

A brief history of headland breakwaters for shore protection in Chesapeake Bay, USA

By

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ABSTRACT

The first private headland breakwater system in Chesapeake Bay was built in 1985 at Drummond Field on the James River in James City County, Virginia. At that time, the Virginia Institute of Marine Science (VIMS) was researching stable pocket beaches where the embayed beach morphology emulates nature. This site provided an opportunity to apply those principles, including stable bay morphology. These principles were considered somewhat extreme since no extensive breakwater system existed on private properties in Virginia for the purpose of shore protection. In order to receive the appropriate permits, the rationale for the project was that state bottomlands would be exchanged for a stable beach with grass plantings. These provided natural habitats along the shore instead of the traditional stone revetments and bulkheads. The site has exceeded expectations, in that it experienced a severe storm just months after installation and has remained intact with little or no maintenance.

Since that time, over 100 headland breakwater systems

have been installed in Chesapeake Bay, both in Virginia and Maryland, thereby providing an extensive data set for assessing performance. The design of the system establishes the minimum protective beach width (B_m) and elevation desired in stable embayed beaches which will be held by the headland breakwaters. With B_m established, breakwater length (L_b), the breakwater gap (G_b) and the bay indentation distance (M_b) are determined from, in part, the wind/wave environment. The empirically derived relationships between these parameters offer guidelines for breakwater design along the sheltered coasts of Chesapeake Bay.

Constructing stable headland/embayed beaches for long-term shore protection can be cost-effective. The procedures developed over the years to evaluate and design headland breakwaters have been effective. The installation of these systems over the last 20 years has created a unique database. This database continues to be updated as new sites are installed, thus providing verification of performance for headland systems as sites continue to mature.

In the 1970s and early 1980s, most shore protection along eroding shorelines in the Chesapeake Bay estuarine system consisted of a wood bulkhead or a stone revetment. Today, these still are the most popular shore protection structures. Wood groins, also very common, capture littoral sands, creating a beach. If an abundant sand supply exists, groins could create a protective beach, but impacts can be significant since the littoral system is interrupted, possibly accelerating erosion downdrift. The construction of different types of structures on individual properties segments the coast as sand supply is reduced or eliminated.

In 1983, Anderson *et al.* documented the effects of a spur that was attached to a terminal groin to alleviate a severe downdrift offset (Figure 1). The spur

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was built parallel to the coast, not perpendicular (like the many groins), and its effect was to diffract the impinging wave crests and allow sand to accumulate in its lee thereby stopping the flanking of the terminal groin (Figure 1). The resultant downdrift evolution of the coast into a log-spiral, pocket beach, or crenulate embayment was empirical confirmation of various research efforts.

EARLY RESEARCH ON HEADLAND BAYS

Natural headlands and their adjacent embayment shore planform are dependent on the predominate direction of wave attack as first assessed by Sauvage and Vincent (1954) and later by Yasso (1965) and Silvester (1970, 1974). Silvester (1976) recognized the difficulty in defining the equilibrium beach to the log-spiral formula. Extensive research on crenulate bays resulted in relating the equilibrium beach planform to maximum bay indentation and incident wave angle. Silvester divided the bay into the updrift shadow reach, or logarithmic spiral, and the tangential reach. The logarithmic spiral reach is affected most by wave diffraction around the updrift headland. The tangential reach, which is slightly

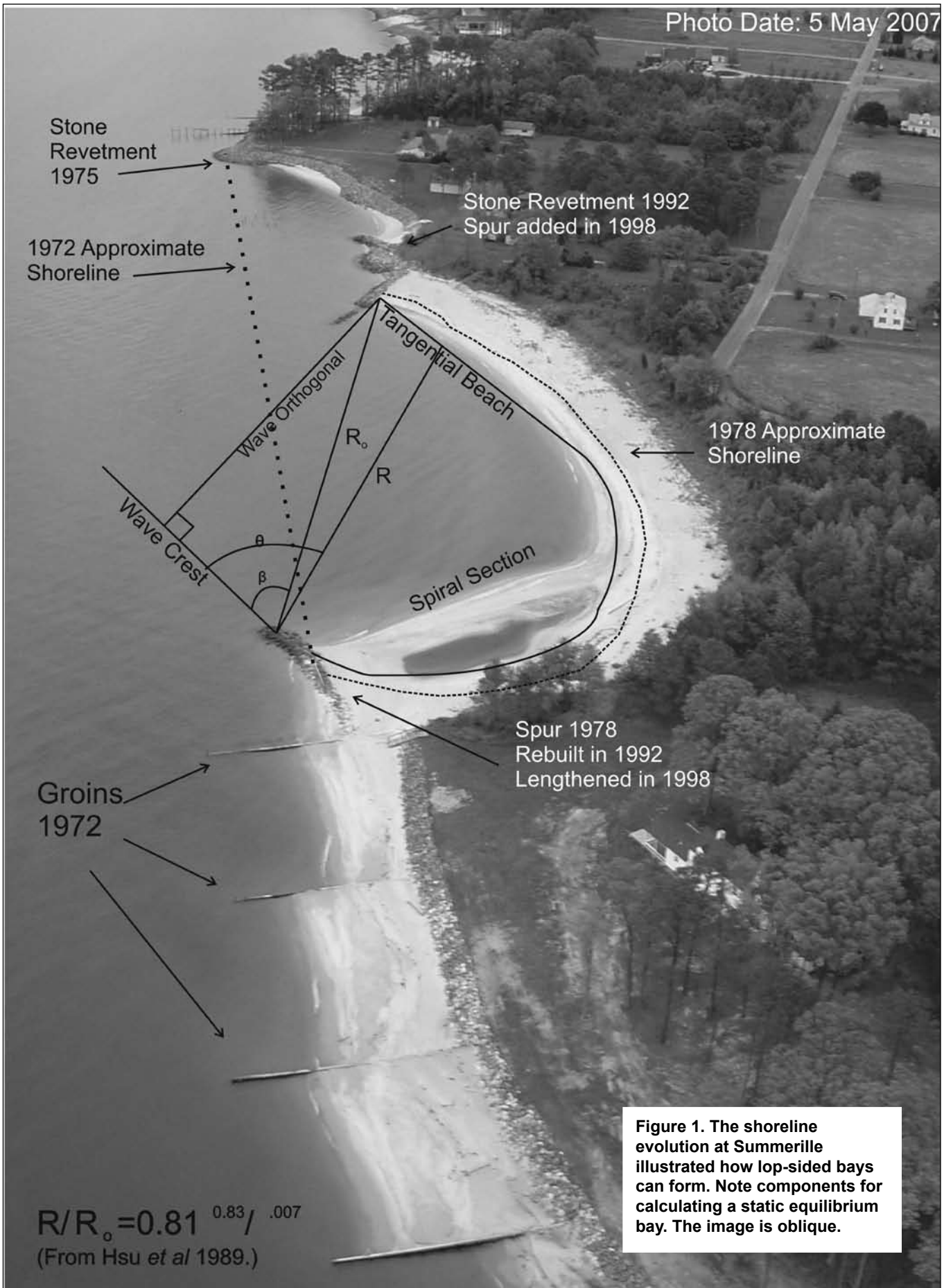


Figure 1. The shoreline evolution at Summerille illustrated how lop-sided bays can form. Note components for calculating a static equilibrium bay. The image is oblique.

Table 1.

Site characteristics and design parameters at select headland breakwater sites in Chesapeake Bay. Wave field describes the general influences of waves at the site as unidirectional (U) or Bimodal (B). Average Fetch indicates fetch length as well as dominant direction. L_b is the installed breakwater length; G_b is the installed gap between breakwaters; M_b is the maximum indentation of the embayed spiral after equilibration; B_m is the beach width after equilibration. These parameters are defined on Figure 3. L_b: G_b and M_b: G_b is ratio between parameters. Site parameters are not always equal along a given site so an average value is provided.

ID	Site Name	Wave Field	Install Date	Average Fetch (km)	Number of BWs	Number of Bays	L _b (m)	G _b (m)	M _b (m)	B _m (m)	LB:GB	Mb:GB
1	Drummond Field	U	September 1985	6 to SW	6	5	27	55	23	9.1	1:2	1:2.4
2	Aquia Landing Beach	U	March 1987	7 to E	4	5	34	49	20	18.3	1:1.5	1:2.5
3	Claiborne	U	April 1988	11 to SE	4	4	26	31	18	4.6	1:1.2	1:1.8
4	Christ	Bi	July 1988	3 to NE	3	2	21	24	17	7.6	1:1.3	1:1.5
5	Elm's Beach	Bi	October 1988	30 to NNE	3	2	47	53	46	13.7	1:1.1	1:1.6
6	St. Catherine's Island	U	March 1989	7 to NW	3	4	31	29	21	21.3	1:1	1:1.4
7	Ware	Bi	September 1989	3 to E	3	3	18	20	14	10.7	1:1	1:1.4
8	Dietrick	Bi	October 1989	3 to SE	4	4	20	29	18	9.1	1:1.5	1:1.6
9	Carden	Bi	December 1989	17 to NNE	4	4	34	34	24	12.2	1:1	1:1.4
10	Rock Hall	Bi	June 1990	11 to SW	1	2	18	25	20	24.4	1:1.4	1:1.3
11	Yorktown	U	August 1994/1997	18 NE	2 to 2	1 to 3	49	91	52	15.2	1:1.8	1:1.9
12	Asbury	U	December 1995	6 to NW	4	5	34	58	37	21.3	1:1.8	1:1.7
13	Kingsmill	Bi	March 1996	11 to SW	7	6	53	64	52	21.3	1:1.2	1:1.2
14	Van Dyke	Bi	September 1997	12 to N	9	8	27	40	23	15.2	1:1.4	1:1.7
15	Luter	U	May 1998	7 to NW	13	12	29	49	31	15.2	1:1.7	1:1.6
16	Patuxent River NAS Fuel Pier	Bi	October 2001	6 to NW	5	4	45	55	35	17	1:1.2	1:1.6
17	West Bank	U	December 2002	11 to SE	8	7	50	100	40	15	1:2	1:2.5
18	Patuxent River NAS Gate 4	Bi	November 2004	3 to NE	5	4	50	40	30	14	1:1.3	1:1.3
19	Davis	Bi	October 2005	32 to SW	3	2	55	64	26	18.2	1:1.2	1:2.5
20	Clarke	U	May 2008	24 to SW	3	2	67	50	38	18.2	1:1.3	1:1.3

convex seaward of straight, is affected mostly by wave refraction and generally aligns with the dominant or net direction of wave approach.

Rea and Komar (1975) studied log-spiral bays through numerical modeling, and found that the shoreline always will attempt to achieve an equilibrium configuration which is governed by the patterns of offshore wave refraction and diffraction and by the distribution of wave energy flux. If the system is closed, then true equilibrium is achieved when the shoreline takes on the shape of the wave crests and breaker angles are zero everywhere. If the system is not closed and sediment continues to be transported downdrift, equilibrium will occur where the breaker angles are precisely those that transport sediment eroded from the updrift section of the beach. Under this definition of equilibrium, the shoreline continues to erode but retains its overall shape.

Everts (1983) recognized the difficulty in using a logarithmic spiral shape is that the center of the spiral must be

established by trial and error. He noted for equilibrium, crenulate-shaped bays to form, there must be a fixed downdrift boundary. Without one, the rate of sediment loss will not decrease progressively with time after headland or breakwater construction. Only with a fixed boundary will the alongshore length of the bay be controlled and the total volume fixed. However, the downdrift boundary does not have to be a littoral barrier. It must, though, provide a fixed limit of the bay such that the angle between the equilibrium-tangent-sector-alignment and the pre-construction shoreline becomes constant at the downdrift boundary until equilibrium conditions are reached.

Suh and Dalrymple (1987) noted that when the gap between diffraction points (i.e. the ends of adjacent breakwaters) becomes approximately twice the incident wave length or more, the shoreline behind each breakwater responds independently as if there is no interaction among breakwaters. This mechanism might provide the response of the tangential section of spiral bays or pocket beaches as it orients

itself into the dominant direction of wave approach. Wave length is an important parameter in wave diffraction and wave refraction both of which are important mechanisms in breakwater wave attenuation and pocket beach configurations.

Hsu *et al.* (1989 a, b) and Silvester and Hsu (1993) determined that defining the headland bay curvature through the log-spiral method was not precise and should be replaced with empirical relationships. These new relationships revolve around what was called a static equilibrium bay (Figure 1). The line (R_0) joins the point of diffraction (on the structure) to the downcoast limit of the bay and is termed the "control line". Its angle to the incident wave crests is the obliquity of the waves (β), which is the only input variable that determines bay shape. This angle (θ) is the same as that between R_0 and the downcoast tangential beach when the bay is in static equilibrium.

Numerous studies, as documented in Chasten *et al.* (1993), show that, as a breakwater is lengthened relative to its distance offshore, a tombolo is more

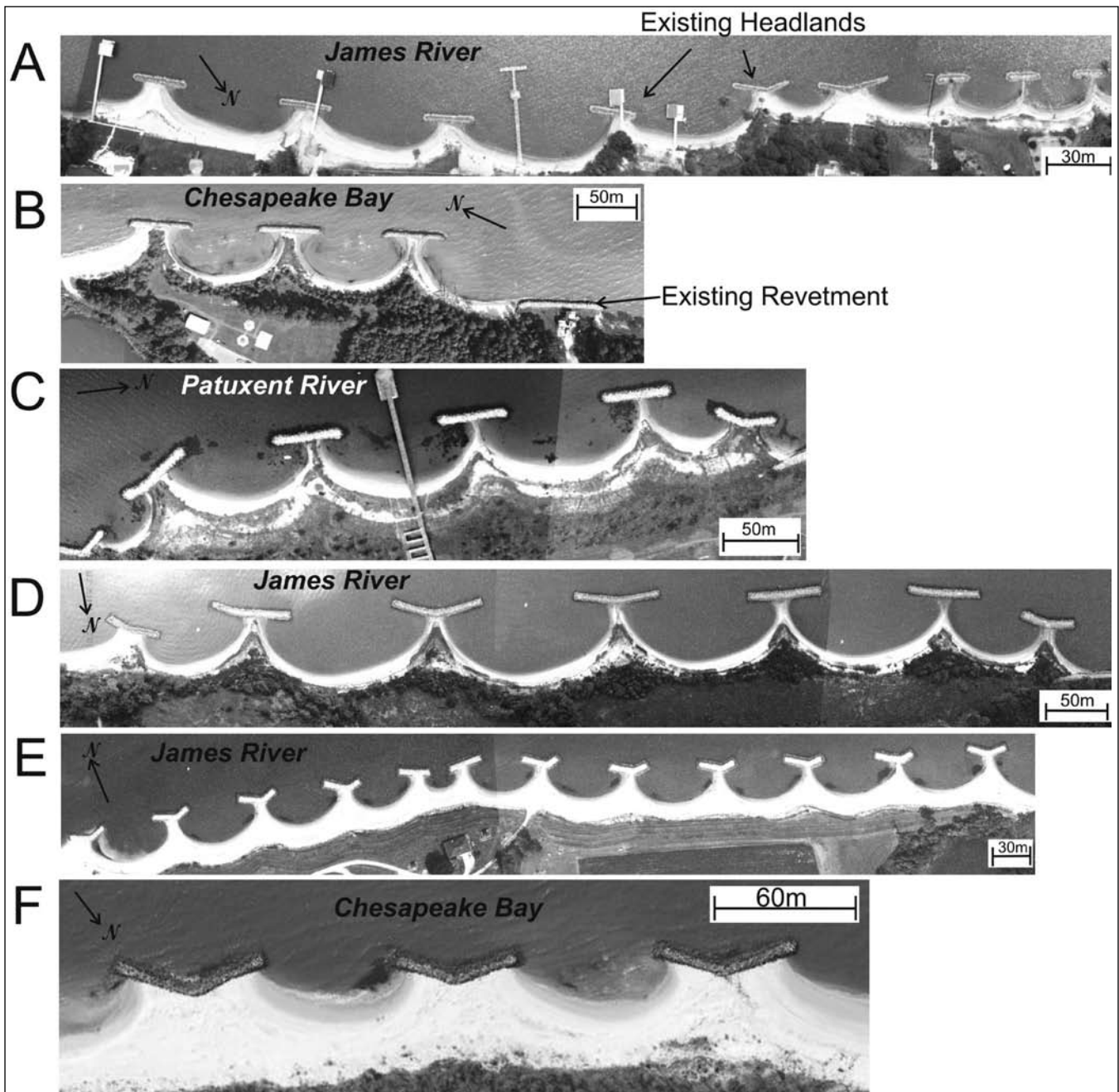


Figure 2. Headland breakwaters in Chesapeake Bay. A) Drummond Field 20 years after installation of six breakwaters on the James River (photo: 30 June 2005). B) Elm's Beach 23 years after installation of three breakwaters on the Chesapeake Bay (photo: 20 October 2005). C) Asbury 10 years after installation of four breakwaters on the Patuxent River (photo: 20 October 2005). D) Kingsmill 8 years after installation of seven breakwaters on the James River (photo: 26 August 2004). E) Luter 4 years after installation of 13 breakwaters on the James River (photo 18 September 2002). F) Davis 1 year after installation of three breakwaters on the Chesapeake Bay (photo: 2009).

likely to develop. A tombolo (sand behind the breakwater) is an essential element in headland breakwater systems although the degree of sand attachment between breakwater unit and the shore can vary. In Chesapeake Bay, the tombolo often must be created with beach nourishment since the natural supply of sand generally is limited. Hardaway and Gunn (2000) found that when breakwater length approaches double the design wave length, the structure can better hold a tombolo,

particularly when the breakwater acts as a headland in multiple breakwater unit systems.

HEADLAND BREAKWATERS IN CHESAPEAKE BAY

The first headland breakwater system in Chesapeake Bay was built in 1985 at Drummond Field on the James River in James City County, Virginia. At that time, personnel at the Virginia Institute of Marine Science (VIMS) were researching

stable pocket beaches where the natural evolution of the shoreline into spiral embayments provided practical guides of bay shape.

Historical aerial imagery of the Drummond Field site showed the development of a log-spiral type embayment between two naturally occurring headlands at the project site. Through some trial and error, the relationship between the gap, the headlands, and the general shore shape

was applied to the rest of the project site and a series of man-made headlands with beach fill was constructed (Figure 2A).

Until that time, no extensive breakwater systems existed on private properties in Virginia for the purpose of shore protection. In order to receive the appropriate permits, the rationale for the project was that state bottomlands would be exchanged for a stable beach with grass plantings which provided natural habitats along the shore. The site has exceeded expectations, in that it experienced a severe storm just months after installation and has remained intact with little or no maintenance.

At the next project (Aquia Landing, Stafford County, Virginia, 1987), the alignment of sand in series of groins at the site provided the net direction of wind/wave impact. The project was designed using slightly larger breakwater lengths (L_B) and similar spacing of the gaps between breakwaters (G_B) than at Drummond Field (Figure 3). These sites both are set in unidirectional wind/wave settings, and the relationship between the maximum indentation of the embayed spiral (M_b) and breakwater gap (G_B), (M_b/G_B) was 1:2.4 at Drummond Field and 1:2.5 at Aquia Landing (Table 1). The locations of all the sites discussed in this report are shown in Table 2.

Three sites — Claiborne, Christ, and Elm's Beach — were designed and constructed in 1988. Christ and Elm's Beach were considered to have bidirectional wind/wave climates (Table 1). When plotting the log spirals from two wind/wave directions (i.e. bimodal), a design concern was losing the beach nourishment during shifts in wave direction. To solve this problem, deeper bays were designed for Elm's Beach on Chesapeake Bay (Figure 2B). Impacts to adjacent shoreline were a concern especially on the higher wave energy coast. At Elms' Beach an existing revetment transitioned to the downdrift coast (Figure 2B).

In 1989, four projects were designed and installed — three in Maryland and one in Virginia. The Maryland projects — St. Catherine's Island, Dietrick, and Ware — were funded in part by the Maryland Non-Structural Program. St. Catherine's was built to hold dredge material and thus had decreased gap widths while Carden, in Virginia, was built to protect a spit. Dietrick and Ware are near the earlier

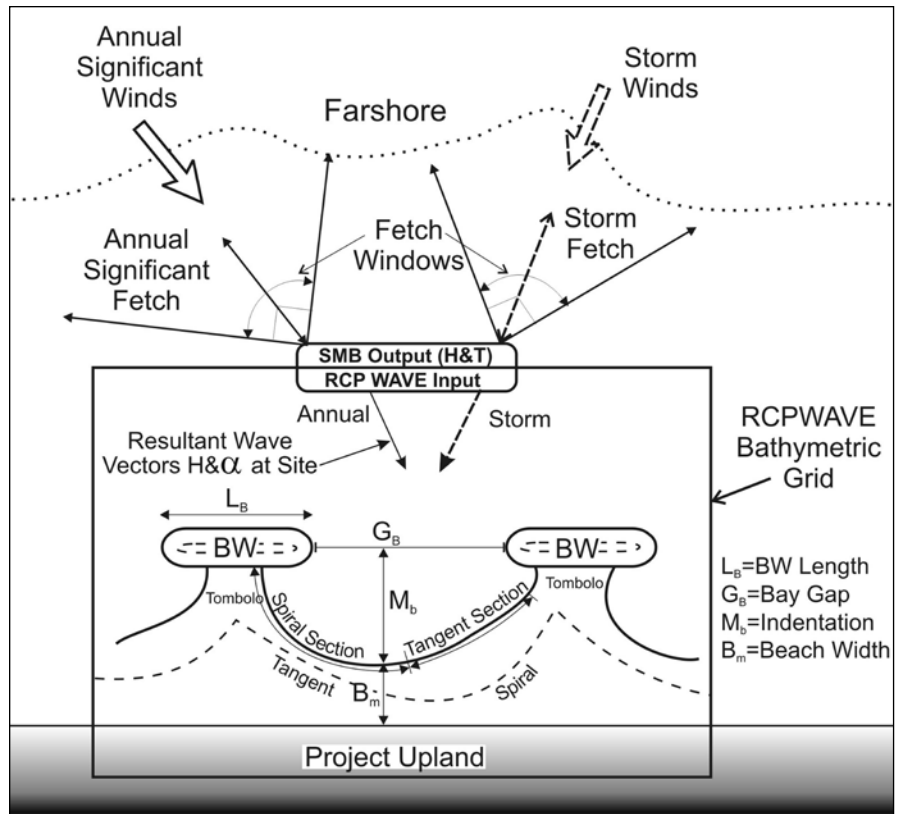
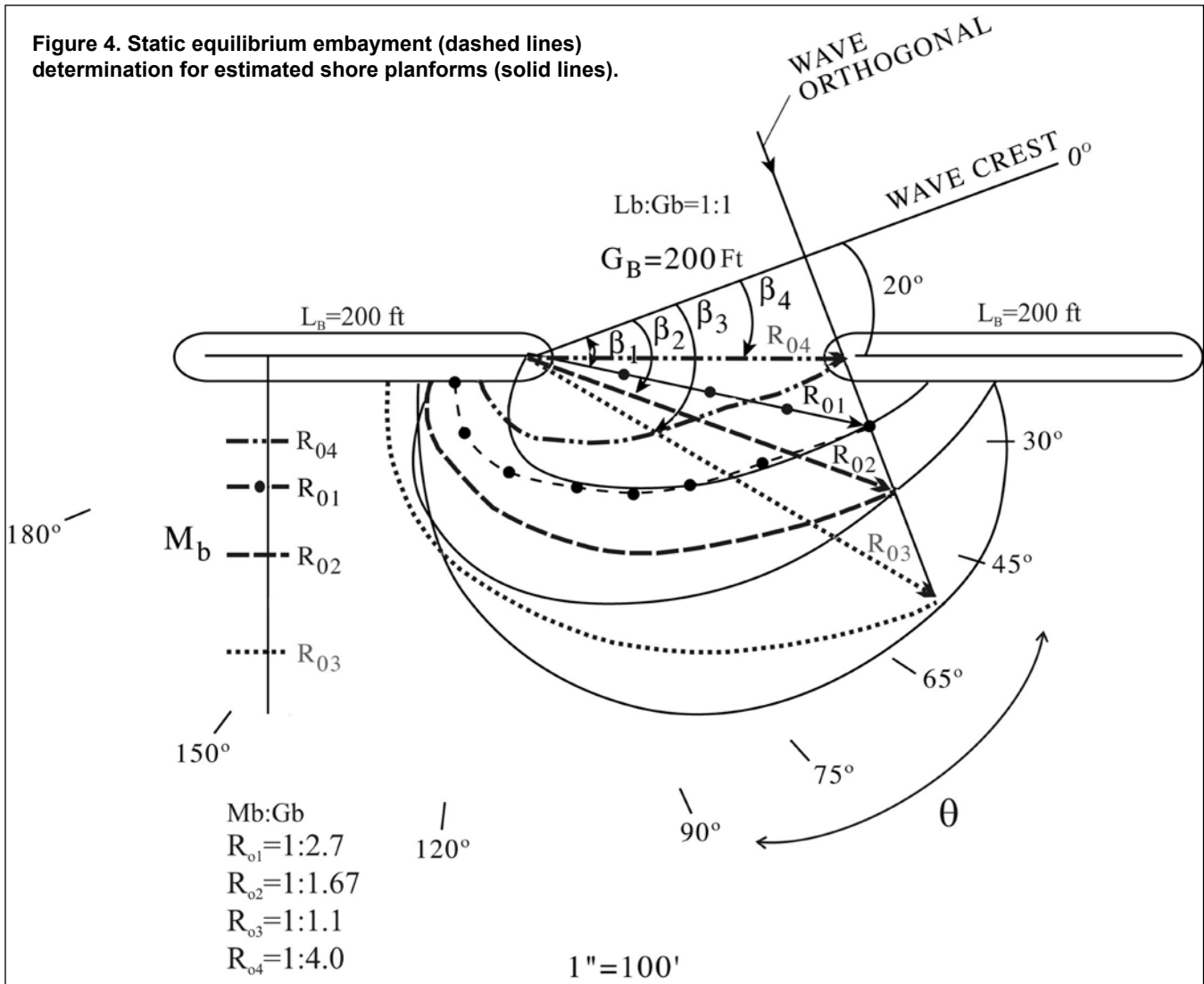


Figure 3. Definition sketch of wind/wave modeling and pertinent headland bay parameters. Parameters related to win/wave generation (SMB), nearshore wave refraction (RCPWAVE) and beach planform prediction (SEB).

Table 2. Locations of sites discussed in the report.

Site	Latitude	Longitude
1 Drummond Field	37 14' 01.67" N	76 47' 48.76" W
2 Aquia Landing Beach	38 23' 16.22" N	77 18' 56.96" W
3 Claiborne	38 50' 9.94" N	76 16' 44.97" W
4 Christ	38 39' 56.26" N	75 57' 30.94" W
5 Elm's Beach	38 11' 47.73" N	76 21' 57.89" W
6 St. Catherine's Island	38 14' 26.65" N	76 47' 51.07" W
7 Ware	38 37' 21.98" N	75 59' 32.46" W
8 Dietrick	38 37' 54.67" N	75 59' 24.43" W
9 Carden	38 2' 35.37" N	76 31' 22.24" W
10 Rock Hall	39 8' 14.26" N	76 15' 23.82" W
11 Yorktown	37 14' 10.94" N	76 30' 21.49" W
12 Asbury	38 20' 45.51" N	76 28' 12.78" W
13 Kingsmill	37 13' 18.91" N	76 39' 19.02" W
14 Van Dyke	37 2' 10.94" N	76 36' 57.48" W
15 Luter	37 1' 37.02" N	76 35' 2.88" W
16 Patuxent River		
NAS Fuel Pier	38 18' 16.31" N	76 25' 17.43" W
17 West Bank	37 2' 6.6" N	76 37' 58.29" W
18 Patuxent River NAS Gate 4	38 17' 36.02" N	76 27' 18.67" W
19 Davis	37 10' 56" N	75 59' 39.05" W
20 Clarke	37 19' 57.46" N	76 0' 45.02" W

Figure 4. Static equilibrium embayment (dashed lines) determination for estimated shore planforms (solid lines).



Christ site on the Choptank River and have a lesser wave climate and the planted backshore vegetation tends to migrate riverward with little or no beach berm.

Hardaway and Gunn (1991) and Hardaway *et al.* (1991) first developed empirical relationships (Rules of Thumb) between site parameters based, in part on linear regression analysis of key site parameters for headland breakwater sites (Table 1). These included bay indentation (M_b), breakwater length (L_B) and breakwater gap (G_B) (Figure 3). They found that the breakwater length to gap ratio ($L_B:G_B$) had an R^2 of 0.63. However, the M_b to G_B relationship was very good with an R^2 of 0.89.

Other parameters, in particular the mid-bay backshore beach width (B_m) and backshore elevation, also are important. The design beach at the site must be a certain width and elevation to protect adjacent upland banks from a specified

storm surge and the accompanying wave energy. However, (B_m) is quite variable among study sites and was not directly comparable to the above three parameters due to different applications (*i.e.* public beach and private property).

From a management perspective problems arise when coastal processes are altered by manmade structures such as breakwaters, groins, bulkheads or revetments (Kraus 1988, Everts 1983, and Pope and Rowen 1983). These modifications can accelerate erosion on adjacent shorelines by reducing sand supply or altering nearshore waves and currents responsible for sediment transport. At a minimum, the perception of adverse impacts exists when the recession rates along the protected reach are slowed or stopped relative to the unprotected adjacent shorelines. However, the "end effect" tends to become less pronounced with increased distance from the shoreline structure down coast.

According to Hardaway *et al.* (1993), three possible conditions can exist at the terminus of a breakwater system: an updrift boundary, a downdrift boundary or a boundary where onshore-offshore transport occurs. Generally, these conditions may be produced by either the "annual" wave climate or by the "storm" wave climate. The term "annual" climate is applied to those wave conditions that produce the most persistent breaker angles and wave heights capable of moving sand. These conditions occur when water elevations correspond to predicted lunar tides. By contrast, the term "storm" climate is applied to those conditions produced by major storm events that affect the project shoreline. Within Chesapeake Bay, these events generally occur as northeasters and hurricanes where water levels are typically super-elevated as a result of meteorological conditions.

Hardaway *et al.* (1995) revisited 12 breakwaters installations and applied

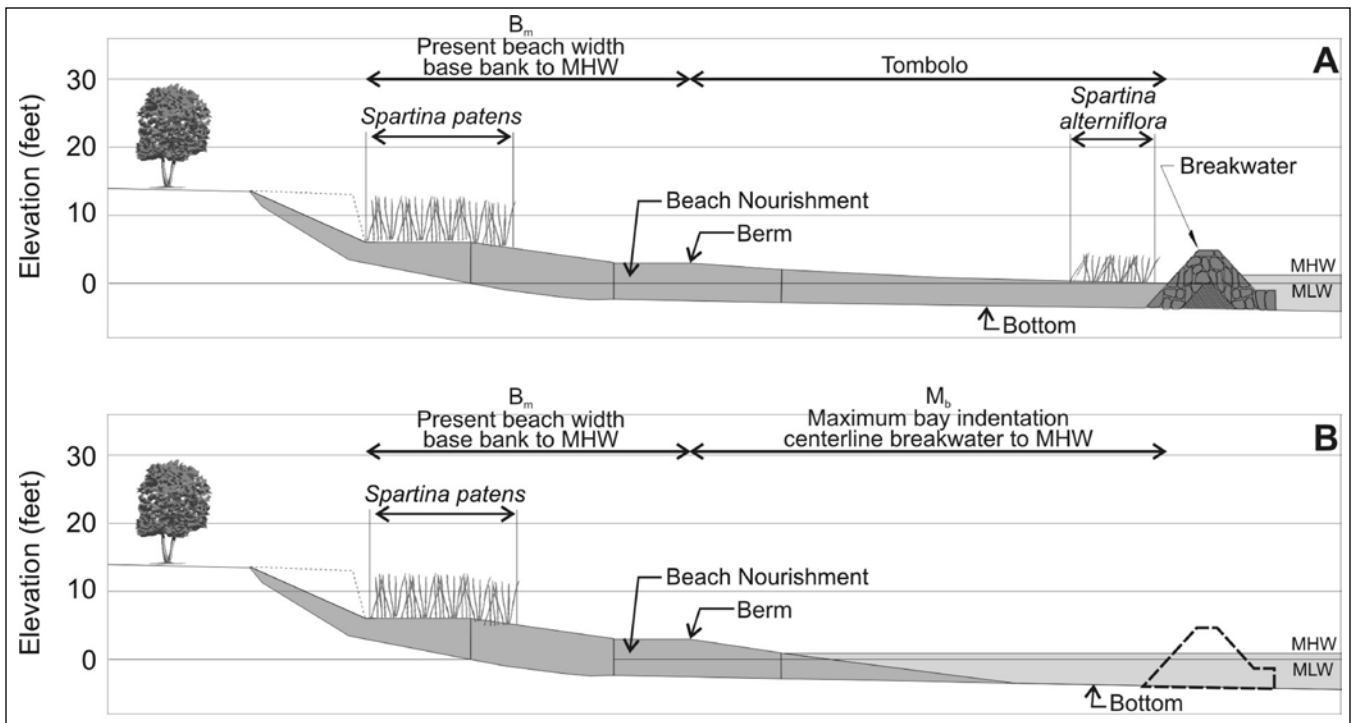


Figure 5. Typical cross-section of A) breakwater beach and B) bay beach (from Hardaway and Gunn 2000).

wave climate analyses using a combination of wave hindcasting (SMB) (Sverdrup and Monk 1947 and Bretschneider 1966) to obtain offshore wind/wave input to RCPWAVE (Ebersole *et al.* 1986), which in turn provided the net direction of wave approach (Figure 3). Comparisons were made using the Static Equilibrium Bay (SEB) model between the RCPWAVE model predicted angle of wave approach and the angle of wave approach interpreted from the beach planform's tangential section using a form of reverse wave refraction. The average deviation of beach planform between those predicted by RCPWAVE and those determined by field bay beach morphology were +2.0° from the annual condition and +4.8° for the storm condition. This means that RCPWAVE slightly under-predicts the impinging wave approach. It appears that every 1° of deviation in predicted wave vectors accounts for about 1 ft to 2 ft difference in planform position, particularly in the log-spiral section of the bays.

The design process using the three model approach of SMB, RCPWAVE, and SEB was further tested by Hardaway and Gunn (1998 and 1999). Five more sites — Asbury, (1995) (Figure 2C), Murphy (1995), Kingsmill (1996) (Figure 2D), Van Dyke, (1997) and Luter (1998) (Figure 2E) — were installed, including one that used Headland Control. The

Asbury and Van Dyke sites used sand from the bank for the beach fill. Thick stratigraphic sequences of bank sands suitable for beach fill made these sites very cost-effective. The Kingsmill and Luter sites required equal quantities of sand for the pocket beach but was obtained from nearby borrow pits.

The Murphy site was an application of Headland Control where breakwaters are built relatively far apart with only enough sand for construction. The shorelines in between the breakwaters, the embayments, are left to erode toward a state of equilibrium. This is very applicable to unmanaged wooded or farm land where the property owner cannot afford a continuous system of breakwaters or even a stone revetment. This concept and application was further illustrated by Hardaway and Gunn (2007) showing two more sites installed as Headland Control methods.

By 2000 the Drummond Field site, which was then 15 years old, was re-evaluated in terms of project performance and the assessment of the empirical relationships of site parameters L_B , G_B and M_b . In "Shore Protection: Design Guidelines for Pocket Beaches in Chesapeake Bay," Hardaway and Gunn (2000) provided a summary of parameter relationships in terms of site wave climate setting, unidirectional versus bimodal for 14 installations, eight bimodal and

six unidirectional. The results showed that, typically, breakwater systems with bimodal wind/wave exposures have breakwater length to gap ratios ($L_B:G_B$) between 1:1.0 to 1:1.5. When breakwater systems are sited in more unidirectional settings, the $L_B:G_B$ ratio is 1:1.5 to 1:2.0, particularly within embayed coastal settings that usually have an appreciable amount of natural littoral sands such as at Asbury on the Patuxent River in Calvert County, Maryland. The average $L_B:G_B$ ratio of bimodal sites was 1:1.2 whereas for unidirectional sites it was 1:1.8.

Hardaway and Gunn (1991) demonstrated that the linearly regressed relationship between breakwater gap to pocket beach indentation ($M_b:G_B$) is about 1:1.65. Further analysis by Hardaway and Gunn (2000) showed that for a unidirectional project setting the $M_b:G_B$ can average 1:1.9. For a bimodal wind/wave climate setting, the average $M_b:G_B$ ratio reduces to 1:1.4

Hardaway and Gunn (2000) concluded, with guiding relationships, that projects located in bimodal wind/wave settings should allow for what can be called omnidirectional wave attack at varying water levels. The breakwater gap (G_B) may have to be reduced relative to both breakwater length (L_B) and pocket beach indentation (M_b) so that major shifts in beach planform will adjust within the embayment. On sites

with definite unidirectional wind/wave approach, the breakwater (G_B) can be opened relative to L_B and M_b . Some ratios of $M_b:G_B$ are as high as 1:2.5, and yet the tangential feature of the pocket beach does not change significantly alongshore. The sand volume, which is the protective beach, required to be placed in the headland breakwater systems is determined by the dimensions of the breakwater system that fall within the boundaries of the aforementioned parameter relationships.

Hardaway and Gunn (2003) added two more sites on the Patuxent River — one at the Fuel Pier and the other at Solomon's Annex — to the breakwater database. Both of these sites had abundant bank sands and deep water for barge-based construction.

Hardaway *et al.* (2005) analyzed four sites in detail before and after Hurricane Isabel which impacted Chesapeake Bay on 18 September 2003. They found that heavily vegetated backshores (Kingsmill) and gently graded upland banks (Van Dyke) withstood the approximately +2.4 m MLLW water levels and associated wind/wave climate of hurricane force winds. Low upland banks experienced sand overwash but not infrastructure damage (Yorktown, Aquia Landing).

Hardaway and Gunn (2010) provided graphical representation of various bay shapes along with the corresponding SEB model shore planform predictions (Figure 4). They found that headland bays in static equilibrium can have $M_b:G_B$ ratios between 1:1.1 and 1:2.7 with the average about 1:1.65 as shown in Hardaway and Gunn (2000). Ratios about 1:1.1 can be viewed as overly stable with virtually no possibility of losing nourished sands from the system.

BEACH SAND

An essential design consideration for Headland Breakwater systems is sand since a proper beach planform provides the upland bank protection during storms by reducing wave impacts. The source of material will dictate costs and, ultimately, the design. Sometimes sand can be mined directly from an adjacent sandy bank during the grading process which will cost significantly less than sand that has to be brought to the site by truck. Bringing sand in via barge also can be cost-effective on larger projects.

Coarse sand is appropriate for constructed beaches in Chesapeake Bay. The mean grain size (D_{50}) for naturally occurring beaches in Chesapeake Bay is 0.5 mm as sampled at mean high water (MHW) at 225 locations (Hardaway *et al.* 2001). Surveys of intertidal beach slope for the same sites yielded a 12% grade or about 10:1. This research confirmed the use of these metrics in bay beach design.

Beach berms occur on “natural” Chesapeake Bay beaches and typically reside about 0.3 m to 0.6 m above MHW. The more open the site (i.e. greater fetch), the higher the beach berm is relative to MHW due, in part, to increased wave runup. Since a stable pocket beach is the goal, most headland breakwater projects have a constructed beach berm. Empirical evidence can be found on existing beaches whether natural, man-induced (i.e. jetties) or man-made for beach creation (i.e. groins or breakwaters). Bay beaches also may have a storm berm that is 0.3 m to 0.6 m above the beach berm and 1.5 m to 6.0 m landward.

Breakwater systems installed between 1985 and 2003 were generally on shore reaches with minimal sand supply and therefore required significant sand nourishment. These projects required over 25 m³/m of sand to build the minimum beach width (B_m) while the tombolos required up to 38 m³ to 50 m³/m bringing project averages 25 m³/m to 50 m³/m (Hardaway and Gunn 2010).

More recent installations along the lower Eastern Shore of Chesapeake Bay in Virginia have been along areas of more abundant sand supply due to rapidly eroding sandy bank strata. Alongshore transport rates are on the order of several thousand cy/yr with shallow nearshore flats and numerous sand bars. Two sites — Davis (2005) and Clarke (2009) — are examples of west-facing projects in Northampton County, Virginia, that faced special design considerations.

At Clarke, it was important not to interrupt the movement of sand in the offshore sand bars so the structures were built closer to shore and 765 m³ of sand was added to the “downdrift” as compensation. However, in order to maintain the relationship $M_b:G_B$ at 1:1.7 as well as provide an adequate protective beach (B_m), the upland bank at the proposed embayments had to be excavated to

increase the overall width of the system. The Davis site (Figure 2F) is close to the mouth of the bay and experiences sand movement both up and down the coast in semi-equal proportions. The high, sandy upland bank was designed to continue eroding and “feeding” the littoral system thereby allowing a more open gap to bay ratio ($M_b:G_B$) 1:2.5.

ROCK

Stone for breakwater units comes from rock quarries located along the Fall Line of Virginia and Maryland. Rock types can be granite, metamorphosed limestone, or dolomite. The rock is durable and good quality, although some poor material may occur so quality control is an important element in the construction process. Most quarries have to truck their product to site. However, three quarries — one in Havre de Grace, Maryland, the two in Richmond, Virginia — have the ability to provide rock by barge. This is a cost-effective option for sites where the nearshore is deep enough.

PLANTS

Headland breakwater systems would not be complete without the establishment of vegetation across the beach and backshore. The combination of native grasses, shrubs and trees knit the sandy substrate together into an erosion resistant turf that has shown its value during storm events (Figure 5A and B). The beach and berms provide the planting zones for upper beach and dune grasses (*Spartina patens* and *Ammopholia*). Sometimes *Spartina alterniflora* can be established on the flanks of the tombolo in the lee of a breakwater unit between mean tide and spring high water.

CONCLUSIONS

The documented, long-term performance of headland breakwater systems in Chesapeake Bay is testimony to the predictable durability of these systems. Through numerous storm events, these systems have remained intact with no significant shore erosion or changes in shore planform over time, some for as long as 25 years.

Over 100 breakwater systems have been installed around Chesapeake Bay, many using these relationships described here. Ongoing analysis of selected sites after storms and periodic re-evaluations are ongoing. The sites listed in this paper were designed with some level of performance expectation. Usually this would

be a design to maintain the integrity of the headland breakwater system for a 50-year event but built to survive a 100-year event.

The empirical relationships of L_B , G_B and M_b have been shown to be useful guidelines for headland breakwater design in Chesapeake Bay, but site-specific conditions, including geomorphic setting, access and property lines, can influence breakwater and beach position along the shore. The empirical guidelines can be summarized as follows. For unidirectional wave climate sites, the $M_b:G_B$ can range from 1:1.6 to 1:2.5 (average = 1:1.9). The $L_B:G_B$ can range from 1:1.5 to 1:2.0 (average = 1:1.8). For bimodal wave climates, the $M_b:G_B$ will range from 1:1.0 to 1:1.7 (average = 1:1.4) and the $L_B:G_B$ can range from 1:1.0 to 1:1.5 (average = 1:1.2). For the overall Chesapeake Bay estuarine system, regardless of wave climate, the overall average $M_b:G_B$ is 1:1.65 and the overall $L_B:G_B$ is 1:1.4. Other design concerns include addressing potential impacts to the adjacent coast, ensuring breakwater length approaches 2x L, and using coarse sand.

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