Bodge, K. R., 1998. "Beach Fill Stabilization with Tuned Structures: Experience in the southeastern U.S.A. and Caribbean." In: *Coastlines, Structures and Breakwaters*, N. W. H. Allsop, Ed. Thomas Telford Publishing, 1 Heron Quay, London, c. 1998 (ISBN 0-7177-2668-4); pp. 82-93.

Beach fill stabilization with tuned structures: Experience in the southeastern U.S.A. and Caribbean

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ABSTRACT. 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, and suggests that the structure-induced shoreline can be approximately predicted by a simple "one-third rule". Numerous projects constructed since 1990 in the southeastern U.S. and the Caribbean have thus far performed per the predictions upon which the design protocol is based.

INTRODUCTION

Use of coastal structures to stabilize beach restoration projects has recently re-emerged in the United States and the Caribbean. In the early 1990's, the author's firm was among the first to successfully introduce such structures after a 10 to 20 year informal "ban" of their use in much of the U.S. The present paper describes the design methodology and performance of beach restoration projects that include structures which partly or completely stabilize the beach fill. Such "shoreline stabilization structures" are specifically intended to develop or extend the practical life of the beach fill. They typically include headlands, groins, and/or nearshore breakwaters.

PROJECT CONDITIONS THAT MAY WARRANT STRUCTURES

In the author's design experience to-date, the incorporation of coastal structures to beach fill projects has been warranted in a variety of circumstances. These include instances where

(1) erosion stress is sufficiently severe to preclude an economically- or physically-practical beach fill life;
(2) the proximity of environmentally sensitive natural resources or marine structures preclude construction of a wide beach fill; and/or
(3) the desired beach amenity is advanced far seaward of the adjacent shoreline, or is located upon an otherwise non-littoral coastline.

The first instance includes project sites that feature accelerating (erosive) longshore transport gradients or sediment deprivation, including sites where the littoral drift has been interrupted or where there are alongshore losses to 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 "advance nourishment" fill volume increases the project's width, and therefore increases its perturbative effect to the shoreline. This, in turn, increases the fill's loss rate (primarily via end-effects) so that, the increased fill volume yields little net decrease of the requisite renourishment frequency.

The second instance 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 limit the size of the allowable beach fill; and, as such, restrict the project's life and/or its likelihood of success.

The third instance includes sites that do not naturally (or no longer) feature sandy beaches. This includes coastlines that are naturally rocky or irregular, or where a historically sandy beach has mostly vanished due to sediment starvation, mining, etc. This also includes sites where an "artificial" beach is to be constructed at a new location within a lagoon or along a finite section of a lakefront park, etc. In each of these cases, the desired beach represents a perturbation to the existing coastline that is inherently unstable, primarily due to severe end-losses of the beach fill.

AVOIDING IMPACTS TO ADJACENT SHORELINES

It is well known that "hard" coastal structures (groins, breakwaters, etc.) placed amidst a sedimentary coastline with obliquely incident waves can induce erosion of the adjacent (downdrift) shoreline. This effect can occur via impoundment of the ambient littoral drift against the structure, and via offshore losses induced by rip-currents created by the structure. This adverse effect may be avoided or minimized in the following circumstances.

1. Most simplistically, project sites that are located at the terminus of a littoral cell inherently avoid adverse downdrift impacts (since there is no downdrift beach). Practically, such sites include the ends of an island or the terminus of the strand along an otherwise rocky coastline.
2. Structural fields that are pre-filled with sand (imported from outside of the littoral system) are theoretically "transparent" to the ambient littoral drift. The purpose of this "advance nourishment" is to mitigate the structures' tendency to impound sand by "saturating" the field to (or beyond) its design beach capacity during or immediately after its construction.
3. Structures that include spurs or perpendicular heads at their seaward ends appear less prone to create rip-currents, and are therefore less likely to promote offshore losses. While the author knows of no definitive test results of this specific assertion, 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. Likewise, rough-surfaced (dissipative) structures appear less likely to create rips than smooth-surfaced structures.
4. Structural fields that terminate within a *decelerating* longshore transport gradient are less likely to promote erosion downdrift of the field. In contrast, fields that terminate within a *stable* gradient are marginally resistant to downdrift erosion effects (so long as the structural field remains filled and "transparent" to the ambient littoral drift). Fields that terminate within an *accelerating* transport gradient are more-or-less inherently *prone* to downdrift erosion -- as the last structure(s) in the field preclude beach fill material from being transported into the zone where the erosion stress increases.

DESIGN METHODOLOGY

In general, our design protocol of a structurally-stabilized beach fill involves the following steps:

1. Computation of the average and extreme wave angle orientation -- and the alongshore gradients in net and gross longshore sediment transport potential -- along the project site;
2. Prediction of the nominal shoreline location that will result from the principal and extreme wave angles in response to the width and orientation of the gap between adjacent structures;
3. Translation of the predicted nominal shoreline location to the upper (berm) shoreline location, and identification of the minimum "target" shoreline location to be maintained;

4. Determination of the approximate number and size of the structures and beach "cells" needed to create the target shoreline;

5. Orientation of the structures' ends such that the openings between structures are aligned in desired accordance with the principal, local wave angle; and

6. Lay-out of the structural field and prediction of the resultant beach contours for principal and extreme wave angles; and, iterative re-design to optimize the lay-out as a function of construction quantities, risk of shoreline stability, architectural considerations, etc.

Each of these design elements are specifically described below.

1. Wave Angle & Longshore Transport Potential

Computation of the principal (and extreme) wave angle and the alongshore variation in the longshore transport potential are central to designing the limits and orientation of the structure -- and to predicting the resultant beach fill response within and adjacent to the structural field. Toward this end, we typically employ a grid-based, numerical wave refraction model with alongshore grid spacing not greater than the anticipated distance between structures. Offshore wave conditions are input from hindcast or measured data -- but more often (and more simply) are input in 15° to 25° arc-increments with assumed periods and heights for sea and for swell.

For each input wave condition, and at each alongshore grid column, the wave angle α_b and height H_b at the point of *incipient* breaking are estimated by reference to the shoreward-most *non*-broken wave angle α_1 and height H_1 at each column of the refraction grid. (The latter are identified as the shoreward-most occurrence of a wave height that is less than κd, where κ is the breaker index (usually about 0.8) and d is the local grid depth). Assuming regular depth contours between the shoreward-most non-breaking wave and incipient breaking, and assuming shallow water waves,

$$H_b = A_1 / [1 - (A_1 A_2) / 5] \qquad (1)$$

$$\alpha_b = \sin^{-1} [(\sin \alpha_1)(g H_b / \kappa)^{1/2} / C_1] \qquad (2)$$

where g is gravitational acceleration, C_1 is the wave celerity at the reference wave location, and

$$A_1 = (\kappa/g)^{1/5} H_1^{4/5} (C_{g1} \cos \alpha_1)^{2/5} \qquad (3)$$

$$A_2 = (g \sin^2\alpha_1) / C_1^2 \qquad (4)$$

after Bodge et al. (1996), where C_{g1} is the group celerity at the reference wave location and α_1 is expressed relative to the grid column's local shoreline orientation. The magnitude of the corresponding longshore transport potential is estimated from the "CERC formula" as

$$Q_L = K H_b^{5/2} \sin(2\alpha_b) \qquad (5)$$

where K is taken as an arbitrary coefficient of value 1.0 and units are neglected. (Herein, alongshore *variations* in transport potential are of interest moreso than absolute magnitudes.) The *average* breaking wave angle α_b and longshore transport potential Q_L are computed as the weighted average of each input wave case's angle and transport values at each grid column; i.e.,

$$\alpha_b = \Sigma_i (p_i \alpha_{bi}) \quad \text{and} \quad Q_L = \Sigma_i (p_i Q_L) \qquad (6)$$

where p_i is the average annual occurrence of wave case i. The occurrence data are taken from hindcasts or measurements, if available. Where such data are not available, the occurrence probabilities are simply assumed from local knowledge and/or examination of wind and fetch data. The latter point is particularly important where few data are available and time or budget preclude detailed study. In most cases, the average wave angle and transport gradient that characterize a project site are fairly insensitive to the assumed occurrences of input wave conditions -- relative to the uncertainty and annual variability that typically characterize measured or hindcast wave data. This is especially true where irregular nearshore bathymetry acts as a strong refractive "filter" that results in similar breaking conditions regardless of the offshore wave incidence. Such sites are very often those that warrant structural stabilization in the first place; i.e., sites of irregular coastline morphology or bathymetry. Accordingly, prior to undertaking complicated study of a project site's wave conditions, it can be fruitful to examine the breaking conditions' sensitivity (or lack thereof) to reasonable variations in *assumed* offshore conditions.

Once computed or estimated, the average wave crests and longshore transport gradient are plotted along the project area. The former are used to orient the structures; the latter are used to help identify limits of the structural field. Computed alongshore variations in transport (particularly accelerating gradients) may warrant structures to stabilize the beach fill. The downdrift limit of the structural field is established where the transport potential returns to a quasi-stable level.

2. Shoreline Prediction

Shoreline prediction adjacent to the structures is 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. 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 preliminary design, a simple empirical rule is that *the mean low water shoreline will be located about one-third 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") between adjacent structures or headlands. Here, the mlw shoreline is roughly estimated as a line parallel to the control line, located a distance G/3 behind it. The shoreline in the lee of each headland is estimated as a circular arc of radius G/3 with center at the control line's endpoints. This rough estimate is only valid where the angle between the wave crest and the structures' control line is small (say, less than about 20°). 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 G/3 behind the headland and ends at the downdrift "stable" point. The shoreline in the headland's lee is a circular arc of radius G/3 with center at the headland.

For final design, a composite is made of this simple "one-third rule" shoreline and the predicted shoreline from Silvester & Hsu, 1993 (**Figure 2**). The stability of the beach fill is better ensured when *flanking* of the structures' heads is prevented. In this way, as shown in the figure, the heads' trunk (groin section) is extended at least as far landward as the predicted limit of nominal wave run-up at high tide, and buried within the beach fill. The elevation of the structures' heads is established so as to minimize wave overtopping during the design event of interest.

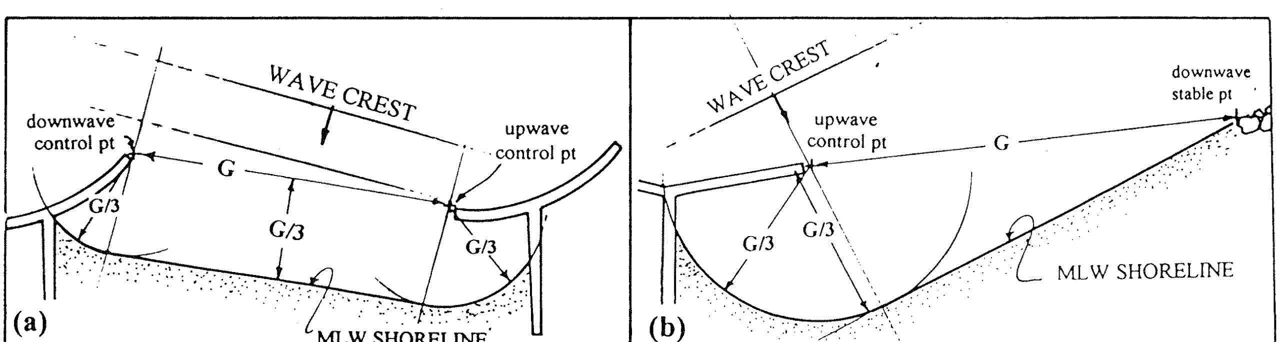

Figure 1. Prediction of the structure-induced m.l.w. shoreline by the "one-third" rule.

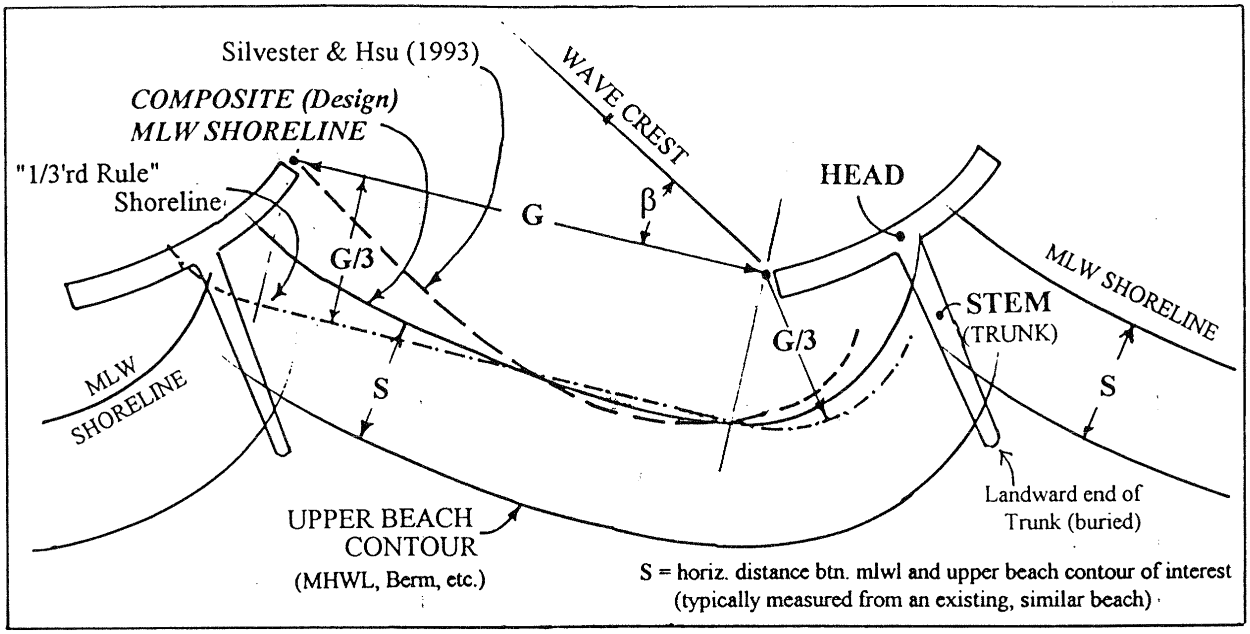

Figure 2. The "design" mlw shoreline is an average of that predicted by the "one-third rule" and Silvester & Hsu (1993) log-spiral geometry. Upper contours are drawn as a uniform offset from the design mlwl.

3. Predicting Berm Location & Requirements

The location of the upper beach contours (i.e., the berm, the limit of storm wave run-up, etc.) is predicted by an up-slope translation of the mlw shoreline described above. This may be most reliably done by measuring the profile slope(s) of an existing, nearby beach with wave exposure and sediment type similar to the project beach fill. The *desired* ("target") location of the project's design berm must also be identified. This is the line beyond which the project's *predicted* berm should fall for incident wave angles of interest.

4. Number and Size of Structures

From **Figure 3**, the total project shorefront length, L, is composed of n beach cells with gap-width G, and n+1 structures of head-width H; i.e., L = n(G+H). 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 "one-third rule", this requires that H = 2(G/3). *Typical* design requires that the high water or other shoreline elevation reaches the head. This requires that H = 2(G/3 +X) where X is the horizontal distance between the mlw shoreline and the contour elevation of interest; i.e.,

$$H = 2G/3 + 2X, \quad \text{where } X \geq 0 \qquad (7)$$

Therefore, the number of requisite beach cells is

$$n = L/(5G/3 + 2X) \qquad (8)$$

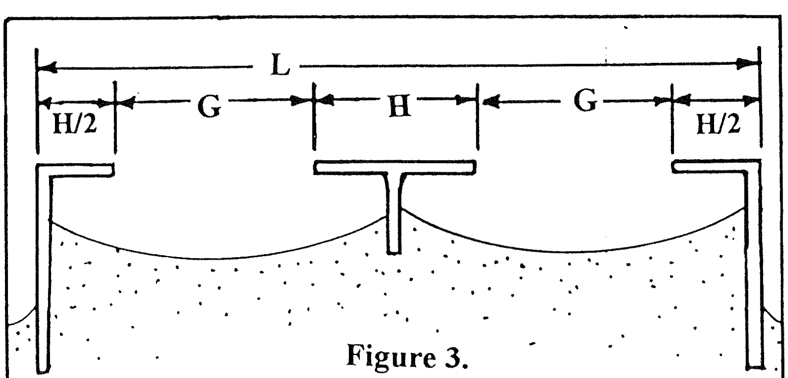

Figure 3.

This represents a family of solutions from which the number of project beach cells, n, can be selected as a function of the desired gap width, G. Of course, to maintain the project's berm at or beyond the "target" location, larger gaps (fewer cells) require larger heads located further from shore. Smaller gaps will re quire smaller structures, though more of them, built closer to shore. The ultimate decision of n, H, and G is therefore influenced by cost (i.e., many small structures vs. fewer large structures), as well as upland architectural or landscape factors.

5. Structure Orientation

The ends of the structures' heads are located so that the openings between them ("control lines") are aligned accordingly with the angle of the local wave crest. The nominal design aligns the heads' ends so that the openings are *parallel* to the principal wave angle at each opening (**Figure 4a**). In this case, the angle between the wave crest and the structures' opening is zero, and the "one-third" rule (described above) is readily employed to predict the mlw shoreline. (In contrast, the Silvester & Hsu (1993) and other predictors are not defined for angles less than about 20°.)

The stability of the beach fill is 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 4b**). 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.

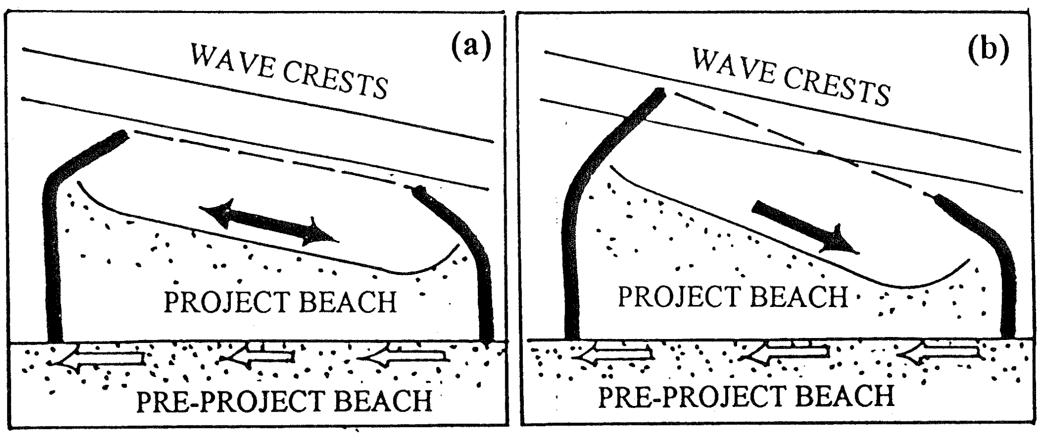

Figure 4. Relative to the expected transport direction on the pre-project beach (white arrows), aligning the opening between the structures (a) parallel with, or (b) against, the wave crests can acts to induce a null or reversal transport direction (black arrows) within the projects' beach cells.

The benefit of "tuning" the structures' head orientation to the local wave angle (relative to traditional, shore-parallel heads) is illustrated in **Figure 5**. For identically-sized structures, those with "tuned" heads yield greater shoreline stability (smaller longshore transport potential within each cell) and greater net beach area than groins with shore-parallel heads.

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.

6. Iterative Design and Final Lay-Out

The lay-out of the structural field is usually an iterative process. For an assumed value of the heads' lengths (or the gap lengths betwixt), the number, size and spacing of the structures can be initially developed from Eqs. 7 & 8. In a first-draft layout, for a given beach cell of gap length G, the endpoint of the upwave structure's head should be located at least (G/3 + Y) seaward of the

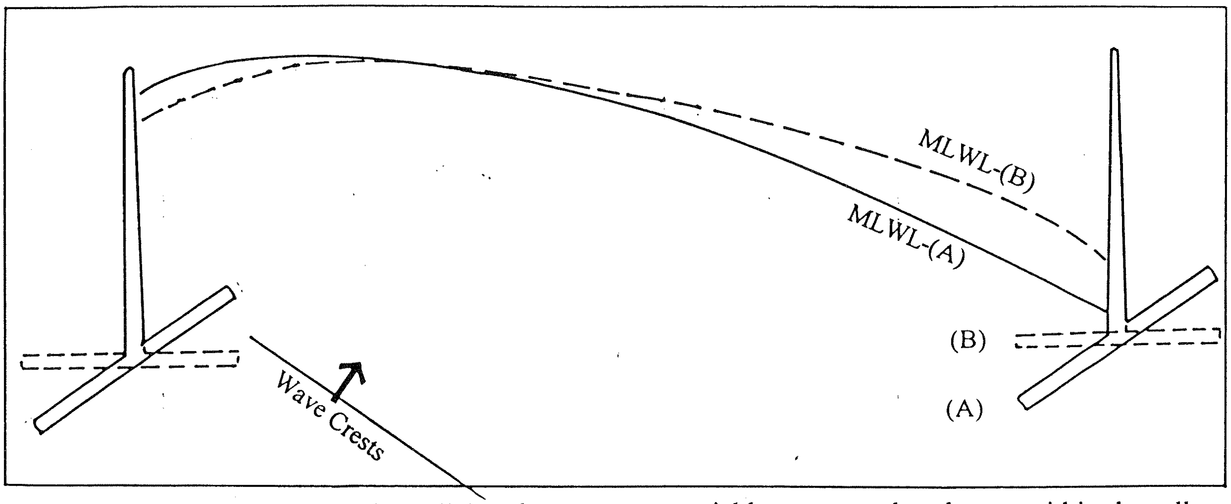

Figure 5. Openings aligned parallel to the wave crests yield greater net beach area within the cell.

"target" shoreline; where Y is the horizontal distance between the mlw contour and the beach contour that corresponds to the "target" shoreline (i.e., berm, etc.). The endpoint of the down-wave structure's head should be located so as to be more-or-less parallel with the cell's principal wave crest. With the structures' lay-out drafted, the mlw and other shorelines are predicted and drawn between the structures. As noted above, the author generally strikes a composite-average of the "one-third rule" shoreline and published "log-sprial" shoreline (Silvester & Hsu (1993), among others), where both are assumed to describe the mlw contour.

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 accomodate 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.

Particularly where the structures are intended to *promote* fill stability in a littoral environment rather than to *provide* it, the structures' heads are reduced to short spurs -- yielding T-head groins 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 end of the groin. Our design work, to-date, assumes that the updrift shoreline contour intersects the seaward face of the groin at mean low water minus half the mean tide range (i.e., one mean tide range below the 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.

PROJECT PERFORMANCE

The empirical design approach highlighted above has been developed and tested through the construction and monitoring of numerous projects. 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 8 seconds period. The ratio of net to gross transport potential is typically

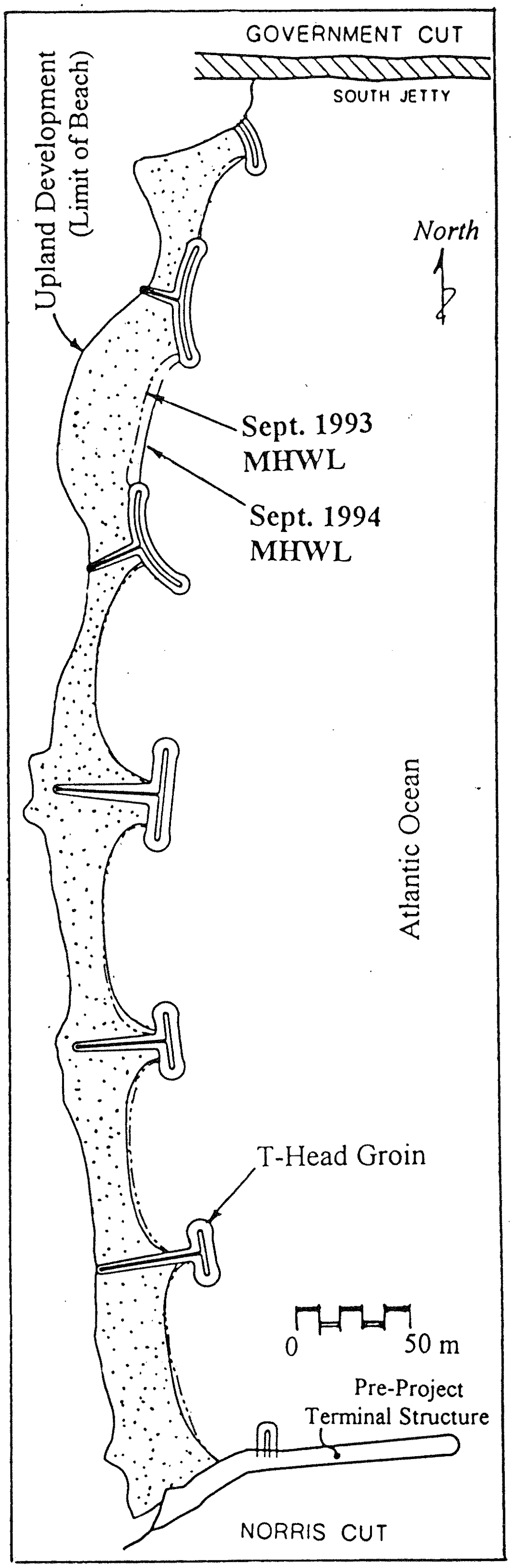

Figure 6. Fisher Island, Florida.

0.3 to 0.45. Median beach grain sizes range between 0.15 and 0.3 mm.

At Fisher Island, Florida (c. April, 1991), seven headland-rock structures were built along 650 m of shoreline immediately downdrift (south) of Miami Beach (**Figure 6**). 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 the construction of the Government Cut inlet and its jetties. Structures were introduced to the project because of the site's severe erosional stress (>1 m/yr) and the presence of near-shore 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 (±0.12 s.dev.) times the gap distance between adjacent structures. The orienta-tion of the two southernmost cells structures' are "over-corrected" to the local wave angle in order to impose a northerly (reversed) transport direction. The other 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 offshore wave data. No adverse impacts (burial) were reported to the nearshore seagrass beds during 4 years of monitoring (CSA, 1995).

At Jolly Harbor, Antigua (B.W.I.), 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 7**). 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 protocol outlined above, except that 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 these two sites is high (probably >0.8). To-date, the post-project shoreline has almost precisely matched the design predictions. Of particular note

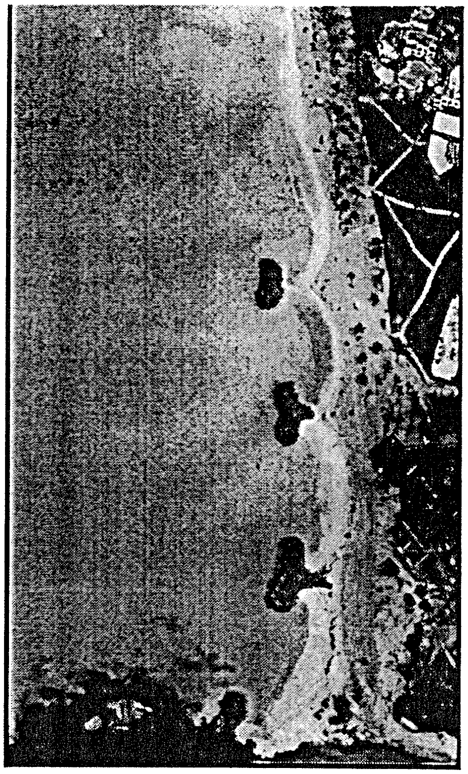

Figure 7. Jolly Harbor, Antigua.

is the terminal structure at the south end, where the head was turned landward (as a transition to the existing beach) and where no groin trunk was constructed. 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.

An analogous problem was noted at the terminus of a rock revetment designed (by others) along the interior of New Pass Inlet, at Lighthouse Point, Florida in May, 1997 (**Figure 8**). A classic crenulate embayment formed at the downdrift end of this structure within several weeks after construction. The consequent erosion of the downdrift beach (presently about 22 m) is approximately 30% of the distance between the end of the revetment and a historically stable point located further downdrift in the lee of a large shoal. The revetment was constructed to armor a shoreline that had eroded downdrift of a previously-constructed revetment. That shoreline likewise featured a crenulate shape with maximum recession of about 78 m, or about 34% of the distance between the original revetment and the previous, historical downdrift stable point.

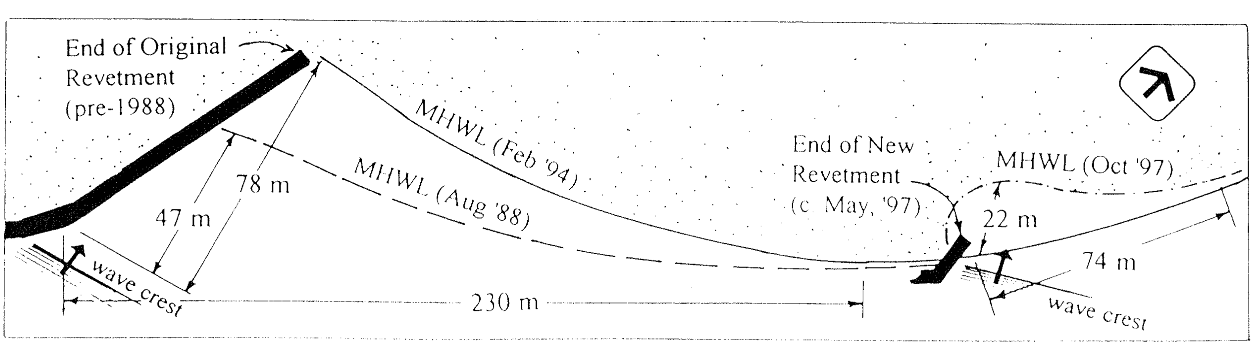

Figure 8. Shoreline recession at the terminus of rock revetments that were built in the 1980's and in 1997; Lighthouse Point (along New Pass Inlet on Florida's west coast).

At Tybee Island, Georgia, in 1995, three rock T-head structures were constructed downdrift of an existing rock groin, then filled with dredged sand (**Figure 9**). 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, are oriented so that the openings between gaps are aligned with the average wave direction (estimated from aerial photographs). In each of the cells, from east to west, the mlw shoreline is located behind the structures' face by distances of approximately 0.15, 0.38, and 0.57 times the cell's opening size; or, 0.37 on average. (The variations between cells are due to the effect of the long updrift groin at the east end, and a leaky groin at the west end.)

Figure 9. T-head groins at the southern end of Tybee Island, Georgia (c. 1995). Photograph: Sept., 1997.

On the north coast of Lyford Cay, New Providence Island, Bahamas, three different stabilized beach fill projects have been constructed since 1993, one of which is shown in **Figure 10**. 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 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 bathymetry and by refraction analysis of assumed offshore wave directions that were reckoned from limited wind data and site observations. The post-project mlw shorelines at each site, measured between 1 and 4 years after construction, were located leeward of the structures' seaward faces by a distances of 0.3 to 0.5 times the structures' gap distances at each beach cell. The measured beach slopes at each site are about 10% less (gentler) than the open-coast, natural beaches that were referenced during design. This is attributed to the lesser wave energy within the bounds of the projects' structures. By analogy, at an artificial lagoon site near these projects (Atlantis, Paradise Island), a semi-stabilized beach fill designed by others reposed to a slope almost twice as gentle as an open coast beach with similar grain size (i.e., 1:18 vs. 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 12-15 s period).

At Bonita Beach, Florida, in 1995, a pair of traditional rock groins were 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 11**). 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 2M 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 (Longard Tubes), and were placed in 1996 and 1993, respectively.

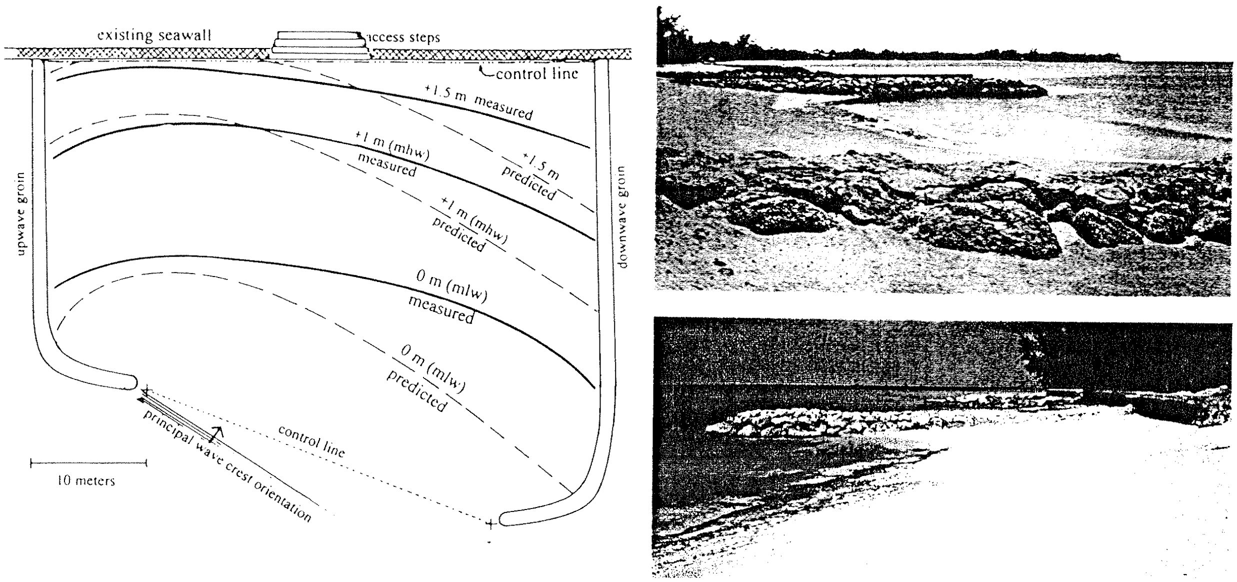

Figure 10. Surveyed beach fill contours approx. 6 months after construction of the R. Arnold stabilized beach restoration project; Lyford Cay, New Providence Is., Bahamas.

Figure 11. Groins at the downdrift end of the Bonita Beach, FL fill project (1995). Photo: August, 1997.

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 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 a zone of 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 1/3'rd of the distance of the cell's openings, behind a line drawn between the structures' seaward face. The shoreline updrift of a conventional groin (or that with a modest T-head) 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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