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DESIGN ASPECTS OF GROINS AND JETTIES

Kevin R. Bodge, Ph.D., P.E.[1], Member ASCE

Abstract: Design guidance is presented in regard to the planform and elevation of coastal structures intended to augment beach stability. These structures include groins, T-head or fishtail groins, headlands, nearshore breakwaters, terminal structures and jetties. Brief discussion is presented as to sites where such structures may, and may not, be warranted. A design protocol for structurally-stabilized shore protection projects prescribes that the structures' geometry and orientation be "tuned" to the incident wave field and the computed alongshore transport potential, where the structure-induced shoreline is approximately predicted by simple empirical means.

INTRODUCTION

Groins are shore-perpendicular coastal structures utilized to compartmentalize beaches and impound a portion of the longshore sediment transport. T-head groins (also called fish-tail groins) include a perpendicular spur or head near or at the seaward end of the groin to diffract waves and further compartmentalize the beach and reduce sediment transport. Nearshore breakwaters are shore-parallel structures that act somewhat similarly to a T-head groin (or vice-versa), except that they do not include the shore-perpendicular trunk (groin) section that typically attaches the head to the backshore. Both T-head (or fishtail) groins and nearshore breakwaters are typically intended to act as artificial headlands. When employed properly, these groin and breakwater structures can be used to augment the stability of beach restoration projects. Improperly used, they can exacerbate beach erosion.

Jetties are generally longer structures than groins and are constructed parallel to inlet channels. By definition, jetties are intended to channelize an inlet's tidal flow so as to increase the flow velocities and reduce sedimentation within the inlet channel. Properly employed, they can prevent sand from leaking into the inlet channel and being transported by waves and currents to tidal shoals (and thence potentially lost from the beach system). Improper or "leaky" jetties can exacerbate inlet-adjacent beach erosion. Terminal structures refer to either groins or jetties constructed at the end of an island, littoral cell, or beach project, where the term usually refers to the downdrift end of the cell or project.

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1 Vice-President, Olsen Associates, Inc., 4438 Herschel Street, Jacksonville, FL 32210 USA. kbodge@olsen-associates.com

This paper presents observations regarding the design of groins, jetties and terminal structures for purposes of augmenting beach stability and improving sand management at beach restoration projects and inlets. The design protocols included herein are based upon the response of sandy shorelines to such structures as observed by others, and in large part from the experience of the author's firm in the design of about twenty stabilized beach restoration projects constructed since 1990 in the southeastern U.S. and Caribbean.

In present practice, design guidance is based principally on empirical experience. Useful guidance and project descriptions are offered in Yasso (1965), LeBlond (1972), Walton (1977), Wong (1981), Pope and Dean (1986), Berenguer and Enriquez (1988), Hsu and Evans (1989), Rosati and Truitt (1990), Hanson and Kraus (1991), McCormick (1993), Silvester and Hsu (1993), Laustrup and Madsen (1994), Peña and Covarsi (1994), Hardaway et al. (1995), Chrzastowski and Trask (1997), Hardaway and Gunn (1999), Hanson and Kraus (2001), among others including additional project examples described below.

APPROPRIATE USE OF STRUCTURES

The incorporation of coastal structures to beach fill projects may be warranted in the following instances:

(1) Structures may be appropriate where erosion stress is sufficiently severe to preclude an economically- or physically-practical beach fill life. This includes sites of accelerating (erosive) longshore transport gradient and sites where the littoral drift has been interrupted or there are alongshore losses to adjacent littoral sinks (such as inlets). In these cases the rate at which the beach fill erodes may be so high that excessively frequent renourishment is required to maintain the project shoreline. Attempts to extend the renourishment interval by increasing the fill volume increase the project's width and its perturbative effect to the shoreline. This, in turn, increases the fill's loss rate so that the increased fill volume yields little net decrease of the requisite renourishment frequency.

(2) Transport from localized erosional "hot spots" to depositional "cool spots" can be potentially reduced by structures. This includes sites where structures can locally reduce *accelerating* transport gradients in order to retain sand from adjacent areas of *decelerating* transport gradient. A headland is a common example of this type of site.

(3) Structures may be warranted where the proximity of environmentally sensitive natural resources or marine structures precludes construction of a wide beach fill. This includes cases where an otherwise wide beach fill would bury nearshore natural resources such as reefs or seagrasses beds, or encroach upon (shoal) marine facilities such as docks, outfalls or water intakes. A related instance includes cases where the nearshore seabed slope is steep or abruptly drops. Each of these cases physically limits the size of the allowable beach fill; and, as such, restricts the project's life and/or its likelihood of success.

(4) Structures may be warranted where the desired beach amenity is located upon a non-littoral coastline or at the terminus of a littoral cell, and/or represents a perturbation to the existing coastline that is inherently unstable, primarily due to severe end-losses of the fill.

This includes sites where a beach is to be created anew, or where a historically sandy beach has mostly vanished due to sediment starvation, mining etc., and/or where there is no potential for adverse impacts to adjacent shorelines.

Minimizing Downdrift Impacts

Groins and similar structures will induce erosion to adjacent downdrift beaches through their interception of littoral drift and potential offshore diversion of littoral material via seaward return flows. Minimizing these impacts requires that

1. a volume of sand equal to the structures' gross impoundment effect is added to the system during project construction;
2. the structures' potential impoundment volume must be finite;
3. the project is located at the terminus of, or outside of, a littoral system;
4. the structures must not induce offshore losses extraordinary to the natural system;
5. structural fields should not terminate in a zone of *increasing* longshore transport potential.

Unless the structures are being installed to trap sediment (such as to prevent littoral drift from leaking into an inlet, etc.), then pre-filling the structural field with littoral material is essential. Specifically, the project site should be nourished with beach-compatible sediment with a volume equivalent to the project's design beach volume (or more), plus allowance for the gross impoundment effects along both shorelines adjacent to the structural field (if applicable), plus contingency allowances. This "advance-fill" mitigates the structures' tendency to impound sand from the natural system and best ensures that the design objectives will be met. Properly designed, this fill is intended to render the structural field "transparent" to the littoral drift. The beach fill should be placed as part of the project's construction, and preferably after installation of the structures.

On most long, uniform beaches, it is theoretically impossible to mitigate the long-term downdrift erosional effect of coastal structures, such as groins, without dedicated beach nourishment and/or sand bypassing. That is, from theoretical considerations, the updrift impoundment effect of structures on a long beach strand – and therefore the downdrift erosion potential – is infinite. This effect is exacerbated if the structural field promotes offshore transport, particularly during storms. In these cases, the structurally-stabilized beach may recover rapidly after a storm by intercepting littoral drift – thus contributing to the downdrift erosional impact of the structures.

Groins can divert sand offshore via rip currents; i.e., the seaward return flow of longshore current and wave mass transport that is otherwise interrupted by a groin. To reduce this deleterious effect, the use of permeable, rough, or dissipative structures is recommended. In recent practice, T-head groins are also recommended to reduce the net effect of the offshore (rip current) flow. There is not yet a significant body of research on the subject, but field experience suggests that T-head groins lend particular stability to the beach cells between the groins -- even in storms with high cross-shore transport potential.

DESIGN OF GROINS AND JETTIES

Planform Considerations

The following considerations are noted for the layout of groins to stabilize a beach fill. These comments particularly pertain to T-head (or fishtail) groins. In general, it is important to recognize that the beach responds to the *gap* between the structures moreso than to the structures' heads. That is, the beach is dictated by the waves that are transmitted to it, and the wave transmission is a function of the length and orientation of the gaps (openings) between the structures' heads. Accordingly, in prudent practice, the *gaps* are designed first – followed by the layout of the heads needed to effect these gaps. (This is somewhat analogous to designing a building – whereby the locations of the doors and windows are selected prior to laying out the framing needed to create them.)

Shoreline Prediction. Design prediction of the shoreline adjacent to headland-type structures can be accomplished through a combination of a simple empirical formula developed from experience and the "log-spiral" coastline geometries tabulated by Silvester and Hsu, 1993 (or Hsu & Evans, 1989) . The latter method, including others by Yasso (1965), Pope and Dean (1986), Benenguer and Enriquez (1988), McCormick (1993), among others, are limited in their *functional* utility in that they do not indicate the *tidal elevation* of the predicted shoreline. In practical design, this is problematic. For example, for a typical site with 0.8 m tide range and 1:10 beach slope, this introduces up to 8-m uncertainty as to the design shoreline location. This represents a fairly significant portion of the project's overall offshore width (15% to 25% typically), which in turn, represents a significant impact to the project's cost, regulatory review, and aesthetic. The author's experience suggests that these previous investigators' observations appear to best apply to an elevation between mean low water (mlw) and mean tide. For conservative designs, one might presume that the mlw shoreline is predicted. For less conservative designs, an elevation midway between mlw and mean tide level may be presumed, though the designer must be cognizant that this lessens the project's design contingency.

For preliminary design, a simple empirical rule is that *the mean low water (mlw) shoreline will be located between about one-third and two-thirds of the structures' gap distance behind the structures' seaward face.* Specifically, in **Figure 1a**, the structures' gap distance, *G*, is the opening (or "control line") distance between adjacent structures or headlands. Here, the mlw shoreline is roughly estimated as a line parallel to the gap opening (control line), and located a distance γG behind it, where $\mathbf{0.35 < \gamma < 0.65}$. The shoreline in the lee of each headland is estimated as a circular arc of radius γG with center at the control line's endpoints. Similarly, in **Figure 1b**, the structures' gap distance (control line) is the distance *G* between a headland and a downdrift point of stability. Here, the mlw shoreline is a line that begins at a distance γG behind the headland and ends at the downdrift "stable" point.

In practice, the value of γ is determined by the designer from experience and as a function of numerous considerations. Larger values of γ within the 0.35 to 0.65 range (or potentially beyond that range) are appropriate for (1) more energetic environments (e.g., open ocean coast), (2) larger gap distances (e.g., > 55 m), (3) uncertainty as to long-term sediment supply or potential fill losses, (4) uncertain or poor beach fill compatibility, (5) shore-detached headland structures (i.e., no groin stem), and/or (6) greater desired level of conservatism.

Smaller values of γ within this range may be appropriate for (1) less energetic environments (bays, harbors), (2) smaller gap distances (<45 m), (3) T-head groins (i.e., shore-attached stem), and/or (4) greater allowable tolerance in the post-project beach planform, particularly where structure length or size is limited. This "γ shoreline" rule of thumb, originally termed the "one-third rule" (Bodge 1998) because of the minimum γ=1/3 value, is considered applicable where the angle between the wave crest and the structures' control line is small (say, < 25^o to 30^o).

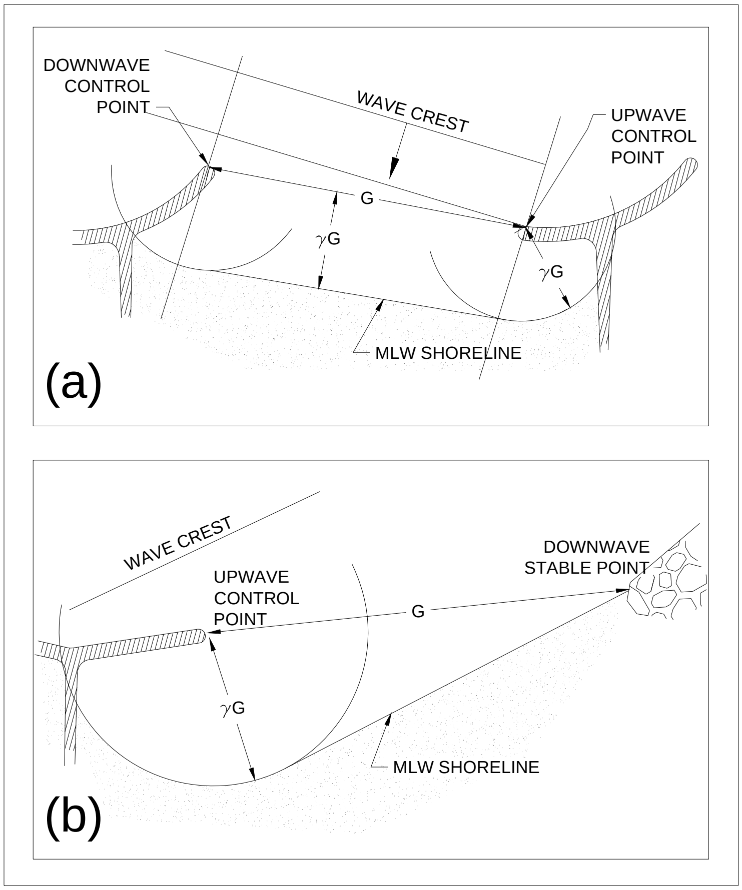

Fig. 1. Prediction of the mlw shoreline by the "γ shoreline" rule-of-thumb.

The endpoints of the gap, or control line, are usually taken at about the mid-tide elevation of the structures' ends. For final design, the shoreline prediction can be made from a composite-average of the simple "γ-shoreline" rule and the crenulate-bay prediction of Silvester & Hsu, 1993. See **Figure 2**. In this approach, the upper beach contours are drawn as a uniform offset from the design mlw shoreline.

The stability of the beach fill is better ensured when *flanking* of the structures' heads is prevented. In this way, as illustrated in Figures 1 and 2, the heads' trunks (groin section, or "stem") are extended at least as far landward as the berm (i.e., the predicted limit of nominal wave run-up at high tide), and buried within the beach fill. As noted above, values for γ in the

0.35 to 0.65 range presume the use of such stems. Larger γ values within or beyond this range may be appropriate for shore-detached headlands (breakwaters), particularly as their distance from the original shoreline increases and/or as the gap width between headlands increases.

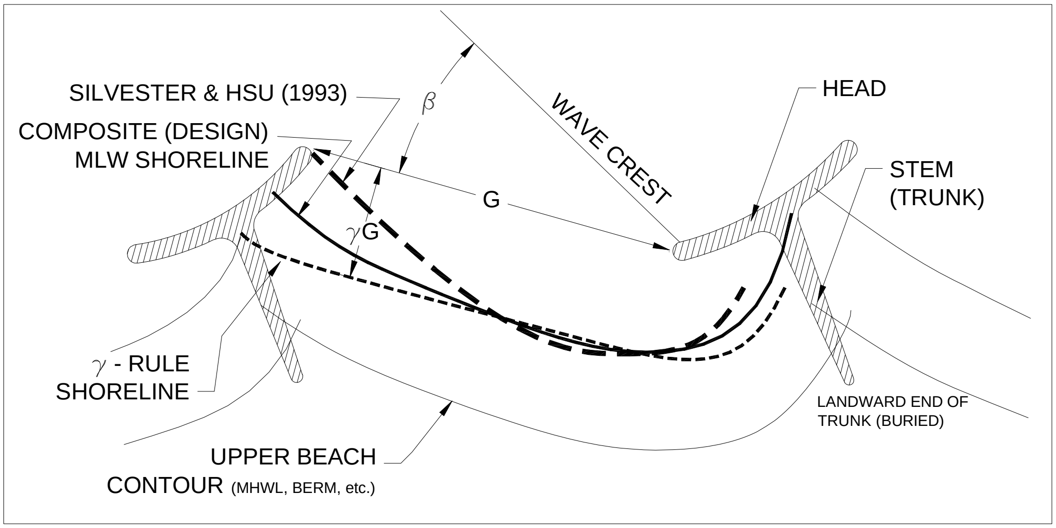

Fig. 2. The "design" mlw shoreline is an average of that predicted by the "γ shoreline" rule (Fig. 1) and Silvester & Hsu (1993) log-spiral geometry.

A surrogate (or maximum) measure of γ is the "indentation ratio" of the pocket beach shoreline between headlands, A_1/G, as illustrated in **Figure 3**, where A_1 is the maximum distance between the gap and the shoreline. **Figure 4** depicts measured values of A_1 versus G for projects constructed in Spain (Berenguer and Enriquez, 1988), the Chesapeake Bay (Hardaway et al., 1995), and various typical projects designed by the author in the southeastern U.S. and Caribbean. The scatter in the data reflects significant variations in project designs and site conditions, including the degree to which the projects are filled with sand during construction, and the overall littoral supply of the project site.

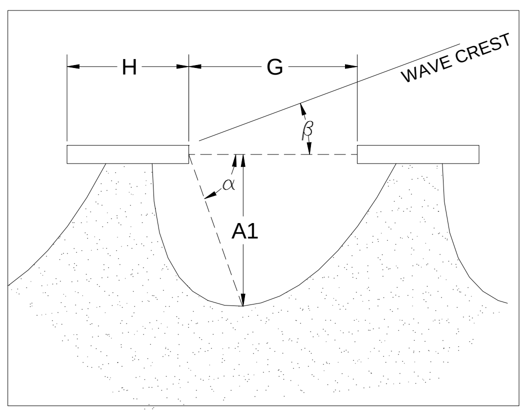

Fig. 3. Shoreline indentation A_1 and structure gap width *G*.

In Figure 4, the projects in Spain include both shore-attached and detached structures. From their data, Berenguer and Enriquez (1988) suggest the empirical relationship $A_1 = 25$ *(meters)* $+ 0.85\,G$. The Chesapeake Bay projects are typically all detached breakwater structures; and, the mean and standard deviation of these data are $A_1/G = 0.62 \pm 0.16$. The tide ranges for both the Spain and Chesapeake projects are modest (<0.85 m); however, the tidal shoreline that is reported is not certain. The data points from the author's projects feature all shore-attached T-head structures and "saturated levels" of beach fill placement during construction (i.e., filled to the predicted beach geometry). These data reflect the mlw shoreline, and the sites' tide ranges vary from 1 to 3 m. The mean and standard deviation of the author's data are $A_1/G = 0.34 \pm 0.13$. All three of the datasets reflect shoreline conditions several years after project construction.

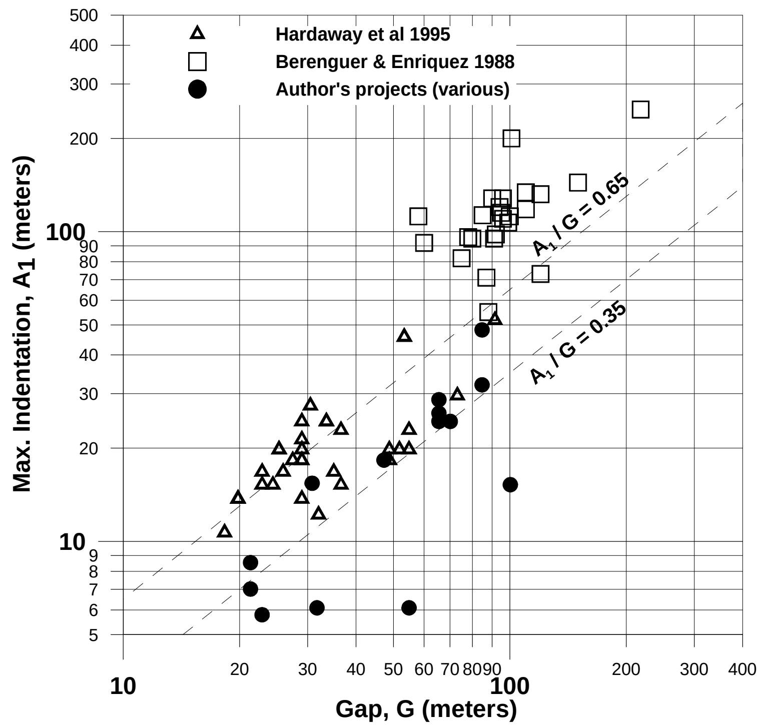

Fig. 4. ***A_1* versus *G* for example constructed projects.**

Silvester and Hsu (1993) predict an indentation ratio of $A_1/G = 0.15$ to 0.65 for waves approaching at an angle β = 10 to 90 degrees, respectively, to the gap opening.

Design Approach. Key steps in the planform design of a structurally stabilized beach fill are highlighted below.

1. Estimate the longshore sediment transport potential and the breaking wave angle for the site's principal incident wave conditions at discrete locations along the general project area. Compute the average annual (or seasonal) net and gross longshore transport potential and breaking wave angle along the project area. While hindcast or other data are of certain value in this regard, an exhaustive study or data collection effort is often not necessary. That is, the site's nearshore bathymetry may act to transform most offshore wave conditions

(particularly the most common wave occurrences) to a fairly narrow angular window that is sufficiently useful for the project's lay-out.

2. Identify the appropriate alongshore limits required for the structural field. These limits are typically defined by the terminus of a littoral cell and/or the beginning and end of transport gradients, as practically modified by riparian property limits. Unless ending at a terminal structure or end of a littoral cell, the downdrift end of the structural field should terminate in a stable or decelerating longshore transport gradient.

3. Identify the minimum target (design) location of the post-project berm. This is the line beyond (seaward of) which the project's predicted berm location should ideally fall for all incident wave angles of interest.

4. Estimate the probable post-project slope of the beach. Predict the distance W between the mean low water (mlw) and mean high water (mhw) shorelines. Predict the horizontal distance S between the mlw shoreline and the probably berm contour. These beach slope and beach width values may be estimated from existing or adjacent beaches; however, for design purposes the average post-project beach slope within a structure field, between the berm and mlw, is often slightly gentler-sloped than on a non-stabilized beach. This is counter-intuitive because the overall profile of a seaward-advanced shoreline must be steeper in order for the offshore contours to close. Nonetheless, in practice, the decreased (diffracted) wave energy that reaches the beach between the structures' gaps appears to result in a slightly relaxed berm and intertidal beach slope.

5. Identify the number of beach cells, n, and average gap width, G, and structure head width, H. From **Figure 5**, the total project shorefront length, L, is composed of n beach cells:

$$L = n\,(G+H) \tag{1}$$

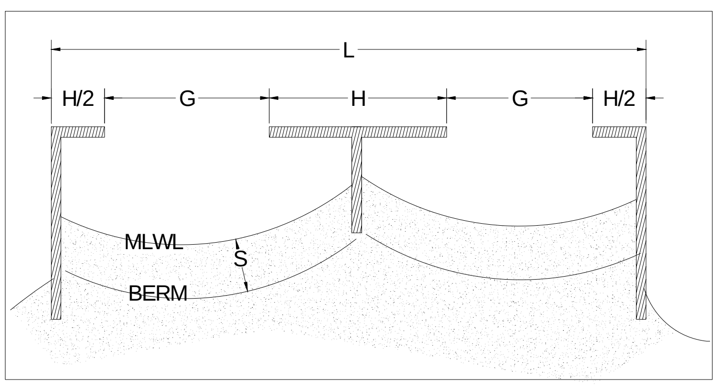

Fig. 5. Pocket beach cells.

As a *minimum*, the structures' head-width, H, should be large enough to ensure that the mean *low water* shoreline reaches the head. From the "γ shoreline" rule, this requires that $H = 2\ \gamma\ G$. A more *conservative* design requires that the mean *high water* shoreline reaches the head. This requires that $H = 2(\gamma\ G + W)$. In general, then,

$$H = 2\ (\gamma\ G + X) \qquad \text{where } 0 \leq X \leq W \tag{2}$$

Substituting (2) into (1),

$$G = (\ L\ /\ n - 2X\ )\ /\ (2\gamma + 1) \tag{3}$$

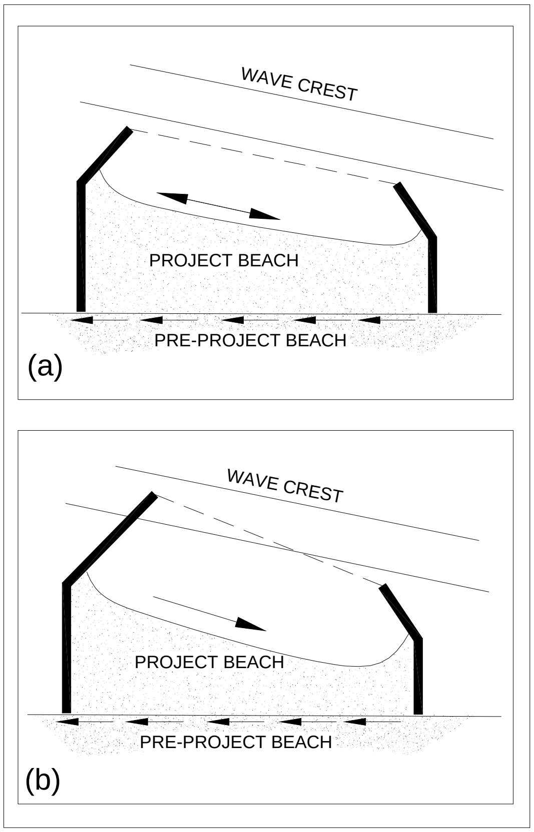

Fig. 6. Aligning the opening between the s, the wave crests may induce a null or reversal transport directions within the beach cell.tructures (a) parallel with, or (b) against the wave crests may induce a null or reversal transport direction within the beach cell.

In practice, one selects a number of beach cells, n, for a presumed value of γ and desired value of X in order to develop a physically reasonable gap width, G. Gap widths less than about 22 m are not generally recommended (at least for recreational beaches). Small openings present a physically "restrictive" aesthetic, and in some instances, can cause strong offshore-directed return flows through the narrow gap (particularly during long-period or narrow-spectrum swell). Gap widths greater than about 100 m have not been tested by the author in the prototype. Additionally, for recreational beaches, one attempts to balance values of γ and X such that the head widths are smaller than the gap widths ($H \leq G$). This is principally a socio-aesthetic consideration; viz., an attempt to create a beach horizon that is "more sea than structure".

6. Initially locate the gaps and heads along a line that is $S + \gamma\ G$ seaward of the target (design) berm location.

7. Orient (rotate) the gaps to be more closely parallel with the average, or principal, crest orientation of the breaking waves at each cell; that is, to minimize the wave crest angle β in Figure 3. This initially fixes the ends of each gap, between which the structures are drawn. The nominal objective is to align the heads' ends so that the gap openings are *parallel* to the principal wave angle at each opening (**Figure 6a**). In this case, the angle between the wave crest and the

structures' opening is zero, and the "γ -shoreline" rule is readily employed to predict the mlw shoreline. (In contrast, Silvester & Hsu (1993) and other log-spiral predictors are not well defined for angles less than about 15^o.)

Where practicable, the stability of the beach fill is potentially enhanced by "over-correcting" the alignment of the heads' ends so that the transport direction within the beach cell is *reversed* from its expected, open-coast sense (**Figure 6b**). This design acts to drive the fill material "updrift" so to speak. By intent, this design is less transparent to littoral drift, and may be inappropriate where downdrift erosion is of concern. For identically-sized structures, those with head orientations "tuned" to the local wave energy in this manner yield potentially greater shoreline stability (smaller longshore transport potential within each cell) and greater net beach area than groins with traditional shore-parallel heads (Bodge, 1998).

If the terminal structure in a field employs a head or spur on its downdrift side, it is better offset *seaward* than landward. While intuition suggests that a *landward* offset is a more natural transition from the structural field to the downdrift shoreline, it inherently induces a crenulate bay in the structure's lee that will erode into the native beach (see Figure 1b). Attempts to fill (nourish) this crenulate bay may be fruitless -- as the embayment shape is an irreconcilable result of the structures' proximity to the shoreline.

7. Re-predict the shoreline for the draft lay-out of the structures' heads and gaps, utilizing a composite of the "γ -rule" shoreline and other methods as appropriate; and adjust the lay-out if needed. Once satisfied that the project lay-out's predicted shoreline and berm will satisfy the "target" shoreline for the *principal* wave direction (and/or, that the predicted toe of the beach will not encroach seaward beyond a specified limit), then the shorelines and berm locations are predicted for *extreme* wave directions. This aims to assess the degree to which the structures might be exposed -- or the target shoreline(s) violated -- by seasonal or storm events where the waves deviate from their principal (average) direction.

Adjustments may be necessary to accommodate the wave extrema; and/or, should at least result in recognition of the limitations or risk of the project's performance in the event of these extrema. Additionally, alternate lay-outs are evaluated that might retain fewer (larger) structures or more numerous (smaller) structures. In this way, the lay-out of the structural field is typically an iterative process.

Traditional Groin Lay-Out. Particularly where the structures are intended to *promote* fill stability in a littoral environment rather than to *provide* it, the structures' heads may be reduced to short spurs – ultimately yielding minor T-heads rather than headlands. When reducing the heads lengths' in this way, the point at which the shoreline behavior changes from a headland response, described above, to a groin-type response is not yet clear (at least to this author). While it is generally accepted that the shoreline orientation updrift of a groin will be mostly parallel to the principal wave crest, there is little design guidance as to where this shoreline *location* will fall relative to the seaward end of the groin. In areas that are not sediment deprived, there is some limited prototype indication that the updrift shoreline contour intersects the seaward end of the groin at one-half to one tide range below mlw (i.e., 1 to 1.5 times the tide range below mean tide level). The locations of higher beach contours are subsequently estimated by a simple upslope translation from this subtidal elevation -- by reference to the ambient beach slopes from nearby locations. Generally, however, design of groin fields mostly continues to follow the traditional guidance outlined in the Shore Protection Manual (USACE 1984), where alongshore spacing is recommended at 2 to 3 times the active groin length measured from the berm crest to the seaward head.

Structure Elevation

For T-head and similar headland structures, the elevation of the heads should be established so as to minimize wave overtopping during the design event of interest. At a minimum, this would include at least typical seas at higher high tides. The elevation of the stems (trunks) may be lower than the heads, but of sufficient elevation to prevent flanking of the heads (or at least tidal overtopping) at higher high tides.

The profile elevations of the terminal structures should be higher than the design beach profile predicted adjacent to (upwave of) the structure. In the case of a rock structure, then the crest elevation should be *not less* than ½ armor stone diameter above the predicted design profile (plus some contingency). This consideration arises because the irregular surface of the structure crest, formed by the rocks, is inherently porous to sand transport. That is, a rock structure with crest elevation specified as "z" meters will clearly not be sand-impermeable at elevation = "z" meters.

Particularly in the case of a jetty or terminal structure, the elevation of the landward end of the structure (landward of the beach berm), should be as high or higher than a "healthy" beach profile, including the dune, projected onto the structure's location. This consideration represents a fundamental shortcoming of many older jetty structures – where the structure is routinely overtopped by wind-, wave-, and tidal-washed sediment because the structure is far too low relative to the adjacent beach and dune. In these cases, these older structures were apparently designed principally to "jet" the tidal currents within the inlet with little regard of the potential for sand from the adjacent beaches to be transported through and over the structures. Identifying and repairing this problem is often a simple and significant means by which to improve sand management at the inlet and adjacent beaches.

Sand-Tightening and Other Design Considerations

Except where a sand weir is indicated, terminal structures (including those at the end of a T-head field) should be "sand-tight"; i.e., impermeable to sand transport. Methods for sand-tightening include (1) specifying use of small chinking stone to create a core and to fill voids between armor stone, (2) placement of a composite geogrid/geotextile as an internal membrane within the structure, extending from the adjacent beach grade to the structure's crest, and/or (3) grouting or pumping concrete into voids along the center of the structure. Installation of sheetpile or other panels, preferably armored with rock, is also employed. Sand-filled geotextile tubes are also employed as interim sand-tightening measures; and, such tubes are also available with integral T-head design (Olsen, 2001b). Again, as noted above, the use of rock *without* an internal barrier or membrane should include a generous additional crest-height allowance to ensure that the structure is sand-tight up to the desired design elevation.

When using rock structures, it is also of fundamental importance to ensure that the design stone *will physically fit* within the structure's profile. Many designers fail to physically and accurately draw the structures, including the specified stone sizes, *to undistorted, true scale.* As a result, an imprudent design may, for example, specify two layers of 1 m armor stone in a structure that is only 1-1/4 m in profile height. In that typical instance, it is often necessary to excavate the grade to found the structures and/or to re-think the design.

To better ensure project stability, it is also highly advised that the structures be placed upon the existing (pre-project) grade rather than upon a post-construction beach fill grade. Where tidal currents are strong in the vicinity of the structures' heads and/or where underlying stratum is of poor quality (e.g., clays), a mattress or other confined foundation structure is recommended beneath the beach-stabilizing structure (Olsen, 2001a, b).

The minimum beach fill volume to be placed during construction is computed as the 3-dimensional difference between the design (predicted) beach contours and the existing contours, adjusted for overfill ratio if necessary to account for grain size compatibility, and also including allowance for impoundment adjacent to the structural field. An additional allowance for compaction must be made if the material is to be placed and measured by truck-haul. In practice, it is prudent to also increase the computed design fill volume by a contingency factor (1.2 minimum), and/or to completely overfill the cells with sand to the seaward ends of the structures. In either case, the beach planform will retreat to its own equilibrium, and the client group must be made aware that the placed beach sand is expected to initially retreat and re-adjust to create an equilibrium shape.

PROJECT PERFORMANCE

Examples of the empirical design approach outlined above are briefly noted below. Except as otherwise noted, tides at these project sites are semi-diurnal and about 0.8 to 1.0 m mean range, with average waves of about 0.5 to 0.8 m height and 5 to 9 seconds period. The ratio of net to gross transport potential is typically 0.3 to 0.45. Median beach grain sizes range between 0.15 and 0.3 mm.

At <u>Fisher Island, Florida</u> (c. April, 1991), seven headland-rock structures were built along 650 m of shoreline immediately downdrift (south) of Miami Beach (**Figure 7**). The beach was then nourished with 20,000 m^3 of aragonite sand imported from the Bahamas. The site had been completely severed from the dominant southerly drift by construction of Government Cut inlet and its jetties. Structures were utilized because of the site's severe erosional stress (>1 m/yr) and the presence of nearshore seagrass beds that were not to be buried by sand fill. This island site more or less represents the terminus of southeast Florida's littoral system, such that downdrift impacts were of minor concern. Four years after project construction, the in-place fill volume was within 1% of the placed volume (Raichle, 1995). The project succesfully weathered Hurricane Andrew (1992) with less than 10% volume loss (subsequently recovered within the next year), and less than 1 to 2 meters of shoreline retreat. While the shoreline orientation "shifts" within the cells as a function of wave direction, the average shoreline location has changed less than 1 m. The mlw shoreline near the middle of each cell is located about 0.34 ($\pm$0.12 s.dev.) times the gap distance between adjacent structures. The orientation of the two southernmost cells structures' heads are "over-corrected" to the local wave angle in order to impose a northerly (reversed) transport direction. The other

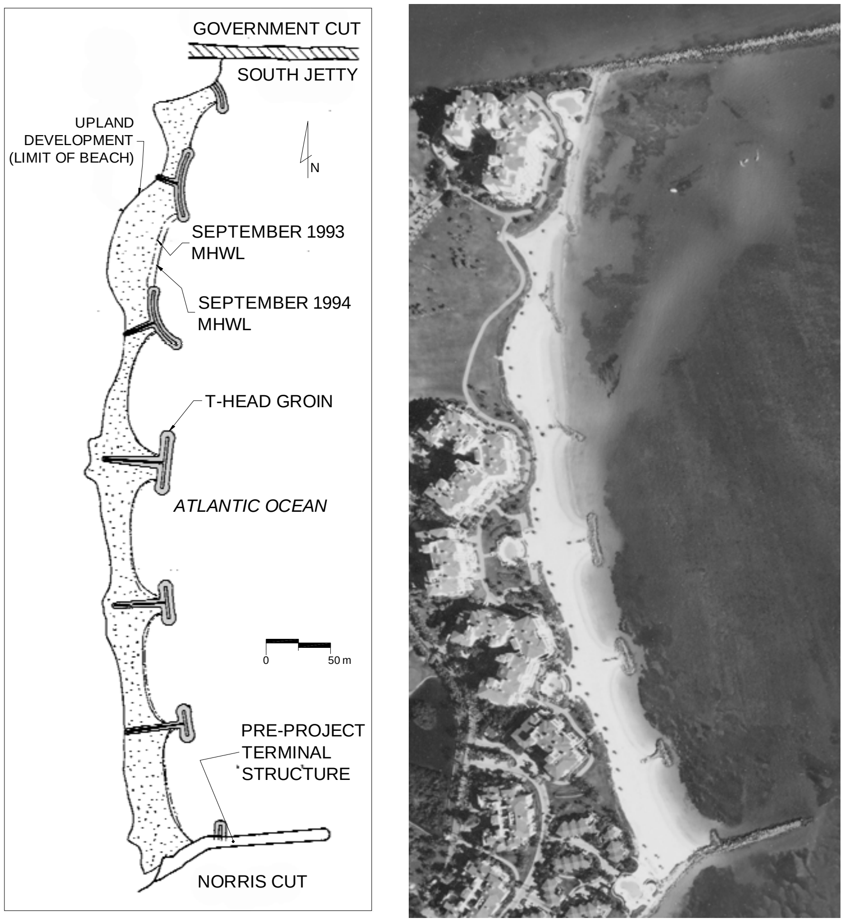

Fig. 7. Fisher Island, Florida.

structures' endpoints are aligned parallel with each cell's average wave angle, where the latter was estimated from grid-based wave refraction analysis using 20 years of hindcast data. No adverse impacts (burial) occurred to the nearshore seagrass beds over 4 years of monitoring (CSA, 1995).

At Jolly Harbor, Antigua (British West Indies), eleven rock T-head structures were built at the northern and southern ends of a 1-km long embayment and subsequently filled with dredged sand in 1994-95 (see **Figure 8**). Both sites had been previously filled with (unstabilized) dredged sand that eroded almost immediately thereafter -- presumably because of the large incident angle of the waves as they diffracted into the embayment. The structurally-stabilized design employed the

Fig. 8. Jolly Harbor, Antigua (north embayment, prior to sand fill).

protocol outlined above, with gap openings of 27 to 44 m and γ = 0.35. The wave angle at each cell was estimated from examination of the nearshore bathymetric contours and visual observation from atop a nearby mountain. The net-to-gross transport ratio at the site is high (probably >0.8). To-date, the post-project shoreline has almost precisely matched the design predictions. Originally, the terminal structure in the field featured a head turned landward (in an attempt to transition to the existing beach) with no groin trunk. The proximity of the head to the ambient beach, and the lack of a stem, led to minor erosion of the existing beach as a crenulate embayment formed in the lee of the head, similar to Fig. 1b.

At <u>Rose Hall, Montego Bay, Jamaica</u>, three rock shore stabilizing structures were built in 1999 to contain a small beach fill (150 meters in length) along a highly energetic rocky shoreline (**Figure 9**). The structures included two terminal L-groins and an intermediate detached breakwater. The intermediate structure provided for an uninterrupted beach between the two terminal structures. The two gap openings are about 36 and 40 m each, with design γ = 0.5. Sand for the final beach grading was derived from the existing littoral material at the site. That material was comprised of a matrix of sand and cobble. The sand was screened from the cobble, and the cobble was buried and capped with the clean beach sand. The shore stabilizing structures were designed according to the protocol outlined above to provide minimal seasonal changes within the groin cells -- as only minor variations in the shoreline would expose the buried cobble material. To-date, the beach has remained stable as per design predictions.

Fig. 9. Montego Bay, Jamaica.

At <u>Tybee Island, Georgia</u>, in 1995, three rock T-head structures were constructed downdrift of an existing rock groin, then filled with dredged sand (**Figure 10**). The site is at the southern end of the island and is adjacent to an unstable tidal channel and swift currents (1 m/s). The mean tide range is 2.1 m. The structures' heads, constructed upon rock-filled geogrid mattresses (Olsen 2001a), are oriented so that the openings between gaps are aligned with the average wave direction (estimated from aerial photo-graphs). From east to west (right to left in photo), the gap openings are 100 m, 85 m, and 85 m; and the equilibrium γ values for each cell are approximately 0.15, 0.38, and 0.57; or, 0.37 on average. The variations be-tween cells are due to the effect of the long updrift groin and high littoral input at the east (right) end, and a leaky terminal groin at the west (left) end.

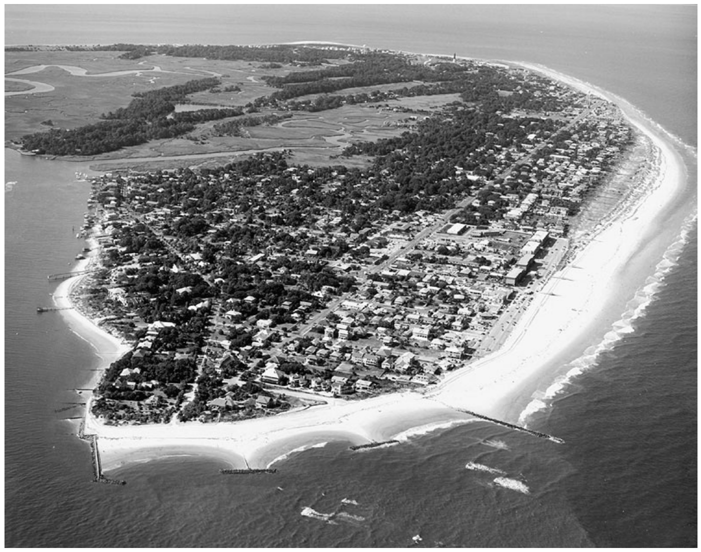

Fig. 10. Tybee Island, Georgia (Sept. 1997).

On the north coast of Lyford Cay (Nassau), New Providence Island, Bahamas, five different stabilized beach fill projects have been constructed since 1993, two of which are shown in **Figures 11** and **12**. Each project, built for private residences, includes about 100 to 140 m of shorefront along a once sandy coastline that has since been denuded of sand. The designs for each project followed the protocol outlined above, where the average and extreme breaking wave angles at each site were estimated from examination of the local pre-project

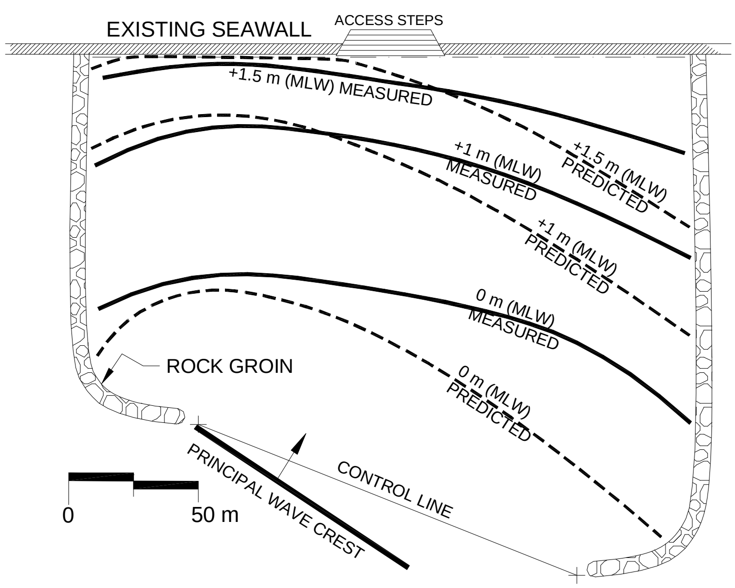

Figure 11: Measured and predicted beach contours; residential beach project; Lyford Cay, Bahamas.

bathymetry and by refraction analysis of assumed offshore wave directions that were reckoned from limited wind data and site observations. Gap openings G for each project vary from about 21 m to 45 m. Measured between 1 and 4 years after construction, the mlw shorelines at each site were located between about γ = 0.35G and 0.55G leeward of the gaps. The measured beach slopes (1:11) at each site are about 10% less (gentler) than the open-coast, natural beaches with identical aragonite grain size (0.43 mm). This is attributed to the lesser wave energy within the cells.

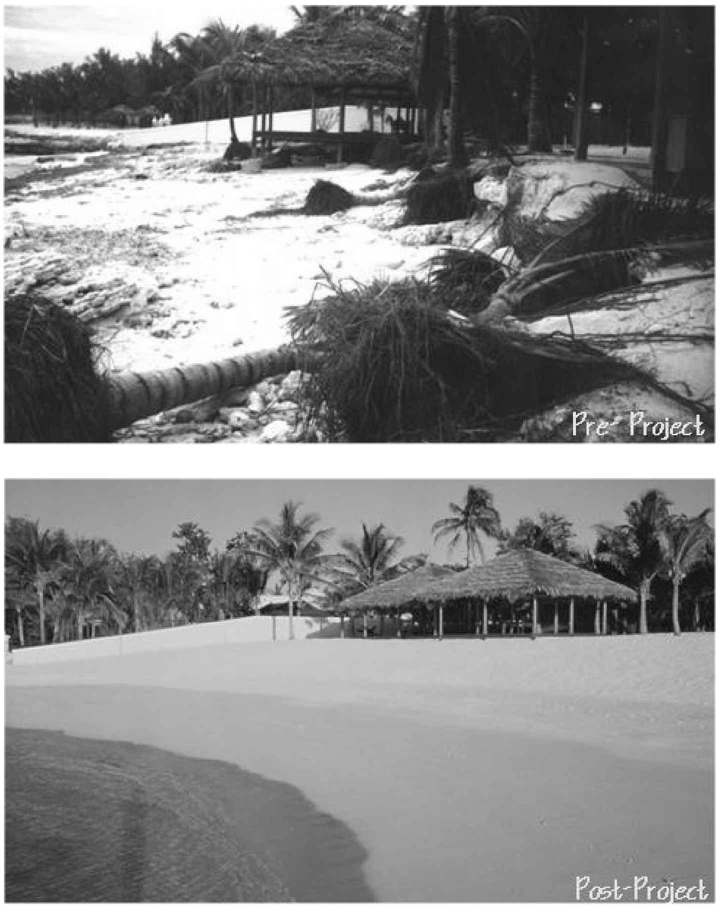

Fig. 12. Residential pocket beach, Lyford Cay, New Providence Is., Bahamas. Upper photo: pre-project. Lower photo: post-project.

Similarly, at a recreational beach constructed using stabilizing T-head structures within a protected lagoon (Atlantis, Paradise Island, Bahamas), the equilibrium beach slope for the same sediment was almost twice as gentle as the open coast beach (1:18 versus 1:10). The lagoon site is a semi-quiescent environment subject only to tidal fluctuation (1 m range) and small long-period waves (0.1 m height and 12 to15 seconds period). Three T-head groins are utilized at this site, with gap openings of 22 m and 28 m, and a post-project mlw shoreline of about γ = 0.3.

At Bonita Beach, Florida, in September 1995, a pair of traditional rock groins was used to stabilize the downdrift end of a 190,000 m^3 beach fill updrift of a small tidal inlet and to promote a bypass bar that would naturally transfer sand to the shoreline downdrift of the inlet. After two years, net fill losses total less than 10% of the placed volume, and the bypass bar has naturally developed (**Figure 13**). Per predictions, the locations of the shorelines updrift of the groins correspond to a profile that intersects the seaward face of the groin at about -0.8 m; or, about equal to the mean tide level minus the mean tide range. Likewise, terminal groins built at the downdrift end of a $2x10^6$ m^3 beach fill at Amelia Island, Florida, and as an interim sand-tightening measure at the south jetty of Port Canaveral, Florida, exhibited similar shoreline response (not shown). These groins consist of sand-filled geotextile tubes, and were placed in 1996 and 1993, respectively.

Fig. 13. Bonita Beach, Florida. Pre-construction, May 1995 (left); post-construction, October 1996 (right).

There are numerous other example projects designed by the author's firm and by others not mentioned herein. Recent descriptions of the latter in the U.S. include Rosati & Pope, 1989; Hanson & Kraus, 1991; Hardaway et al., 1995; Chrzastowski & Trask, 1997; among many others. Other notable examples include many projects and studies in Europe and the Mediterranean (e.g., Spataru, 1990; Peña & Covarsi, 1994; Laustrup & Madsen, 1994; among others). Examination of these projects (precluded herein by space limitations) suggests that their performance is in general agreement with the design protocol and predictions described above.

SUMMARY

The stabilization of beach fill by structures may be warranted at sites where erosion stress is sufficiently severe to require otherwise impractical (frequent) renourishment intervals; or where the proximity of natural resources or marine structures preclude construction of a wide beach fill; or where the project shoreline is advanced far seaward of the adjacent shoreline or located upon a non-littoral coastline. Where the project is not located at the natural terminus of a littoral cell or upon a non-littoral coastline, adverse impacts to downdrift shorelines may be minimized by (i) advance-nourishment of the structures' impoundment field with imported beach fill, (ii) use of T-head or other headland structures that do not promote rip currents and offshore losses, and (iii) termination of the structural field in non-accelerating longshore transport potential.

Beach fill stability is enhanced when the structures' heads are oriented such that the gaps between adjacent structures are approximately aligned with (or beyond) the angle of the local, incident wave crest. The mean low water shoreline within the beach cell between structures can be roughly estimated as lying a fraction γ of the distance of the cell's openings, behind a line drawn between the structures' seaward face, where $0.35 < \gamma < 0.65$. The shoreline updrift of a conventional groin (or that with a modest T-head structure) is approximated as being parallel to the local, average wave crest angle, where the beach profile intersects the seaward face of the groin at an elevation of 1/2 the mean tide range below mlw. Projects constructed since 1991, in 0.8 to 2 m tide ranges, with low- to moderate average wave heights (1 m), have more-or-less performed per the design protocol outlined in the paper.

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