Review and Recommendations for

For

Hotel Porto Real,

Playa Del Carmen, Quintana Roo

Mexico

Submitted to

MARENTER, S.A. DE C.V.

By The

REEF BALL

Development Group, Ltd.

With Partners Including:

LEE E. HARRIS, Ph.D., P.E.

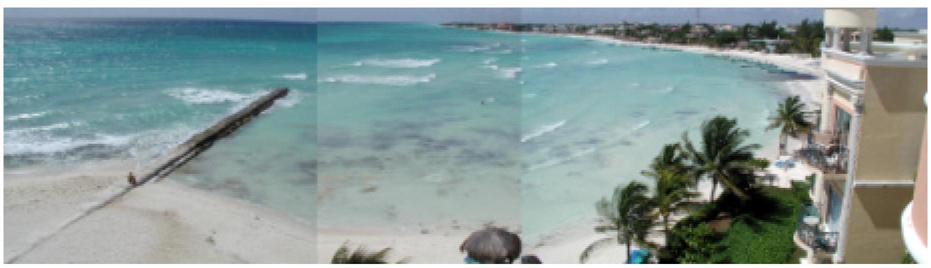
Consulting Coastal/Ocean/Civil/Engineer

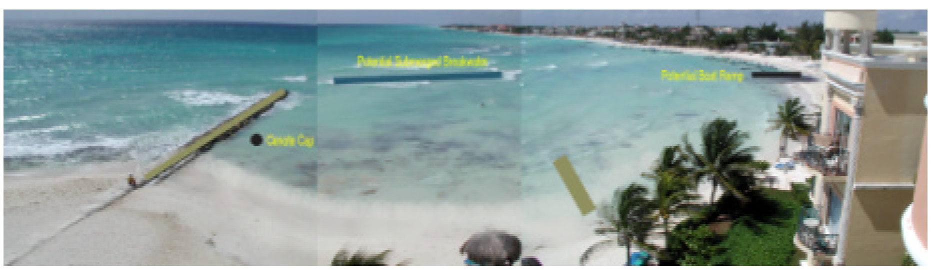
Executive Summary

Introduction

The Hotel Porto Real in Play Del Carmen contracted with Marenter, S.A. De C.V. to undertaking an engineering and construction project to improve the southern beach front area of the Hotel including beautification of an existing groin, construction of a boat ramp, addition of seaward erosion protection, and creation of a submerged breakwater to further protect the beach from erosion processes. Marenter, S.A. De. C.V. has recently become a Reef Ball Authorized Contractor and therefore sought Reef Ball advise on the use of Reef Balls for the submerged breakwater. Dr. Lee Harris, a professor at the Florida Institute of Technology was additionally contracted to look into the engineering aspects of the breakwater as well as to comment on the other designs being considered.

October 2001 ariel photo by Dr. Lee Harris of the south section of the beach being examined by this project. The second picture depicts possible enhancements.





Recommendations

We first conducted a site review and examined plans dated 25 Sept, 2001 by Marenter which showed the proposed submerged breakwater, proposed boat ramp, Palapa Plan, and walkway.

Enrique Chacon explained to us the permitting issues and the various options and client goals and expectations.

After a second day of observations of the wave climates, usage patterns and physical geography of the site, we then undertook in water surveys. Dr. Harris discovered a flowing cenote 30 meters from the beach, 8 meters inside of the current groin (see above photo for approximate position). This cenote may have some impacts in keeping sand in suspension at the site, certainly we observed this on the day of our investigation.

Mr. Barber did a biological survey to determine what type of reef could be expected in the proposed location. As a result of these investigations, we observe the following:

· The bay is located in a naturally silty/sand suspension area. The property itself acts as a kind of natural groin in the sand system and therefore the area is best characterized as a backside groin environment (the "groin" on the property is actually functioning more as a "Spur" rather than a "groin." Although clear during some weather conditions, the high sediment load (and freshwater mixing from the cenote) prevents hard corals from forming. The natural biology is mostly calcerous and non-calcerous algaes. No hard and minimal soft corals were observed in the areas surveyed. Additionally the area is too rocky and too shallow for general snorkeling use (even when visibility is good which may be less than common). During our underwater survey, the visibility was too low to observe any fish. We therefore conclude that the selected site would not be biologically productive nor would it add any significant biological and/or snorkeling assets to the property.

-Reef Balls were designed specifically with hard and soft coral biology in mind. Because this site is not ideal for reef biology, some of the special treatments used in making Reef Balls such as a specially textured surface (which enhances the settlement and growth of hard corals) may not be needed. Therefore, Marenter may propose a more "generic" reef unit that might offer some cost savings to the client. Although the Reef Ball brand name would not be used because it is marketed to tourists primarily for reef creation, the same physical engineering standards would be used to ensure a breakwater that meets all of Reef Ball Development Group's tough standards for non-biological uses.

· Although the proposed submerged breakwater is not likely to add significant biological enhancement, Dr. Harris concludes that the site selection, in terms of reduction of the wave climate in the bay was appropriate. The presented design depth, width and height would attenuate the waves as desired in the bay. An attenuation paper was prepared by Dr. Harris (See Appendix C) and transmitted to Marenter so that Marenter could design the most cost effective width and height given the clients desire of wave attenuation.

Note: additional attenuation can be achieved by 1) a longer breakwater, 2) greater width of the breakwater, 3) units placed closer to the surface, or 4) Any combination of the above.

· We suggest that the client “cap” the cenote with a single Reef Ball. This cap would include a pipe fitted into the cenote so that the fresh water would emerge near the surface of the water instead of in the sand. This will reduce the sand suspension that the cenote is generating and therefore may aid in holding sand on the beach. It was unknown if the cenote is seasonal, variable in water volumes or stable at one location. These factors should be considered before installing a cap. Now that it’s location has been documented, we suggest monitoring it over a period of time by Martener staff.

· The walkway built around the beach as a “hidden” shore protection to reduce wave run up on the property should be engineered with a “stair” step profile to the beach side rather than a more solid wall as shown. Solid walls can create strong wave reflections during storm events and more quickly erode the beach. “Stair” stepping or sloping the ocean side of the protection can direct energy upward rather than reflecting it back so that less sand is lost.

· The proposed design shows a boat ramp which will also function as a mini-groin system (replacing the small functioning sand filled container groin which provided temporary, yet excellent sand accretion). Because the sand fillet from the temporary groin represents a classic example of groin sand accretion, we found the solution in this case to be effective without causing down drift problems to adjacent beaches (due to the natural groin shape of the property itself). Therefore, if it is possible to get permits, we suggest the addition of 2 small groins central to the beach…an excellent example of such a structure would be one that Marenter constructed at Bahia Principle Hotel. (See the photos above to see an example location of a small groin).

· We do not recommend the construction of the Palapa or any walking are as along the current groin. However, the groin does need to be covered to create a better look on the property but the wave climate is too strong to allow guests access to this area without risking injury, in our opinion. If access is desired, it should be by a narrow walkway with strong handrails on the inside of the breakwater engineered so that injury was unlikely in the event of a rouge wave breaking unexpected on guests.

· If a snorkeling trail is desired, the North side of the property is biologically suited for reef development, but a submerged breakwater would be required to calm the waves for usage in normal wave conditions.

· There are another technologies, such as Protect Tubes ™ which could be used on the beach side to protect the property more effectively than a walkway from storm run-up. However such technology can be fairly expensive ranging from $200-$500 per linear foot. Martener has access to this technology through the Reef Ball Development Group, Ltd. should you desire a more formal proposal on this approach.

Reef Balls as the Submerged Breakwater

Ultra Reef Balls being deployed at Gran Dominicas Hotel in the Dominican Republic, note the appearance of the Reef Ball submerged breakwater as only a dark line in the water. This would be the same look generated at the Mayan Palace.

Stability

Physically, the site experiences a significant wave climate and the Mayan Riviera faces threats from hurricanes such as Gilbert. Reef Balls can be engineered heavier, with anchoring systems, and with modular bases for extra weight. However, we conclude that only the use of a fiberglass rebar anchoring system; along with higher than normal weight Reef Balls is sufficient for the property. Such a system has been proven by a category III hurricane's direct hit on the Dominican Republic Reef Ball site. Modular bases would have made handling the Reef Balls difficult without a barge since we are planning a floating deployment staged from the beach and we believe this is unnecessary given the excellent hard bottom for anchoring on the site.

Longevity

Reef Ball has a long history of using high tech concrete to engineer structures designed to last centuries rather than decades. Our work has required it because longevity is an important design criterion when building coral reefs that potentially last for thousands of years. By using specially designed, high strength concrete and using proprietary admixtures, we will create a high strength, abrasion resistant concrete, (without iron rebar in the modules), that will have an engineering life of hundreds of years. Therefore, the client can consider this solution a final one. Appendix A contains the typical concrete mix design used to build our modules. Martener will use a similar custom mix for you.

Beach Creation

There are three options to obtaining the beach sand; sand nourishment, natural accumulation of sand, or a hybrid approach of seeding some while accumulating the rest.

Environmentally, a natural accumulation of sand is desired and Reef Ball submerged breakwaters are normally set up with this system.

Right: Natural accumulation of sand in the Reef Ball Dominican Republic Beach Creation Project after 4 months.[5]

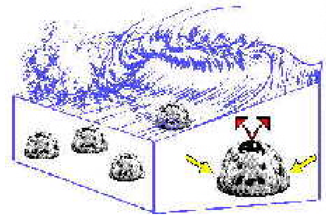
However, due to the natural "groin" configuration of the property, during some seasons, most of the natural transport of sand is bypassing this section of the property's beach. Fortunately, building events are natural in the area during certain wind directions too. During this time, the property will build more with a submerged breakwater installed than it builds today.

In order to rapidly seed the beach, it would be possible, but not to required, to renourish the beach using medium grained sand (with minimal fines) with the submerged breakwater protecting it from loss during storm and tidal events. Accreted sand would then build up over time as a top layer.

However, our recommendation, provided the client has the patience, is to construction the submerged breakwater without sand pumping. If sand reserves are not great enough given time, consider extending the submerged breakwater additional meters to further protect the area during southern winds.

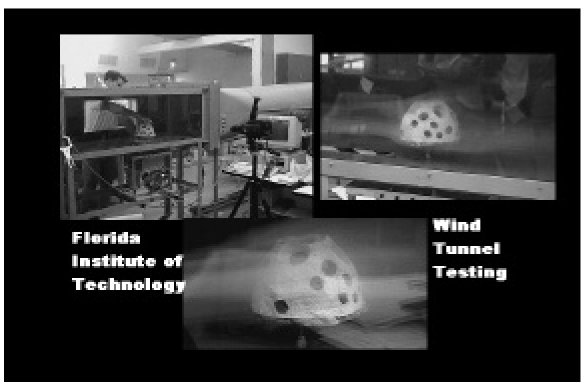
1 month after a category III direct hit from Hurricane Georges showing natural replenishment by Reef Balls.

Why Reef Balls Work Better than Solid or Rock Submerged Breakwaters



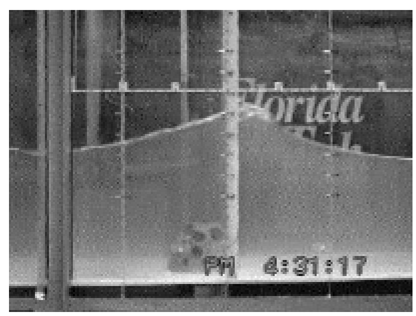
Reef Balls were initially designed to be biologically active (to create natural reefs) and to be stable in hurricanes. Essentially Reef Balls needed to be the base of a natural reef. To do this, we had to design our holes to create whirlpools so that corals could be fed better by passing currents. Additionally, we created a large hole in the top of the Reef Ball so that waves and currents would be jetted from the top, adding to the stability of Reef Balls. Our goal was to use the least amount of concrete to make a unit that was stable in hurricanes.

Traditional and barrier submerged breakwaters work by making waves break. As a wave breaks, it loses much of its energy. The problem with these systems is that if the wave does not break, very little energy is lost. And as the wave is lifted over the submerged breakwater, if it does not break, then the acceleration, as it goes down on the other side of the breakwater, can create washout.



Wind tunnel demonstrating whirlpool effect of Reef Balls

Reef Balls work on an additional principle. Being full of holes that create whirlpools, and offering a variety of angles of reflection from the round shape, any wave that traverses a field of Reef Balls has to “fight itself” and therefore looses energy



Reef Ball Stability Tests at FIT Wave Tanks

in relation to the number of rows of Reef Balls that are traversed. The original wave keeps its shape; it just gets smaller and continues to the beach without washout. Therefore, it does its normal job of carrying sand, at the lower energy level, to the beach.

With major storm events, the width of the Reef Ball fields must be wide enough to cause a break on the larger waves like a traditional submerged breakwater. This slows them by the normal breaking process and also by fighting the whirlpools/wave reflections as with non-breaking waves. In these major storm events, wash out is possible even with the Reef Balls because the breaking waves always create wash.

(Note: Since the seafloor on the lee side of the proposed position of the Reef Ball is rock or limestone outcroppings, wash out is not an issue for the proposed solution.)

Traditional solid submerged breakwaters may perform better than Reef Balls for natural sand accumulation if the wave climate is very light. This is because Reef Balls will slow down even smaller waves and if there is normally barely enough energy to bring sand to the beach, the Reef Balls may slow the waves too much and cause sand to fall out just past the Reef Balls, rather than on the beach. If average waves are very small and storms also bring proportionately small waves, consider either placing your Reef Balls closer to shore or use a solid submerged breakwater if using a system designed to naturally create sand. If you are renourishing your sand, it is always better to have a lower wave climate so Reef Balls are the best choice. This is because you don’t want waves to carry away renourished sand. At the Hotel Mayan Palace, the wave climate is anything but small and the recommend solution includes renourishment and this is not a consideration to be worried about.

Non-submerged structures that stick out of the water rely on reflection to stop waves. Reflection puts a huge stress on walls and that is why most reflective structures must be massively engineered and even then failure is possible. Non-submerged structures are also unappealing to the eye in most installations. This reflection effect can also push sand away from the property. This is why seawalls often accelerated the rate of sand loss. Reflection of waves has been blamed on a variety of problems (both physical and environmental) with traditional engineering techniques and therefore Reef Ball Development Group, Ltd. does not recommend reflective technologies.

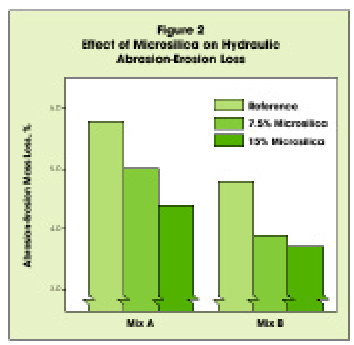
Participating Sub-Contractors & Partners

There will be a variety of companies participating in this project working through the Reef Ball Development Group, Ltd. and contracted through MARENTER, S.A. DE C.V.. MARENTER, S.A. DE C.V. may at its discretion either contract individually with these companies or work through Reef Ball Development Group, Ltd. to manage the construction as a single project.

Todd Barber, CEO of the Reef Ball Development Group, will be the point contact for MARENTER, S.A. DE C.V. to oversee this project due to its unique and complex nature. Mr. Barber is the founder of Reef Ball Development Group, Ltd. and has been working restoring reef systems worldwide since 1992. His work in the management-consulting field with the Alexander Group and TPF&C before starting Reef Ball makes him well qualified to assist in the management of complex projects. Reef Ball has conducted over 3000 projects in over 40 countries worldwide deploying over ½ a million Reef Balls. Information on the companies Mr. Barber manages can be found at www.artificialreefs.org.

Dr. Lee Harris, Ph.D., P.E., Consulting Coastal, Ocean and Civil Engineer of the Florida Institute of Technology will be doing the engineering, physical modeling, survey work and scientific monitoring of the project. He has worked with submerged breakwaters since the 1980s and has been involved with hundreds of projects worldwide.

W.R. Grace will supply critical admixtures including Force 10,000 Microsilica, Adva Flow, Grace Microfibers, Darex II and other proprietary admixtures used to insure that your Reef Balls and breakwater will last for hundreds of years and will be strong enough and abrasion resistant to handle the constant sandblasting effect subjected to a submerged breakwater. We have elected to engineer your breakwater with the some of the best concrete technology available today that is also designed to enhance the biological performance of your breakwater as a living reef. Rick Conlin is in upper management at W.R. Grace and will be our liaison with W.R. Grace. He has worked designing special mixes for the Reef Ball Group since 1993.



Appendix A

Reef Ball Sizes, Weights, Volume & # of Holes

Style	Width	Height	Weight	Concrete Volume	Surface Area	# Holes
Ultra Ball	6 feet (1.83m)	4.5 feet (1.37m)	4000-6000 lbs (1814-2722 kg)	1 yard 0.76m³	150 ft² 13.9 m2	29-34
Reef Ball	6 feet (1.83m)	4 feet (1.22m)	3000-6000 lbs (1360-2722 kg)	0.75 yard 0.57m³	130 ft² 12.1 m2	29-34
Pallet Ball	4 feet (1.22m)	3 feet (0.9m)	1500-2200 lbs (680-998 kg)	0.33 yard 0.25m³	75 ft² 7.0 m2	17-24
Bay Ball	3 feet (0.9m)	2 feet (0.61m)	375-750 lbs (170-340 kg)	0.10 yard 0.08m³	30 ft² 2.8 m2	10-16
Mini-Bay Ball in development	2.5 feet (0.76m)	1.75 feet (0.53m)	150-200 lbs ([illegible] kg)	less than 4 50 lb bags		8-12
Lo-Pro	2 feet (0.61m)	1.5 feet (0.46m)	70-100 lbs ([illegible] kg)	less than 2 50 lb bags		6-10
Oyster	1.5 feet (0.46m)	1 foot (0.30m)	30-45 lbs ([illegible] kg)	less than 1 50 lb bag		6-8

APPENDIX B: REEF BALL TYPICAL CONCRETE SPECIFICATIONS

PART I - GENERAL

1.01 Section Includes

A. Concrete proportioning and products to be used to secure concrete, which when hardened will produce a required strength, permeability, and resistance to weathering in a reef environment.

1.04 References

A. ACI-211.191-Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete.

B. ASTM C 260- Standard Specifications for Air-Entraining Admixtures for Concrete.

C. ASTM-C 1116 Type III- Standard Specifications for Fiber Reinforced Concrete or Shotcrete.

D. ACI - 305R -91- Hot Weather Concreting.

E. ACI - 306R -88- Cold Weather Concreting.

F. ACI - 308- Standard Practice for Curing Concrete.

G. ASTM C 618-Fly Ash For Use As A Mineral Admixture in Portland Cement Concrete.

H. ASTM C 494-92- Standard Specifications for Chemical Admixtures for Concrete.

I. ASTM C 1202-91- Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration.

J. ASTM C 33- Concrete Aggregates.

K. ASTM C 94- Ready Mix Concrete.

L. ASTM C 150-Portland Cement.

M. ACI 304- Recommended Practice For Measuring, Mixing, Transporting and Placing concrete.

N. ASTM C 39 (Standard Specifications For Compressive Testing)

O. ASTM C-1240-93 (Standard Specifications for Silica Fume Concrete)

PART II PRODUCTS

2.01 Portland Cement: Shall be Type II and conform to ASTM C-150

2.02 Fly Ash: Shall meet requirements of ASTM C-618, Type F. And must be proven to be non-toxic as defined by the Army Corps of Engineers General Artificial Reef Permits. Fly Ash is not permitted in the State of Georgia and in most Atlantic States. (In October, 1991, The Atlantic States Marine Fisheries Commission adopted a resolution that opposes the use of fly ash in artificial reefs other than for experimental applications until the Army Corps of Engineers develop and adopt guidelines and standards for use.)

2.03 Water: Shall be potable and free from deleterious substances and shall not contain more that 1000 parts per million of chlorides or sulfates and shall not contain more than 5 parts per million of lead, copper or zinc salts and shall not contain more than 10 parts per million of phosphates.

2.04 Fine Aggregate: Shall be in compliance with ASTM C-33.

2.05 Coarse Aggregate: Shall be in compliance with ASTM C-33 #8 (pea gravel). (Up to 1 inch aggregate can be substituted with permission from the mold user.) Limestone aggregate is preferred if the finished modules are to be used in tropical waters.

2.06 Concrete Admixtures: Shall be in compliance with ASTM C-494.

2.07 Required Additives: The following additives shall be used in all concrete mix designs when producing the Reef Ball Development Group's product line:

A. High Range Water Reducer: Shall be Adva Flow as manf. by W.R. Grace.(ASTM C-494 Type F)

B. Silica Fume: Shall be Force 10,000 Densified in Concrete Ready Bags as manf. by W.R. Grace. (ASTM C-1240-93)

C. Air-Entrainer: Shall be Darex II as manf. by W.R. Grace (ASTM C-260)

2.08 Optional Additives: The following additives may be used in concrete mix designs when producing Reef Ball Development's product line.

A. Fibers. Shall be either Microfibers as manf. by W.R. Grace, or Fibermesh Fibers (1 1/2 inches or longer) as manf. by Fibermesh. Either product can be in ready bags.

B. Accelerators: Either a non-Chloride or Daracell as manf. by W.R. Grace may be used but only when needed due to temperatures less than 40 degrees F. (ASTM C-494 Type C or E)

C. Retarders: Shall be in compliance with ASTM-C-494-Type D as in Daratard 17 manf. by W.R. Grace

2.09 Prohibited Admixtures: All other admixtures are prohibited. Other admixtures can be submitted for approval by the Reef Ball Development Group, Ltd. by sending enough sample to produce five yards of concrete, the current MSDS, and chemical composition (which will be kept confidential by RBDG Ltd.) A testing fee of $2,500 must accompany the sample. Temporary approval will be granted or denied within 10 days based on chemical composition, but final approval may take up to 3 months since samples must be introduced in a controlled aquarium environment to assess impacts on marine and freshwater species.

PART III Concrete Proportioning:

A. **General:** The intent of the following proportions is to secure concrete of homogeneous structure that will have required strength and resistance to weathering.

B. Proportions:

	One Cubic Yard	One Cubic Meter
Cement:	600 lbs. (Min.)	356 kg
Aggregate:	1800 lbs.	1068 kg
Sand:	1160 lbs	688 kg
Water:	240 1bs. (Max.)	142 kg
Force 10K:	50 lbs	30 kg
Darex AEA:	3 oz.	.1 1iters
*Adva Flow (Superplastisizer):	25- 45 oz.	1-1.75 liters

*NOTE: Adjust Adva dosage as needed to obtain workable, placeable mix (170-250mm / 7-10 inch slump), and to achieve .40 w/c ratio.

Fibers: 0-3# (Max.) as needed to reduce micro cracking 1# (Min.) required if Silica Fume exceeds 50#

Accelerator: As needed to achieve de-molding no sooner than: 3-4 hours for heavy duty molds (All Polyform side balls) 6-7 hours for standard molds (Molds with any tether balls)

NOTE: Silica Fume or Force 10K shall be dosed at a 10# minimum in Bay Balls and Pallet Balls while Ultra & Reef Balls shall require a minimum of 25#. All molds must use at least 50# for floating deployments. All mold sizes must use at least 50# for use in tropical waters unless special curing procedures are followed.

?? This product is being specified not only for strength, but also to reduce pH to spur coral growth, to reduce calcium hydroxide, and to increase sulfate resistance. It is a non-toxic pozzalan.

Appendix C: BREAKWATER WAVE ATTENUATION

BREAKWATER GEOMETRY

The main parameters used to describe the general geometry of a submerged breakwater are shown in Figure 1. These include the height of the structure = *h*, water depth at the toe of the structure = *d*, and the freeboard of the structure = *F*, where the freeboard is the difference between the height of a breakwater structure and the water depth at the seaward toe of the structure. The slope of the seaward face of the breakwater is *tan ?*, and the offshore slope of the bottom seaward of the structure is *tan ?* = *m*, which is zero for a horizontal sea bottom.

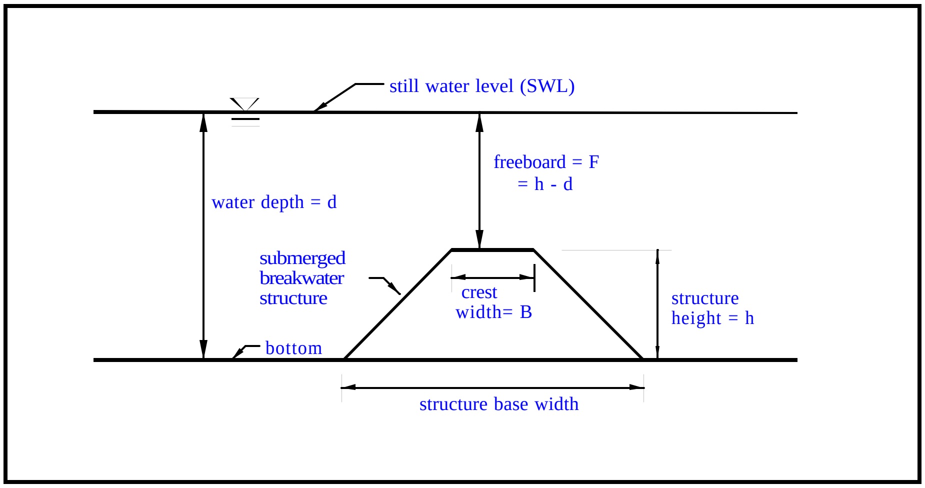


Figure 1. DEFINITION SKETCH FOR A SUBMERGED BREAKWATER

One of the most important parameters for the design and effectiveness of a breakwater is the degree of emergence or submergence. This can be expressed by three different dimensionless terms:

1. the degree of submergence = *d/h* ;
2. the relative structure height = *h/d* ; and
3. the relative freeboard to water depth ratio = *F/d* .

The degree of submergence is the ratio of the water depth to the height of the structure. For an emergent or subaerial structure, whose crest height exceeds the water depth, this ratio is less than one ($d/h < 1.0$), and for a submerged structure, this ratio is greater than one ($d/h > 1.0$).
The relative structure height, which is the ratio of the structure height to the water depth (*h/d*), also can be used as a dimensionless parameter to express the degree of emergence or submergence of a breakwater. The relative height has a value that is

less than one (h/d < 1.0) for a submerged structure, and greater than one (h/d > 1.0) for a subaerial or emergent breakwater.

The freeboard is defined as the structure height minus the water depth,

$F = h - d$ [1.],

where F is the freeboard, h is the height of the structure above the bottom, and d is the water depth at the seaward toe of the structure. An emergent or subaerial breakwater has a positive freeboard value, and a submerged breakwater has a negative value for the freeboard. The dimensionless parameter for the relative freeboard is the freeboard ratio, which is defined as the freeboard divided by the water depth,

$$\frac{F}{d} ? \frac{h ? d}{d} ? \frac{h}{d} ? 1 \qquad [2.].$$

With this definition of the freeboard ratio, an emergent or subaerial breakwater has a positive value for the freeboard ratio (F/d > 1.0), while a submerged breakwater has a negative value for the freeboard ratio(F/d < 1.0).

These three dimensionless quantities, d/h, h/d, and F/d, indicate the relative height of the breakwater compared to the water depth, and are used to determine the magnitude of the wave and current forces on the breakwater, and the effectiveness of the structure in attenuating wave energy. A classification scheme is formulated later in this study to quantify these relationships.

BREAKWATER RELATIVE CREST HEIGHT

Another important dimensionless parameter used for determining the interaction between the waves and a breakwater structure is the freeboard divided by the wave height, which can be expressed as:

$$\frac{F}{H} ? \frac{h ? d}{H} ? \frac{h}{H} ? \frac{d}{H} \qquad [3.],$$

where H is the height of the wave, measured from the bottom of the trough to the top of the crest. The use of the wave height in this ratio provides a direct comparison between the height of the structure above or below the still water level, and the height of the waves impacting the structure. Note that this ratio is equal to the ratio of the structure height to incident wave height minus the ratio of the water depth to the incident wave height.

For a submerged structure, the freeboard and freeboard ratios F/d and F/H all have negative values, and the structure is continuously overtopped by waves. The more submerged the structure is, the more negative the ratio of the freeboard to the wave height, and the interaction between the waves and the structure will decrease.

For an emergent structure that has a positive value of freeboard, F/H is also positive. When the ratio F/H is less than one (F/H <1.0), the structure is easily overtopped by the waves, and significant wave transmission past the structure by overtopping occurs (Ahrens, 1987). When F/H is greater than one (F/H >1.0), the structure height is at least one wave height above the still water level, and most of the wave energy is absorbed and attenuated by the structure. Some wave energy still may be transmitted through the structure if the structure is porous, and some wave energy may be transmitted over the structure by wave overtopping (U.S. Army Corps of Engineers, 1984).

WAVE PARAMETERS

Other dimensionless quantities are used to compare the wave height to the water depth, and to determine the type of wave relative to the water depth. The ratio of the water depth to the wavelength (*d/L*) is used to determine the relative depth of the water compared to the length of the waves. For a ratio of *d/L* greater than one-half, the waves are considered to be in deep wa ter, and for a ratio of *d/L* less than 1/25, the waves are considered to be in shallow water (U.S. Army Corps of Engineers, 1984).

The dimensionless parameter *H/d* is used for the relative height of the wave compared to the water depth, and is often used to determine wave breaking criteria. For a smooth, flat slope, the maximum ratio of *H/d* = 0.78 is commonly used for wave breaking criteria, and increases as the bottom slope increases (U.S. Army Corps of Engineers, 1984).

The surf similarity parameter, also known as the surf parameter or Irribarren Number, is a dimensionless parameter that is used to describe the characteristics of ocean wave phenomena. The surf similarity parameter is defined as

$$? \; ? \; \frac{\tan ?}{\sqrt{\frac{H}{L_o}}} \; ? \; \frac{\tan ?}{\sqrt{\frac{2? \; H}{gT^2}}} \qquad \textbf{[4.],}$$

where *H* is the incident wave height, *T* is the wave period, *g* is the acceleration of gravity, *tan* ? is the slope of the sea bottom or structure slope, and L_o is the deep-water wavelength, where $L_o = gT^2/2?$ using linear wave theory (U.S. Army Corps of Engineers, 1984). The term in the denominator is the wave steepness (*H/L)*, which incorporates the wave period.

The surf similarity parameter is becoming increasingly popular in coastal engineering literature for quantifying wave effects, due to the inclusion of the (1) wave height, (2) wave period, and (3) slope of the structure or bottom, all in one dimensionless parameter. The surf similarity parameter can be used to determine whether breaking or non-breaking waves are occurring, and what type of breaking wave is expecte d. This dimensionless parameter also is used to determine the wave runup on a structure, which then can be used to determine the wave overtopping of a structure (U.S. Army Corps of Engineers, 1984); and for breakwater structural stability (van der Meer, 1987).

WAVE ATTENUATION

The primary purpose of a breakwater is to reduce the wave energy in its lee. The term "wave transmission" is used in reference to the wave energy that does travel past a breakwater, either by passing through and/or by overtopping the structure (U.S. Army Corps of Engineers, 1984). The wave energy that is attenuated in the lee of the breakwater is either dissipated by the structure (such as by friction, wave breaking, armor unit movement, etc.) or reflected back as reflected wave energy. The effectiveness of a breakwater in attenuating wave energy can be measured by the amount of wave energy that is transmitted past the structure. The greater the

wave transmission coefficient, the less the wave attenuation. Wave transmission is quantified by the use of the wave transmission coefficient,

$$K_t = \frac{H_t}{H_i} \qquad \textbf{[5.]}$$

where K_t is the wave transmission coefficient, H_t is the height of the transmitted wave on the landward side of the structure, and H_i is the height of the incident wave on the seaward side of the structure (U.S. Army Corps of Engineers, 1984). Ahrens (1987) defines the wave transmission coefficient differently, using the wave height on the landward side of the structure that would occur in the absence of the structure, in place of the incident wave height on the seaward side of the structure, so that

$$K_t = \frac{H_t}{H_c} \qquad \textbf{[6.],}$$

where H_c is the wave height measured at the same location as H_t, but without the breakwater present.

For submerged breakwaters and artificial reefs, the greater the submergence, the less the wave energy will impact the structure, and the less effective the structure will be for wave attenuation. The *Shore Protection Manual* (U.S. Army Corps of Engineers, 1984) presents numerous graphs of empirical data from wave tank tests that can be used to determine wave transmission coefficients.

Ahrens (1987) presents an empirical formula for subaerial breakwaters, where the crest of the structure is above the still water level and the ratio of freeboard to the incident wave height is greater than one (F/H > 1.0) as follows:

$$K_t = \frac{1.0}{1.0 + \left(\frac{H\,A}{L\,D_{n50}}\right)^{0.592}} \qquad \textbf{[7.],}$$

where H is the incident wave height, A is the cross sectional area of the breakwater, L is the wavelength calculated using linear wave theory for the depth = d, and D_{n50} is the nominal armor unit diameter of the median size (50%) armor unit given by:

$$D_{n50} = \left(\frac{M_{a50}}{\rho_a}\right)^{1/3} \qquad \textbf{[8.],}$$

where M_{a50} is the mass of the median size armor unit and ρ_a is the mass density of the armor material.

Ahrens (1987) presents an empirical formula for “reef breakwaters” where the ratio of the freeboard to the incident wave height is less than one (F/H < 1.0), as

$$K_t = \frac{1.0}{1.0 + \left(\frac{h}{d}\right)^{1.188}\left(\frac{A}{d\,L}\right)^{0.261}\exp\left[0.529\left(\frac{F}{H}\right) + 0.00551\left(\frac{A^{3/2}}{D_{n50}^2 L}\right)\right]} \qquad \textbf{[9.].}$$

The dimensionless terms in parentheses in the denominator are the relative structure height (*h/d*), the ratio of the structure cross-sectional area to the product of the water depth and wavelength (*A/dL*), the relative freeboard (defined in Equation 3 as the ratio of the freeboard to the incident wave height, *F/H* which is the most influential variable according to Ahrens, 1987), and the ratio of the

breakwater cross-sectional area raised to the 1.5 power divided by the product of the median armor unit diameter squared and the wavelength.

Seabrook (1997) performed extensive physical modeling tests of submerged breakwaters, using various depths of submergence, crest widths, water depths, and incident wave conditions. From that data he developed the following design equation for wave transmission at submerged rubble mound breakwaters:

$$K_t = 1 - \left(e^{-0.65\left(F/H\right) - 1.09\left(H/B\right)} + 0.047\left(\frac{BF}{LD_{n50}}\right) - 0.067\left(\frac{FH}{BD_{n50}}\right)\right) \qquad \textbf{[10.].}$$

When using equations 8 and 9 the terms containing the nominal armor unit diameter, *D_{n50}* are often found to be negligible compared to the other terms. This is especially true for Seabrook's relationship in Equation 10, as the freeboard approaches zero as the structure crest approaches the still water level.

Wave transmission coefficients using equations 9 and 10 were calculated for the design of a submerged breakwater using Reef Ball™ artificial reef units. The breakwater design incorporates Reef Ball units placed offshore in rows. The Reef Ball units are 1.2m high and placed in water depth of 1.4m so that the freeboard = *F* = -0.2m. Calculations were performed using 4, 5, and 6 rows of Reef Ball units and for various wave heights and periods. Equation 9 resulted in Ahren's relationship predicting transmission coefficients that did not vary much with varying the number of rows of units or with varying wave conditions. The wave transmission coefficients *K_t* only varied from 0.64 to 0.73 which is only a wave height reduction of 36% to 27%. This predicted wave attenuation is much less that that observed due to the 3-row Reef Ball submerged breakwater in the Dominican Republic.

The results using Equation 10 are shown in Table 1 below, with Seabrook's formula predicting wave transmission coefficients. Note that these values are more indicative of observations of the Dominican Republic Reef Ball submerged breakwater.

Table 1. Wave Transmission Coefficients from Equation 10

wave height = H (meters)	**4 rows**	**5 rows**	**6 rows**
0.50	0.33	0.31	0.30
0.75	0.31	0.29	0.27
1.00	0.33	0.29	0.27
1.25	0.36	0.31	0.28
1.50	0.39	0.34	0.30

Equation 10 was derived from wave tank physical model tests using rubble mound armor stone, not Reef Ball units, so that the results provide more of a design guidance and comparison than actual expected wave transmission. The values in Table 1 show that in order to reduce the wave heights by at least 70% for all of the given wave conditions, 6 rows of Reef Ball units are required. This is the recommended minimum width of the Reef Ball breakwater for wave attenuation sufficient to provide shoreline stabilization in the project area. Five rows of Reef Ball units reduce the wave heights by 66% providing slightly less effective wave

attenuation and shoreline protection. Four rows reduce the wave height by 61%, which is less than that recommended for adequate shoreline stabilization.