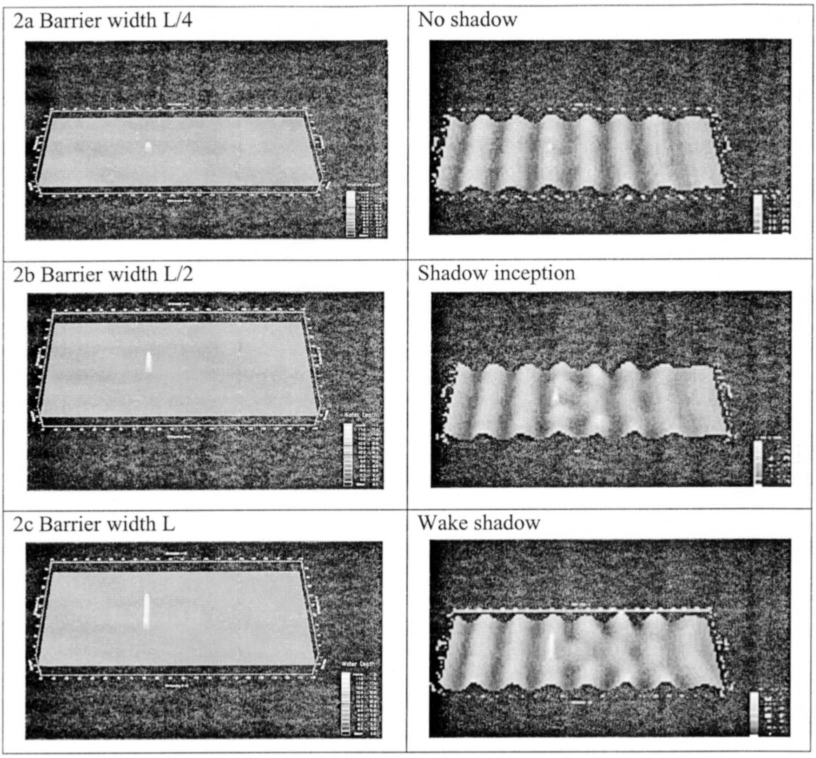

Figure 2. Wave interaction with detached barriers

In terms of real dimensions and common occurrences, at a site in 20 ft. of water, the wavelength could range from roughly 80 ft. for a 4 second period, to 240 ft. for 10 seconds. Therefore the wave barriers need to be at least 40 ft. wide, to maybe 120 ft. wide, minimum, to start being of any functional value. Additional length is needed if there is a chance for oblique wave action. Lesser dimensions will not suffice to offer sheltering to boats or beach.

6) Wave Reflections From Structures: <u>Structure details must be at least a half wavelength wide in order to reflect the wave field</u>. Shown in Figure 3 a,b,c is an example of waves impinging on a horizontally stepped, or saw tooth shaped shoreline. The steps features range in size for 1/8th wavelength to ½ wavelength. The simple monochromatic examples show that reflections back from the steps only begin to occur once the step features reach the half wavelength dimension. Small features only reflect the wave energy as if the wall was straight, smooth, and oblique. This has major implications on wave reflections in harbor settings, where safe navigation and berthing is a concern, and along shorelines where there is a potential to induce erosion on shore opposing shoreline.

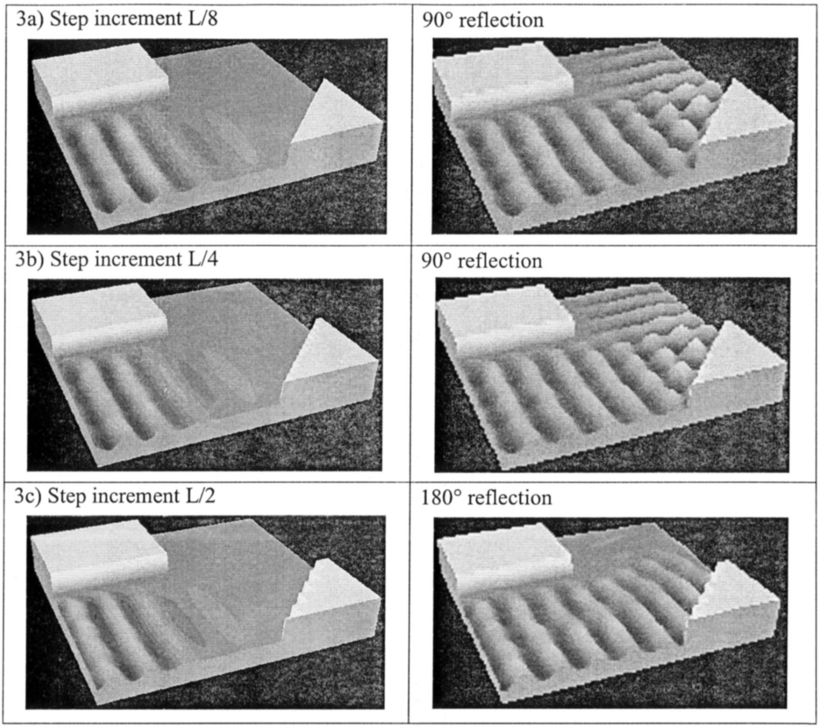

Figure 3. Wave reflection from stepped walls

7) Wave Propagation Between Jetties: <u>The dissipation of wave energy between parallel jetties is equivalent to the dispersion of wave energy through a breakwater slit.</u> Melo and Guza (1990) measured the decay of waves running down a navigation channel between two rock jetties and compared the wave heights to the predicted wave heights, at the same relative location using slit diffraction theory. They found the agreement to be very close. Overlaid on a slit diffraction diagram in Figure 4, is the jetty attenuation pattern showing the close agreement.

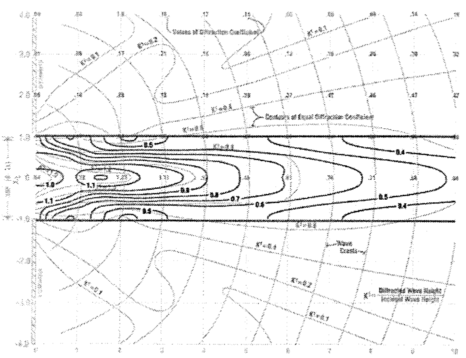

Figure 4. Jetty wave attenuation versus slit diffraction

8) Wave Transmission Over Structures: The size of the transmitted wave is half of the incoming wave less a third of the structure relative freeboard (compared to the wave height). If the storm water level reaches the crest of the breakwater, half of the wave height will pass over. If the amount of freeboard above the water level is the same as the wave height, about 15% passes over. If the breakwater crest is submerged by a wave height, then about 85% of the wave is transmitted. (Note this is also consistent with the wave breaking theory that suggests that a wave will start to break and become height limited when the water depth is between 70% and 130% of wave height, depending on the fronting slope.) Figure 5 (CIRIA, 1991) shows the actual transmission relationship, which is roughly described by the simple relationship:

$$K_t = -0.3(R_c/H_s) + 0.5$$

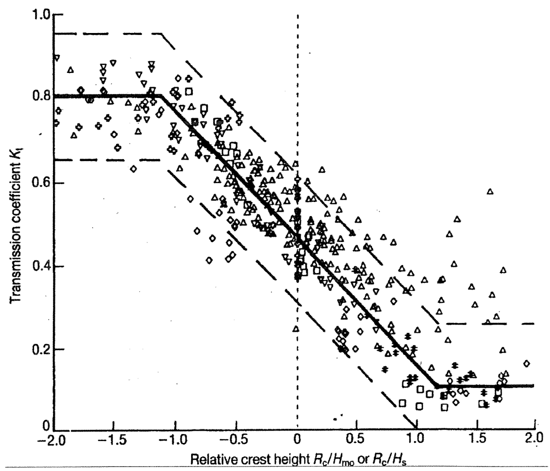

Figure 5. Wave overtopping transmission

9) Shoreline Erosional Response to Gaps in Protection: The maximum retreat of a shoreline edge is roughly a third of the gap width. It is often necessary to estimate how far back erosion can progress. Provided there are some sorts of "hard points" along the shoreline, whether the ends of discontinuous revetments or walls, offshore detached breakwaters or some other shore structures, these features actually serve to limit the ultimate amount of retreat that occurs. (Remember from Rule 5, these features need themselves to be at least a half wavelength wide.) Figure 6, adapted from work by Silvester, et. al. (1980) shows that the magnitude of the eroded abscess in shoreline

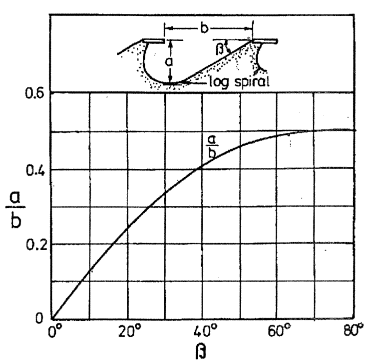

Figure 6. Pocket shoreline recession

position between these hard points can be estimated as a function of the wave approach angle β. For most typical beach situations, the wave approach angle, β, with the shoreline is between 10 and 30 degrees. From the curve then, the maximum recession is 30% or less. Note that the absolute maximum recession is 50% of the gap width when the waves are running virtually parallel with the shore. Generally, the recession distance is assumed at the lowest water level, so that high tide situations would appear to produce greater recession amounts.

10) Floating Attenuator Performance: <u>Floating attenuators need to be at least half the water depth, or half a wavelength in breadth to be effective</u>. Wiegel (1964) developed the power transmission theory for partially penetrating thin barriers. Kriebel and Bollmann (1996) extended that theory to account for reflections from that barrier, and Cox (1987) extended the theory to finite width barriers. The composite relationship, as a closed form expression for estimating wave transmission under a floating attenuator is given as:

$$C_t = C_b K_t$$

where: $C_b = [2\sqrt{(1+(2\pi B/L)^2)}]/[2+(2\pi B/L)^2]$, $K_t = 2P/(1+P)$ and
$P = [4\pi(d-D)/L + \sinh 4\pi(d-D)/L] / [4\pi d/L + \sinh 4\pi(d)/L]$
with B = breadth of float, D = draft of float, d the water depth, and L the local wavelength.

This relationship can be used effectively for oblique waves by trigonometrically adjusting for the apparent wavelength.

Figure 7, from Gaythwaite (1990) graphically illustrates that transmission of waves past a floating attenuator is generally 20-30%, but only up to a certain point. Thereafter, there is an abrupt decrease in wave attenuation. This occurs when the draft is less than 50% of water depth or structure breadth is less than half a wave length. Typical applications in the coastal environment would be for water depths less than 30 ft. Practically, the breadth of a floating attenuator would not exceed 30 ft. so the effective range of wavelength is up to 60 ft. In 30 ft. of water, this translates to a wave period of 3.5 seconds. This type of wave is only observed in the form of a boat wake, or in relatively protected waters of limited fetch. Practically, floating wave attenuators should only be applied when the wave periods do not exceed 4 seconds.

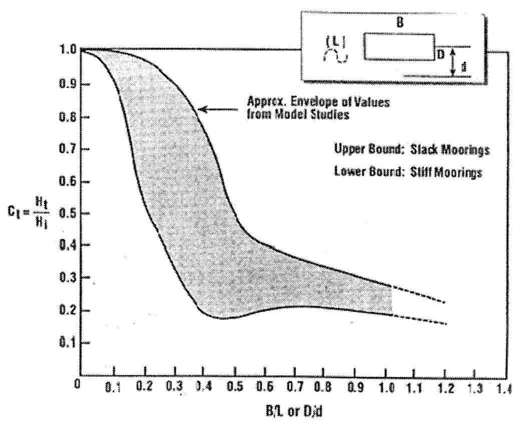

Figure 7. Wave transmission of floating prisms

Coastal Structure Design

The way a breakwater actually works needs to be understood in designing a typical cross section. The part of the breakwater we most often see, that is the large outer armor layer, is not really the most critical part, even though it received the most attention. The impervious, or largely impervious core of the breakwater is what prevents wave transmission. If built too low, then wave action can transmit through the large voids necessarily formed between large armor rocks that sit on the crest. To function properly the core of a breakwater needs to extend at least as high as the static storm water level.

9) Armor Size: <u>The size of a stone needed to resist a given wave has a diameter roughly equal to a third of the wave height</u>. Quarried stone remains the most common material for erosion control. The well-known Hudson's equation gives a relationship between the weight of a required armor piece and the wave height, considering the slope of the structure, the relative density of the material and the shape/placement of the stone. Practically, though, most structures are constructed with a slope near 2H:1V, the dry density of stone is around 168 pcf, and the shape/placement factor is taken to be about 4. Working backwards to find the characteristic dimension of the stone we easily find that for a perfect cube, $\iota = H/3$; for a sphere, $\iota = .4H$; and for a prismatic shape where typically the longest dimension is taken to be triple the shortest, the shortest $\iota = .2H$.

Even more recent armor stability relationships, Ahrens (1988), that consider the added influence of wavelength, still decompose to the rough rule of thumb.

$$N_s = (H_{mo}{}^2 L_{p)})^{1/3}/[d_{50}(\rho_r/\rho_w\text{-}1)] < 7 \text{ (for stable cuboids)}$$

Where N_s = the stability number, H_{mo} = wave height of zeroth moment, L_p = wave length calculated from the peak of the energy spectrum, d_{50} = the 50th percentile rock diameter, ρ_r and ρ_w are the densities of rock and water respectively. Using the same rock parameters and assuming a commonly observed local wave steepness (H/L) of 1:25, then the d_{50} is approximately $0.25H_{mo}$. Therefore the range in size of armor stones falls in the range of $0.2H < \iota < 0.4H$.

10) Armor Placement and Construction: <u>Armor should be laid to a thickness of two stone diameters</u>. The common detailing of the placement of armor on a slope shows veneers of progressingly increasing sized stone armoring placed on the slope. Usually the presumption is that the thickness of the veneer will be placed to a depth equal to two stone diameters to achieve interlock between adjacent stones and to file the voids between stones. Unfortunately graphics and text employ terminology referring to this veneer layer as having "two layers of stone." Contractors, building to this description, actually place the stone on the slope as two separate layers, one on top of the other. This introduces a potential sliding plane between the layers, reducing the presumed stability. Proper specification should call for the layer simply to be built using a certain size (or size range) stone, and that the layer be constructed to full width starting from the toe and working upward. If working with stone diameters as the metric, then the layer thickness should be some multiple of (d_{50} *1.1) to account for imperfect stacking (USACE, 1984).

11) Armor Stone Shape and Size: The ratio of the maximum dimension to the minimum dimension of any armor stone should not exceed three. This standard specification has its basis in three considerations. First, when a stone becomes very slabby in shape, perhaps reaching an aspect ratio of 5, it is subject to breaking in half, once installed, if unsupported. This produces a resulting armor only half the size needed. Second, long, slabby stones tend to be placed parallel to the slope, causing a well-developed layering in the armor previously discouraged. Thirdly, slabby armor produce a generally smoother surface, which though perhaps aesthetically pleasing to look at, actually increases the amount of wave reflection, causing an increase in toe scour, erosion of adjacent shoreline or less tranquil harbor areas.

12) Crest Sizing: Breakwater crests shall be a minimum of three stones in breadth. This is more correctly stated as a breadth of at least three stone diameters minimum. Three stones are viewed as the minimum number needed to achieve good interlocking of the armor across the crest. However, referring to Rule 9), we can also now see that the crest width needs to be at least equal to the wave height. Some designers apply a safety factor of 50%, suggests that the crest width should be 1.5H, or four to five stone diameters.

On shoreline revetments some additional considerations do apply for public safety due to overtopping and providing a scour apron to control any over wash erosion. The splash zone dimension (χ) can be defined by the expression (Cox and Machemehl, 1986):

$$\chi = 0.2(T\sqrt{g})[(R_u\text{-}R_c)^{1/2} - \eta_t^{1/2}]$$

Where R_u is the theoretical run up height on the slope, R_c is the freeboard of the structure, and η_t is the height of the resulting overtopping bore. For erosion control of an unarmored surface in waves, the overtopping bore height needs to be less than a foot (Schiereck, 2001), so the splash apron needs to be at least 16 ft wide to avoid erosion for an overtopping wave cresting 8 feet above the berm top, and having a period of 8 seconds.

For life safety, the Bureau of Reclamation (1988) gives a human safety factor to avoid toppling (product number of depth of flow multiplied by the local velocity) threshold value of 4. The speed of the overtopping bore is roughly $\sqrt{(1.1g\eta_t)}$, where some standing water equal to about 10% of the instantaneous bore depth is assumed. This would indicate the minimum safe splash apron distance is defined when $\eta_t = 0.7$ ft (0.2 m). Substituting that back into the expression above gives, for the same conditions, a public danger zone of approximately 18 ft. Conveniently, designers historically have guessed a splash apron width of 20 ft. as a default dimension, which seems to accidentally work.

As a final consideration, breakwaters are commonly constructed using land-based equipment that must crawl along the crest to advance the work. This construction roadbed, usually at least 12 ft. for one-way traffic, is typically the top of the breakwater core, or sub-armor layer. Therefore the crest dimension may be determined as a consequence of needing to create a minimum width roadbed for one-way or even two-way traffic during its construction.

CONCLUSIONS

The most complicated of design tools do have a basis in practical knowledge, and these rules of thumb are helpful, both in initially scoping a problem and solution set, but more importantly serving as the quality check for reasonableness after much more detailed theoretical study. As for "why are manhole covers round?," the answer lays in nothing theoretical, or structural, or even fabrication. The answer is lays in function. It is the only shape, that when turned in any direction, cannot be accidentally dropped through the hole on someone working below.

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Cox Creek Dredged Material Containment Facility: Hurricane Isabel Evaluation & Coastal Engineering Issues

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Abstract

The Cox Creek Dredged Material Containment Facility (DMCF) project is being constructed for the Maryland Port Administration (MPA) and will be used for the placement of material dredged from Baltimore Harbor. Moffatt & Nichol (M&N) prepared a coastal engineering design for rehabilitation of the existing containment dikes to re-activate the facility. Design components included raising the height of the dikes, placing a stability berm to account for poor geotechnical conditions, and armor stone slope protection. The design process included evaluating various slopes and armor stone combinations to select an optimal configuration as regards initial construction versus long-term maintenance costs. M&N completed the design and prepared bid documents for construction that began in September 2002. M&N also performed the construction oversight for the project. The earthwork and armor stone for the dikes were completed at the end of August 2003. Beginning September 18, 2003 and ending September 20, 2003, the Mid-Atlantic region from North Carolina to Maryland was subject to high water levels and high wind speeds associated with Hurricane Isabel. The purpose of this paper is to present data obtained from recording stations around the Chesapeake Bay and evaluate the performance of the coastal engineering aspects of the Cox Creek DMCF project following this 100-year return period water level event.

Project Description

The Cox Creek DMCF site is located in the Patapsco River as shown in Figure 1 at latitude 39° 12' N, and longitude 76° 32' W. The site is located approximately 7 miles southeast of the Inner Harbor of Baltimore (south of the Francis Scott Key Bridge along the west shore of the Patapsco River) and 3 miles due south of the Dundalk Marine Terminal, in northeast Anne Arundel County, Maryland. The site area useable for dredged material placement is about 115 acres; the length of the perimeter dike requiring shoreline protection is about 5000 feet. A major purpose of the project was to rehabilitate the containment dikes and provide shoreline protection that included armor stone designed to withstand wave energy from a relatively long exposure direction to the southeast (about 17 miles).

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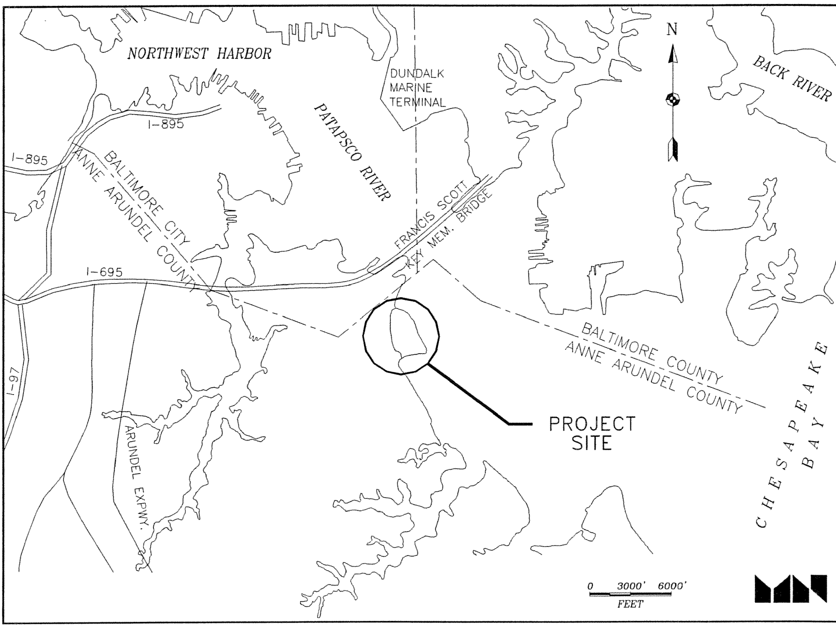

Figure 1. Cox Creek DMCF Project Site Location

Coastal Protection Design

The phenomena most germane to the overall planning of the dike design were bathymetry, water levels, wind conditions, wave conditions, slope stability which dictated maximum allowable combinations of side slopes and structure heights, and settlement which influenced the initial and final crest elevation of the dike.

Bathymetry of the site along the toe of the dikes is generally constant at 3 feet below Mean Lower Low Water (MLLW). The project area is gently sloping eastward at about 1:100. Normal water levels at the site are dictated by astronomical tides which have a mean range of 1.36 feet and a spring range of 1.66 feet. Extreme water levels are dictated by storm surge, which is the temporary rise in water level generated either by large-scale extra-tropical storms known as northeasters, or by hurricanes. The rise in water level results from wind action, the low pressure of the storm disturbance and the Coriolis force. Wave setup is a term used to describe the rise in water level due to wave breaking. Specifically, change in momentum which attends the breaking of waves propagating towards shore results in a surf zone force that raises water levels at the shoreline.

A comprehensive evaluation of storm-induced water levels for several Chesapeake Bay locations was conducted by the Virginia Institute of Marine Science (1978) as part of the Federal Flood Insurance Program. Results of this study are summarized in the following Table 1 for selected locations. The closest station location to Cox Creek is Baltimore (at Fort McHenry).

Table 1. Water Level Elevation per Return Period for Chesapeake Bay Locations (ft, NGVD)

Return Period (Years)	Baltimore	Annapolis	Chesapeake Beach	Matapeake	Solomons Island	Cambridge
10	4.1	4.0	3.5	4.0	3.4	3.9
50	6.8	6.2	5.2	6.2	4.8	5.1
100	8.1	7.2	6.1	7.2	5.5	5.9
500	10.7	9.4	7.9	9.2	7.0	7.5

Design winds for the site were developed on the basis of data collected at Baltimore-Washington International (BWI) airport (Table 2). These winds, which can exceed 90 miles per hour during a 100-year storm, were used to develop design wave conditions. Predominant wind direction is from the northwest.

Table 2. Design Wind Speed per Direction and Return Period for Baltimore-Washington International (BWI) Airport Wind Speed and Direction (MPH)

Return Period (Years)	N	NE	E	SE	S	SW	W	NW
5	40	37	32	37	36	47	50	54
10	48	44	38	45	43	56	54	59
25	59	55	47	58	54	70	60	67
50	69	65	55	69	63	82	64	73
100	81	76	65	82	74	97	69	81

Cox Creek is primarily exposed to wind-generated waves approaching from the eastern direction. The longest fetch distances to which the site is exposed correspond to the southeast direction, thus the highest waves approach from this direction. Hindcast waves were computed and the results indicate that 5-year return period waves from the southeast direction have a significant height (H_s) of 3.8 feet and a peak spectral wave period (T_p) of 4.1 seconds. The 100-year return period significant wave height from the southeast direction was hindcast to be 7.8 feet with a peak spectral wave period of 5.6 seconds. These wave heights represent deep water conditions some distance offshore of the dikes, which are located in relatively shallow water (low water depth of 3 feet). Given the relatively shallow depths fronting the dike, the structure will be exposed to some breaking waves, and wave heights would be