unique in that they can be floated to their drop site behind any boat by utilizing an internal, inflatable bladder. Stability criteria for these units were determined based on analytical and experimental studies. Some technical design aspects are treated in

publications by Harris, mentioned in references, which can be found on the website. Worldwide a large number of projects have already been executed by using this system. More information can be obtained from: [www.artificialreefs.org](http://www.artificialreefs.org) and [reefball@reefball.com](mailto:reefball@reefball.com).

#### 4. Remarks on stability aspects

Structural design aspects of low-crested structures are relatively well described in a number of publications (Ahrens, 1987, Uda, 1988, Van der Meer, 1987, 1988, CUR/CIRIA, 1991, US Corps, 1993, Pilarczyk&Zeidler, 1996, Vidal et al., 1992, 1998, etc). Some useful information on the design of breakwaters on reefs in shallow water can be found in Jensen et al. (1998). For example, as a

consequence of the depth-limited wave conditions on the reef, more frequently occurring wave conditions will impose almost the same wave impacts on the structure as rare events such as, for example, 25-year design conditions. This means that the damage induced by the 25-year condition outside the reef will also be induced by “normal” wave conditions with a return period of less than one year. Since the damage to the armour is cumulative, it is important to take the consequences of the depth-limited waves into consideration as appropriate design criterion for the damage to the armour (i.e., the number of destructive waves will be larger).

It can be noted that in the Japanese manual for artificial reefs (Uda, 1988), a method for stability calculation based on the velocity on the crest of the structure is presented. Another method can be found in (Hirose et al., 2002). Some examples of stability criteria for low-crested structures are shown in Figure 15.

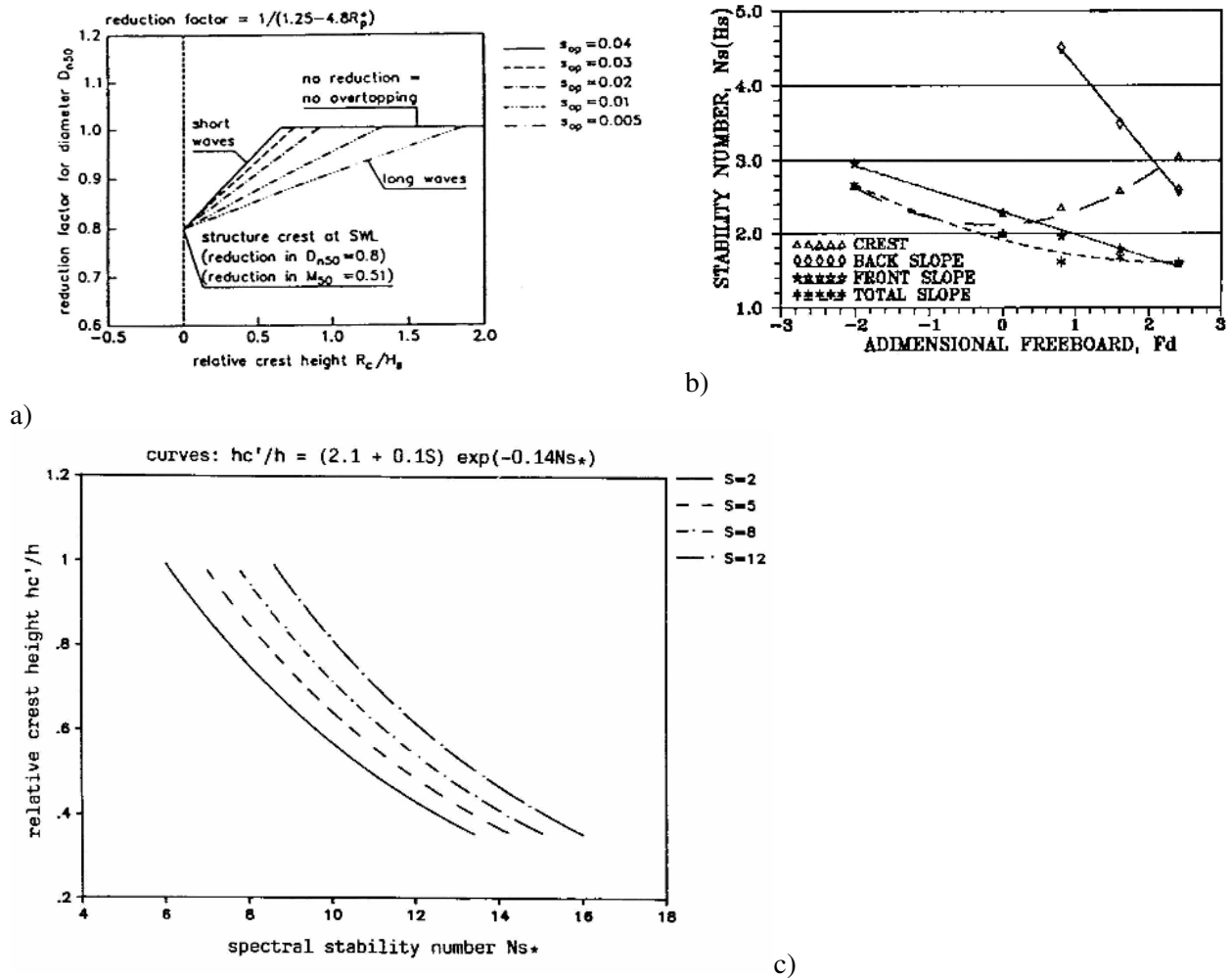


Figure 15. Examples of approaches to the design of stone size

a) Reduction of stone size with the crest height for exposed (emerged) structures in comparison with a standard (high) structure (Van der Meer, 1988, CUR/CIRIA, 1991);  $D_{n50} = (M_{50}/\rho_s)^{1/3}$

b) Criteria for various parts of breakwater acc. to Vidal et al. (1998);  $N_s = H_s/\Delta D_{n50}$ ; adimensional freeboard:  $R_c/D_{n50}$ .

c) Design curves for submerged structures (Van der Meer&Pilarczyk, 1990);  $N_{s*} = N_s \times s_p^{-1/3} = H_s/\Delta D_{n50} \times (H_s/L_{op})^{-1/3}$ ,  $hc'$  = height of structure, and  $h$  = local depth

Usually for submerged structures, the stability at the water level close to the crest level will be most critical. Assuming depth limited conditions ( $H_s = 0.5h$ , where  $h$  = local depth), the (rule of thumb) stability criterion becomes:

$$H_s/\Delta D_{n50} = 2 \text{ or } D_{n50} = H_s/3, \text{ or } D_{n50} = h/6 \quad (12)$$

where  $D_{n50} = (M_{50}/\rho_s)^{1/3}$

It should be noted that some of useful calculation programs (including formula by Van der Meer) are incorporated in a simple expert system CRESS, which is accessible in the public domain (<http://www.ihe.nl/we/dicea> or [www.cress.nl](http://www.cress.nl)).

Alternative solutions, using geotubes (or geotubes as a core of breakwaters), are treated in (Pilarczyk, 1999). An example of this application can be found in (Fowler et al., 2002).



Figure 16. Example of construction of breakwater using geotubes

Useful information on functional design and the preliminary structural design of low crested-structures, including cost effectiveness, can be found in CUR (1997).

## 6. Conclusions

The author does not intend to provide the new design rules for low-crested structures. However, it is hoped that this information will be of some aid to designers looking for new sources, who are considering these kinds of structure and improving their designs.

As was already concluded by Black&Mead (1999), rock walls, breakwaters or groynes usually serve their purpose of protecting land from erosion and/or enabling safe navigation into harbours and marinas, but these same structures could also have recreational and commercial value. Therefore, multi-purpose recreational and amenity enhancement objectives should be incorporated into coastal protection and coastal development projects. Offshore breakwaters/reefs can be permanently submerged, permanently exposed or inter-tidal. In each case, the depth of the structure, its size and its position relative to the shoreline determine the coastal protection level provided by the structure. To reduce the cost some alternative solutions using geosystems can be considered. The actual understanding of the functional design of these structures may still be insufficient for optimum design but may be just adequate for these structures to be considered as serious alternatives for coastal protection.

Continued research, especially on submerged breakwaters, should further explore improved techniques predict shore response and methods to optimise breakwater design. A good step (unfortunately, limited) in this direction was made in a collective research project in the Netherlands (CUR, 1997). Research and practical design in this field is also the focus of the “Artificial Reefs Program” in New Zealand ([www.asrltd.co.nz](http://www.asrltd.co.nz)), the International Society for Reef Studies (ISRS) ([www.artificialreefs.org](http://www.artificialreefs.org)), and the European Project DELOS (Environmental Design of Low Crested Coastal Defence Structures, 1998-2003) (<http://www.delos.unibo.it>).

These new efforts will bring future designers closer to more efficient application and design of these promising coastal solutions. The more intensive monitoring of the existing structures will also help in the verification of new design rules. International cooperation in this field should be further stimulated.

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