Dredging and the Environment

Continuing Education Course

Part 3: Beach Nourishment & Wetland Restoration

Course Summary:

This is a multi-part course examines dredging as it relates to various types of environmental projects. If you are not already familiar with the fundamentals of dredging please review Dredging and the Environment Part One, (available on the SunCam web site) we suggest that you consider taking that course before launching into this course. There are a number of important subjects covered in Part 1 that will be implemented in this course, and without an basic understanding of the material covered in Part 1 you may not get the full benefit of this course. Major points that will be covered in this course are:

1. Beach nourishment projects.
2. Wetland habitat restoration projects (which would also apply to mitigation sites, nesting islands and the like).

This course is recommended as an introduction to the individual who is interested in the overall aspects of how Dredging can be used as an environmental restoration tool. The course material will be very practical in nature, it will cover many of the dos and don'ts – as well as what can and cannot be accomplished using today's available technology.
This course is recommended as an introduction for the individual that is interested in the overall aspects of how the Dredging process can be used as an environmental restoration tool. The course material is suggested for the designer, permitting specialist or regulator; it is intended to help broaden the understanding of this technology. It is also intended to be very practical in nature, and focused on how the dredging process can work best in the restoration of waterways. It will also cover many of the dos and don'ts of dredging and project management – as

well as what can and cannot be expected and accomplished using today's available technology.

This document does not cover the regulatory aspect of the process, which would include the permitting and the analytical testing components associated therewith; although it does cover many of the field components thereof, such as practical ways to obtain the most accurate and complete test samples. Rather, it assumes that the reader already has familiarity with the regulatory/ scientific components of the subject and wishes to understand more about the design and physical aspect of dredging process itself.

Use of this course material for design purposes is strictly subject to the limitations and disclaimers set forth which are as follows:
This course is intended only as a study guide of design considerations and is limited to the specific types of projects discussed within this specific course. It is not intended nor is it possible within the confines of such a course to cover all aspects of dredging design or permitting. It is not intended that the materials included herein be used for design of facilities that exceed the size or exposure limitations as demonstrated by the examples. Nor is it intended that an engineer that is inexperienced in maritime design should study this course and immediately undertake design or permitting of a dredging project without some oversight or guidance from someone more experienced in this field. This is especially important for design of projects that could adversely affect the environment. It is important to know that there are an abundance of regulations regarding the undertaking of a dredging project and how it must be conducted such as to minimize its impact on the environment. Failure to properly follow regulatory procedures can result in severe penalties or other liabilities. This course is intended to build the engineer's understanding of maritime design so that he or she can work with other engineers who are more experienced in this area and to allow them to contribute meaningfully to a project. The author has no control or review authority over the subsequent use of this course material, and thus the author accepts no liability for secondary damages that may result from its inappropriate use. In addition this document does not discuss environmental or regulatory permitting, which is a key component of maritime projects – these matters are best taken up with professionals who routinely perform these functions as regulatory issues can dramatically affect design.

Portions of this document refer to the US Army Corps of Engineers Shore Protection Manual and Coastal Engineering Manual; and the US Army Corps of Engineers Engineering Manual on Hydrographic Surveying – EM-1110-2-1003 we wish to formally thank the COE and acknowledge the contributions and research done by the US Army Corps of Engineers as well as the US Army Waterways Experimental Station, & Coastal Engineering Research Center, Vicksburg, Mississippi for there work in producing these manuals. We also wish to thank the Western Dredging Association (WEDA) for there efforts in bringing the subject of Dredging and the Environment to the attention of the world at large, and producers of Hypack Software for their pioneering efforts and contributions to the advancement of Dredging and Hydrographic Software solutions, as well as the following equipment manufacturers and contractors for their contributions: Ellicott Dredge. LLC; Liquid Waste Technology LLC; Cable Arm.com and Mobile Dredging & Pumping Company.

Part 1: Beach Nourishment:

Overview:

Coastal erosion has been a fact of life since the first land masses were formed millennia ago – and for as long as people have been around there are opinions on both sides as to whether alteration of a beach is beneficial to the environment as a whole or not. This course does not take either side of the discussion, rather it will assume that for whatever reason a beach nourishment project has been undertaken, and its purpose has been deemed beneficial enough to allow it to be permitted. Beach nourishment in this course is defined as the dredging of fine to very coarse sand materials from an aquatic source and placing it on an eroded beach. It will cover the design considerations, equipment and methods necessary for undertaking such a project.

Most large beaches that front on the ocean fall under the direction and control of the US Army Corps of Engineers (COE), who have in turn written their own protocols on the restoration of such beaches. As such – these projects are very closely regulated with respect to design, and because the COE normally performs these projects in-house they will not be discussed in detail as part of this course. Rather this document will focus on the smaller beach nourishment projects that would be undertaken by a State, Municipality or private interest.

Most manuals that discuss the economic/ environmental benefit of dunes/ beaches will laundry list the desirable benefits provided by beaches – and usually near the top of those lists is storm protection, followed by a listing of a variety of environmental benefits from aquatic bird habitat to feeding areas for sea life. With that thought in mind, while one can place a general value on the near-shore structures protected by a beach – the question arises as to how one goes about placing a dollar value on the beach and near-shore habitat? The reality is that there is no linear way to evaluate the true value of beaches, and in fact most engineering manuals that discuss the effectiveness of beaches at providing storm protection would place them only in the "moderately effective" category. The reasoning behind this is – that any beaches that are exposed to any degree of wave action/ weather usually require a fairly high level of maintenance compared to other types of "hardened" shorelines. Conversely - most people that live near – or visit beaches (even infrequently) would rate them as an invaluable resource irrespective of any commercial or environmental benefit they might render. Moreover, as a practical matter, when one considers that the bulk of the US population lives within 200 miles of the Ocean or the Gulf coasts – their care and maintenance influences a huge voting block. Thus to put a dollar value on the cost versus benefit on this diminishing resource is, as a practical matter - moot.

With that said, most engineering sources on the subject would rate beach and dune restoration incremental costs at “Moderate to High”, mostly because the average life expectancy of a restoration project can be as little as one and usually no more than 10 years. While dredging costs can fluctuate wildly depending on site exposure and availability of sand and equipment – experience has shown that the cost of a nourishment project can range from as low as $100 to as high as $2000 per lineal foot of shoreline. Thus given any level of expense or need – it seems logical that the designer should consider making sure that whatever is designed and built be durable, at least to the level that available funding and material availability will allow.

Fundamentals of Beach Nourishment:

The intent of this section is to provide the reader with a fundamental understanding of the ways and means to design and specify a reasonably successful beach and/ or dune nourishment project. Space constraints will not allow for any lengthy discussion on the underlying causes of beach and dune erosion – as that subject alone could fill several volumes. However, in brief - the two greatest sources of beach degradation are (1) wave erosion that wears away the sloped intertidal section of the beach and (2) near shore current erosion, which undermines the toe. Many times these two forces act together, which can effectively double the rate of beach erosion.

The most active erosion zone of a beach is the intertidal area which stretches from two or more feet below the Low Water Line - landward to high end of the “run-up zone” which lies about two to four feet above the Extreme High Water Line (depending on wave height and severity). Essentially water and wave action erode the sloped beach surface and tries to flatten the beach grade to something in the realm of almost horizontal. At the same time the eroded sand is pulled off shore and either deposited in a near-shore berm or transported laterally in some direction along the beach's alignment (termed littoral drift). The rate at which this occurs is largely dependent on the size of the waves that routinely occur as well as the velocity of the near-shore currents. In either case – the grain size of the sand and its angular shape are another major factor in the rate of erosion; that is to say – the larger the grain size and the more angular the individual grains the less tendency they have to erode. Thus, it follows that larger grain sized, angular sand is the most ideal beach nourishment material when it is locally available. Ideally, the sand could also have a component of gravelly material or broken shell intermixed with it, which again retards the erosion process – but with that said – availability, logistics and cost will normally be the driving factor in the material selection process.

The next consideration in the design of a beach nourishment project is that of location, mining, transport and placement of the sand. This aspect has become one of the more difficult aspects of permitting a beach replenishment project, as in many areas of the country the "mining" of sand is given a considerable level of scrutiny with respect to the overall environmental picture. In years gone by – the normal source of replacement sand was to declare almost any offshore area as a "borrow" area – and to dredge the sand from that location and place it on the beach, however in recent years there is an increasing concern for the loss of the offshore habitat where the sand "borrowing" might be taking place. The result of this concern has been increased resistance by regulatory agencies and environmental groups in allowing previously "un-dredged areas" to be utilized for borrowing beach nourishment sand. Thus for all practical purposes (exclusive of Federal beach nourishment projects) beach nourishment sand must come from an area of active dredging (such as a navigation channel) or an upland source. With a few rare exceptions, most beach nourishment projects lend themselves to the "Hydraulic Dredging" method as the means of mining and delivery, and "Mechanical Dredging" which usually requires barging or Truck Hauling and direct placement a more difficult and costly second choice.

There are more than a few factors that inhibit mechanical Dredging as a method, the most common of which is the difficulty of finding upland sources of sand that match the color and texture of the natural beach sand. Even when viable sand sources are available (such as in Florida), when the volume of material required for even a small beach building project are reduced to truck measure - the result is an extremely high number of truckloads to transport. Given the effects of excessive trucking on the local routes often required to access beach sites the designer must also factor in the obvious secondary impacts and be prepared to deal with them. The possible exception to the above would be - if an acceptable upland source of sand happened to be located near a bulkhead facility where it could be loaded in bulk onto barges. With bulk delivery by barge available the process would become more workable with respect to the transportation and logistics, however - with that said, there is yet another issue - that of getting the loaded barges close enough to shore so that they could be unloaded. Methods that are typically employed for this methodology require working with the tides so that the dredge and barge can be brought near shore on the high tide, then allowed to ground out as the tide recedes. Once grounding occurs the mechanical dredge could unload the barge – and the sand could be re-handled and spread using traditional upland earth moving equipment. As a practical matter this process requires a tidal range of at least three to four feet, which for practical purposes rules out its use on the southeast coast from Port Canaveral, Florida to its southern tip and anywhere along the Gulf Coast where the tides are generally two feet or less.

Historically, the most common and least expensive method of performing beach nourishment projects is Hydraulic Dredging, which has a number of advantages, but also some limitations. The biggest limitation is that the source of replenishment sand must be within a reasonable distance to the beach being nourished. The practical limit for pumping sandy material (for dredges under 24" pipe diameter) is no more than two miles - depending on the size of the dredge being used. Longer distances have been achieved; however such projects require the use of in-line "Boosters" which can add considerably to the cost. A few larger dredges (up to 36" pipe diameter) are available, and these dredges can pump several miles without difficulty; but their availability can be sporadic and mobilization cost on projects of less than a million yards is prohibitive. In any case, pumping sand through extremely long pipelines brings an added level of risk - in that the inherent density of the sandy soils increases the chances of the pipe becoming "plugged". Such an unfortunate event can cause long shut-downs while the stoppage is located and cleared. The chances of a plugged line occurring become even higher if the sand borrow site contains significant volumes of clam or oyster shell remnants - because their shape tends to disrupt the flow process within the pipeline. Shell deposits can go undetected during the site investigation process, and they can turn up on virtually any borrow site.

Figure 1: Gravelly sand being placed on a beach as part of a nourishment project.

The designer must be cognizant of this potential problem as their negative impact on dredge production can be substantial and in many cases has resulted in extensive litigation between the contractor and the project sponsor. Other than these issues - the advantages of hydraulic dredging and placement remain significant, irrespective of the drawbacks. The biggest advantage is that the hydraulic placement of the sand on the beach acts as a natural “sand wash” which cleans and refines the final product.

Figure 1 is an example of what a beach nourishment project might look like using sand and gravel from a nearby channel dredging project as beach nourishment material. This project will be discussed later in this text, however worth noting is that the material in the photo looks very dark and "gravelly" when first placed on the beach. The dark color is usually attributable to organic content and normally fades within a few weeks – and by the time the project is completed the color is usually indistinguishable from the adjoining beaches. Note also in the photo that there is an obvious high gravel content in this particular outwash – this gravel is very beneficial with respect to retarding erosion - and is usually redistributed when the final beach re-grading takes place.

Practical Design Application:

The best way to describe a small typical non-Federal Beach Nourishment project would be to describe a sample case. The subject project was actually undertaken by a State agency in 1997 for the restoration of a small section of severely eroded beach in Southeast New England (Figure 2).

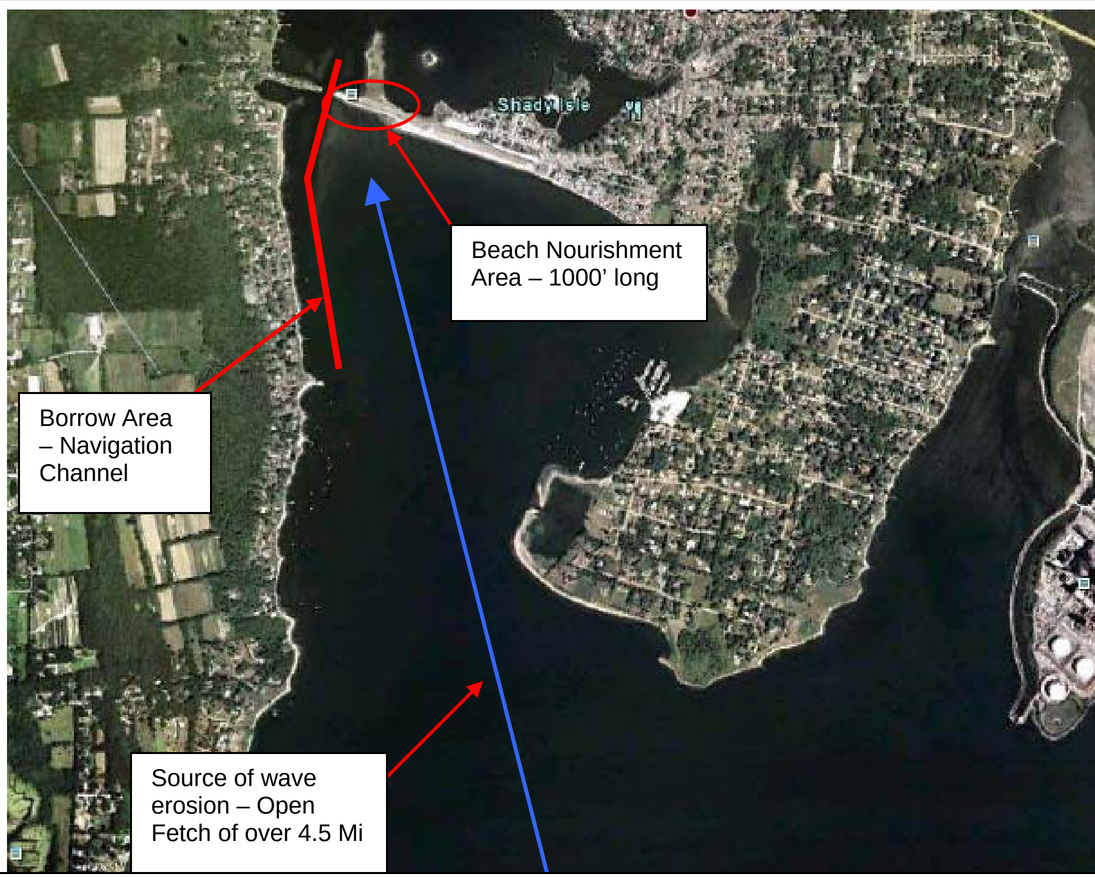

Figure 2: Sample Case Beach Nourishment Site – coastal New England

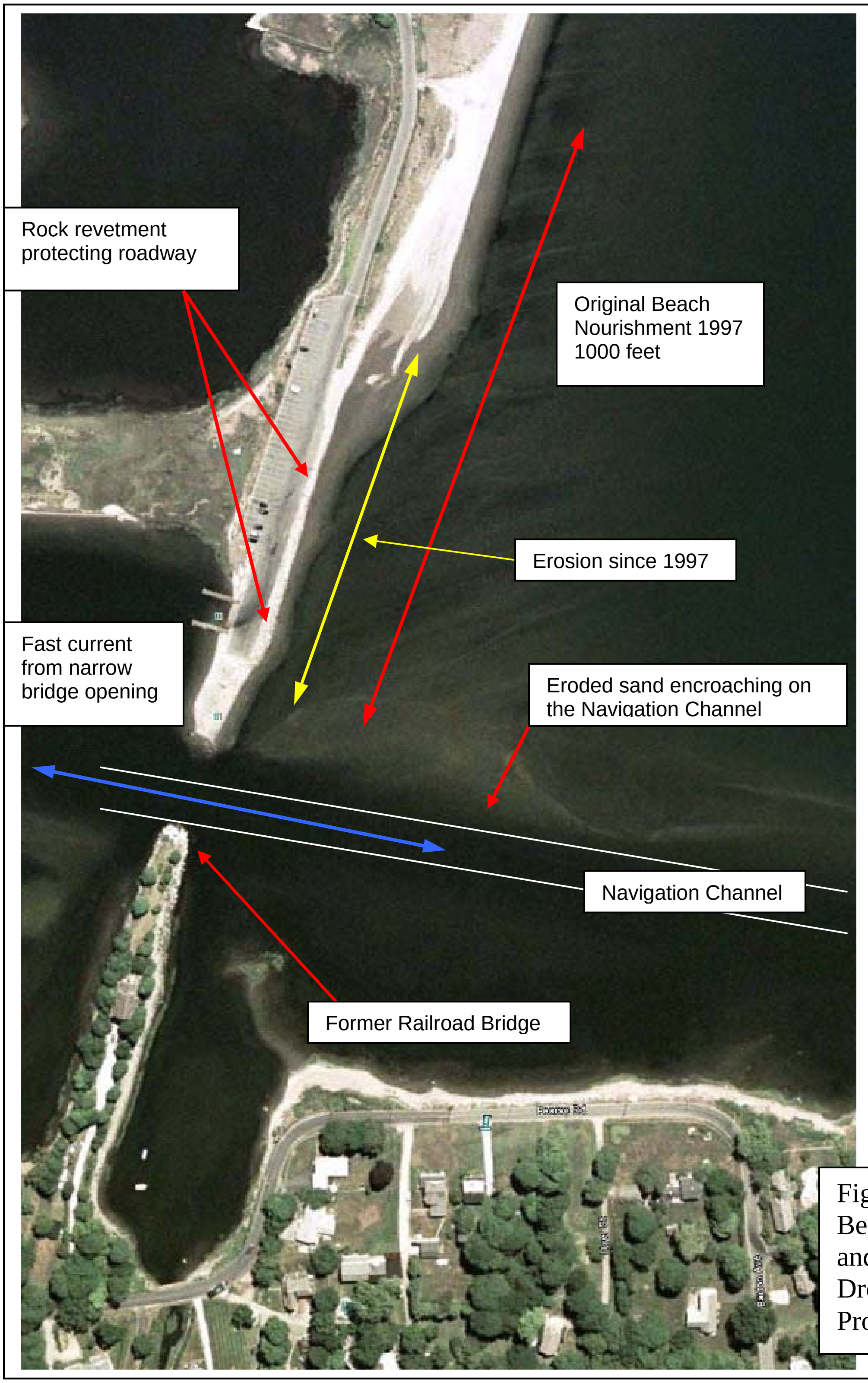

Figure 3: Close-up of Beach Nourishment and Channel Dredging Sample Project

The sample site is an actual case, where the long south-southeasterly wave fetch of a little over 4.5 miles would generate waves approaching three feet in height during high tides (Figure 2). This coupled with the tidal currents (approaching 2.5 knots) in the navigation channel to the west, which contributed to a long-shore littoral drift in the direction of the channel, causing it to fill in with the eroded beach sand.

When the subject project was first undertaken in 1997 the eroded portion of the beach extended for about 1000 feet – and the eroded condition was about the same as the presently eroded section pointed out in the photo. The erosion was then (and is now becoming) so severe, that the only thing preventing the road from washing through is the rock revetment placed along the southern edge (shown by arrows). In 1997 the beach was filled to a width of 150 feet from the rock revetment to the Mean Low Water (MLW) line. This required about 50,000 cubic yards of sand to be dredged from the nearby navigation channel for nourishment material – after consolidation and material losses the net placed volume placed on the beach was in the range of 30,000 cubic yards. The 1997 cost for the project was about $220,000 or about $7.33 per cubic yard, as a point of comparison in 1997 the cost of a gallon of bulk diesel fuel was a little over a dollar.

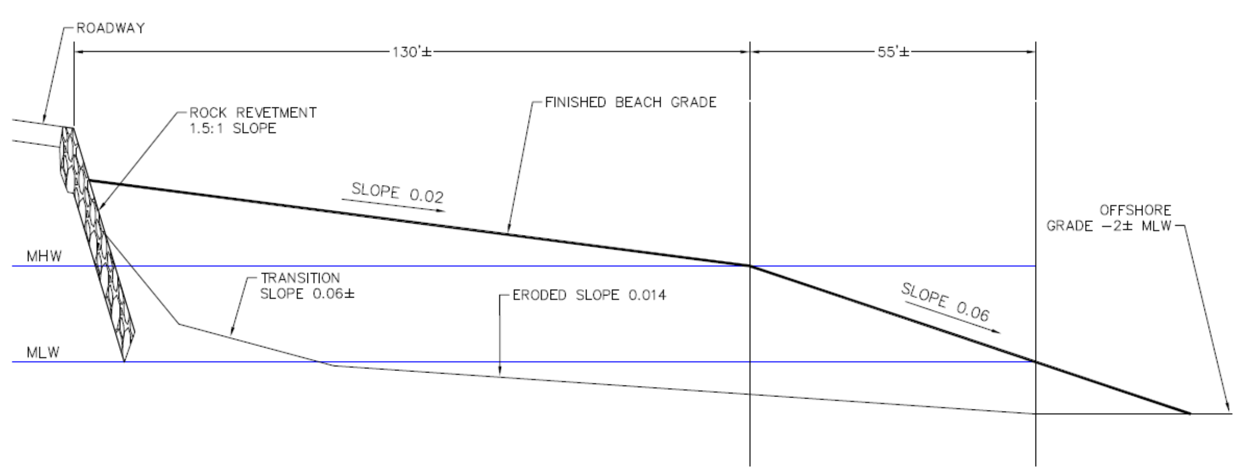

Figure 4: Design beach nourishment template for sample project

Figure 4 is a typical section of the sample project of this lesson. Worth noting is that the existing slope in the eroded section of the beach in 2010 is very little different than it was before the beach nourishment project in 1997, the primary exception being that the present day eroded section of beach is about 650 feet in length – whereas in 1997 the beach erosion extended another 350 feet for a project total of about 1000 feet in length. The photo in Figure 3 shows that the material used as beach fill was coarse sand intermixed with gravel; both the gravel and sand were moderately worn (rounded), but did retain some angular features which improved the erosion resistance.

Note that the present day grade is comprised of two slopes (lower lines of section), the variable inshore slope is about 0.06 (6.0%) for the first 50 to 60 feet from the revetment, whereas the offshore eroded slope is about 0.014 (1.4% - about 4 times flatter). These slopes are the natural end product of the sum of wave action, lack of replenishment source, littoral drift and grain size/ distribution unique to this particular site. The offshore slope of 0.014 is a representation of what a reasonably stable slope would be for this particular site. That is to say, if sand and funding resources as well as space allowed – if the new finished beach were restored to this 1.4% grade, it would last a reasonably long time. Now note the slope immediately to the left of 0.06 slope, this is a transitional slope – it is reasonably stable – but is eroding to eventually match the flatter slope of 0.014 leaving nothing but the reveted slope as erosion protection. The life span with respect to erosion resistance of this transition slope as well as the 0.06 slope is considerably shorter than that of the 0.014 slope. Now referring back to Figure 4, note the heavier lines above the existing grade which represent the grades to which the beach was restored to in 1997. In this case the offshore slope is 0.06 and the inshore slope is close to (albeit somewhat steeper) than the more stable existing offshore slope. Thus it is that the present stable (but eroded) condition is reversed; exposing the steeper, less stable slope to eroding wave action. This is the first design consideration for a durable beach replenishment project: how close to the optimum stable slope of .014 to 0.02 - will the funding, site space availability, and sand material availability allow? This analysis also needs to consider that about 2/3 of the sample project has eroded to pre-nourishment levels in roughly 12 years. Thus for all practical purposes the final design slopes shown should be the minimum considered for any sand nourishment project where coarse sand is available and wave heights are moderate (2.0 feet or less during most of the year). For projects with finer sand and/ or higher wave exposure – flatter beach slopes and wider beach breadth should be factored into the project as a prerequisite to comparable durability.

Locating Material Sources:

Finding a source of quality sand within a reasonable distance of a beach nourishment project can be challenging given the present day permitting climate. The most readily permittable and best potential material sources would be existing navigation projects located within a reasonable distance from the beach nourishment site. Such projects would most likely have either existing permits or at least a permit history. However, with that said – not all existing navigation channel projects contain materials that make for viable beach nourishment, in fact in many areas of the country – most navigation channels inherently do not have sufficient sand quantities to be usable. The unfortunate fact of life is that unless an existing navigation channel is located in the path of near shore sand migration – the most common type of shoal shoal materials will likely be silt (with 10% or less sand), and thus not suitable for beach nourishment.

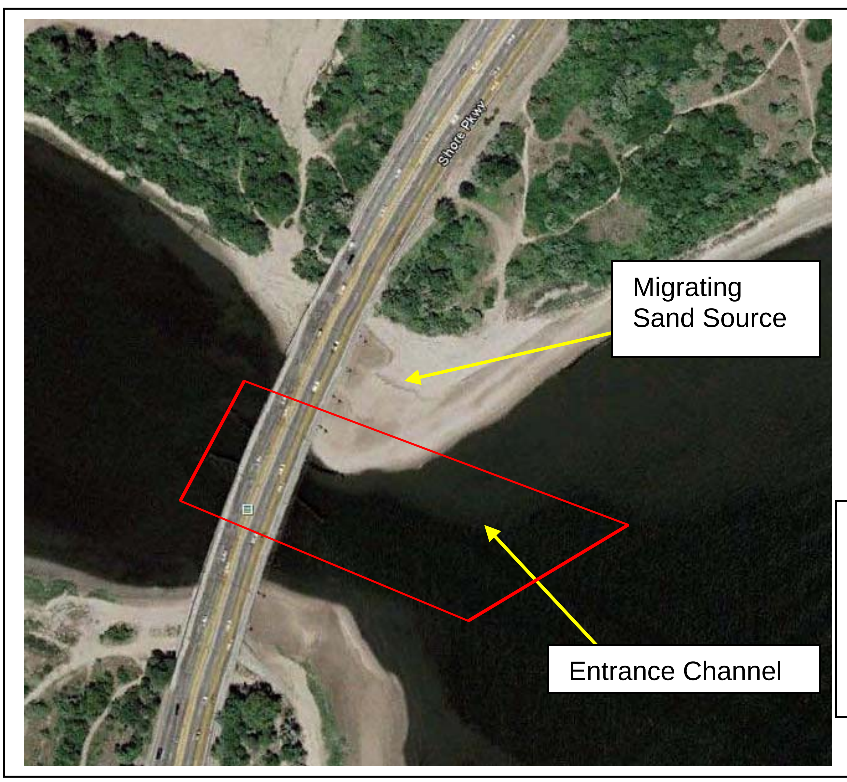

Figure 5: Typical Entrance Channel that crosses a beach zone – note littoral sand build-up in channel

Given this potential – there are a few options that the designer should investigate with respect to finding suitable quantities of sand without taking on the arduous task of permitting a borrow area in an location that has not been previously permitted for dredging. The first - most likely places to locate sand within existing navigation projects is near the point where the navigation channel crosses a beach line (Figure 5).

The Sample project in Figures 2 and 3 are one example – because the natural beach erosion process and the local littoral patterns tend to fill in the approach channel – making it a natural repository. Another example would be a site such as the one in Figure 5, where the littoral process from the adjoining beaches has filled in the channel by migrating it to the south. Local sand sources like this will be compatible in color and texture to the nearby beaches; in addition a channel such as this will likely have been permitted at some point in history. The only problem with sites such as these – is that the available quantities are usually localized (as delineated by the red lines). In some cases limited sand availability can be overcome to some degree - as channels such as this routinely have high shoaling rates and are permitted to be dredged to deeper and wider limits to increase their longevity. In such cases it is sometimes possible to obtain double the amount of material over that which would have come from the originally permitted limits.

<u>Investigations & Design</u>

Before any beach nourishment project begins it would be wise to perform at least a cursory inspection as to what might be the contributing factors causing the erosion. Depending on the scale of the project there are a number of preliminary studies that would be helpful in developing an understanding of the ongoing erosion processes. These might include wave climate analysis, littoral transport and near-shore current analysis, as well as investigation of any new construction that could be affecting sand migration or creating new wave patterns. These are best undertaken by a maritime specialist, preferably someone already familiar with the local area. If the project is small in scale it might include obtaining anecdotal evidence from aerial photographs to ascertain the rate at which the erosion is occurring - so that projections can be made regarding the life expectancy of the project. The importance of these steps is very practical, and is tied to the impermanent nature unique to beach nourishment projects. It is important for the designer to be aware that there have been any number of occasions where newly completed beach nourishment projects have been virtually wiped out in a single season of abnormally stormy weather patterns when a ten year life span was expected. While such anomalies tend to be infrequent, it is best for the designer to have some paper trail of due diligence backing up the design.

Assuming that the project is determined to be viable – the steps involved in preparing for the design process are as follows:

1. Perform a survey of the beach (and if applicable) dune area to at least 200 feet seaward beyond Mean Low Water (MLW).
2. Locate potential sand borrow areas within a reasonable distance. One good potential informational source for this can be local shell fishermen, who tend to know the local waters better than anyone. This is can also be good "PR", as it is a good idea to keep any dredging or filling well clear of shell fishing areas – which they will certainly point out to you during the conversation.
3. Once potential borrow areas have been identified – obtain preliminary hydrographic survey information of the borrow sites – and collect several core samples of the soils at each site.
4. Prepare preliminary designs for the beach/ dune nourishment sites, and determine the approximate volumes of sand needed. Keep in mind that the material shrinkage on beach nourishment projects is higher than other dredging projects (depending on the quality of the borrow area), shrinkage should be estimated at between 1.5 to 1 and 2.0 to 1, that is to say the project will require dredging (in-place measure – in the borrow area) between 1.5 to 2 times the final placement volume measured in-place at the nourishment site. Generally the coarser sand requires about 1.5x while finer sand favors 2x or more.
5. Prepare preliminary estimates of the available volumes of sand from the various sources that were investigated.
6. If preliminary dredging estimates indicate that the potential borrow sites need to be enlarged beyond existing permit limits in order meet the project needs it becomes advisable to meet with regulatory agencies in advance to ascertain the feasibility of such plans.
7. Assuming the preliminary hurdles are crossed, and the project is feasible, the next steps are to target the limits of the feasible borrow areas and

obtain more detailed hydrographic data. This should be followed by more extensive soil sampling to the full depth of the planned dredging.

8. If a planned borrow area is a pre-existing navigation project, it would be wise to ascertain the extent and/or limits of and soft sediment that might overlie the area(s). This is an important step, because if the volumes of pure silt are significant – they will need to be disposed of at some location other than the beach nourishment site (See Dredging and the Environment Part 1 for more information on this subject).
9. The soils planned for use as beach fill should be tested for size fraction gradation to further determine their suitability. At the same time permitting agencies will also require chemical analysis of the materials to determine its suitability for near-shore placement.

<u>Design Considerations:</u>
Sand Placement: Diking of the fill area is not always necessary, and avoidance is desirable if at all feasible. This is because diking tends to create pockets of silt or fine sand within the fill area (rather than spreading it – natural spreading is more desirable). Pockets of fine silt tend to be discolored or become soft which is a less than desirable result. The necessity for diking is largely dependant on the nature of the sand (i.e. fine or coarse; rounded or angular). Generally if the sand is coarse and at least somewhat angular, and the sand source is at least 85% to 90% sand - diking should not be necessary – as most of the desirable sand will stay on the beach and the finer sand will run off and will be taken away by the near shore currents. If the sand is fine in nature, then diking will most likely be required. In these cases it is best to build the dikes out of local sand from the un-eroded sections of the beach as demonstrated in Figure 6.

The compatibility of material sources for the dike construction is very important, because the dikes will ultimately become part of the beach fill as the work progresses. Thus it is the best way to maintain the continuity of the beach when it is completed.

The diking process would involve phasing the project as shown, and thus taking some of the sand from the existing beach and using it to build the dikes for the step one area. Then overfilling the "Step 1" fill area, and then taking that excess sand from the "Step 1" fill area to build dikes for the "Step 2" fill area and then over filling the "Step 2" fill area and so on.

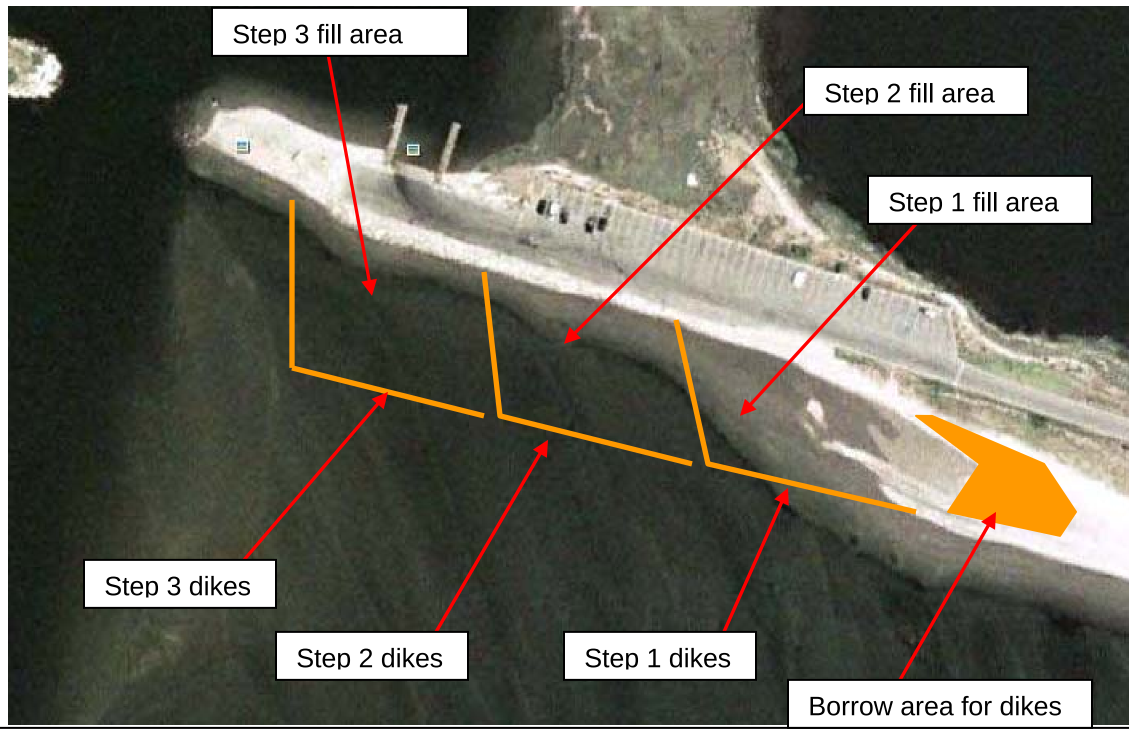

Figure 6: Phased diking of beach fill project – starting with borrow from existing beach to build step one dike, then filling area 1 and using some of the area 1 to build the step 2 dike – and so on

It is always best to work in the direction shown – from the area least in need of fill to the extreme end of the fill area. A typical section of dike would be as shown in Figure 7. Note that the outboard toe of the dike is in the vicinity of offshore limit of fill, this is to limit the distance that the remaining fill will have to be pushed to finish filling out the final fill template. The fill slope on the outboard side of the dike should be built conservatively, as it will be subject to occasional wave action (depending on the site). Generally speaking sand slopes that are continuously submerged will hold about 2:1 (below MLW), in the tidal zone they will generally hold 5:1, but this is subject to wave exposure and should be checked on site before construction.

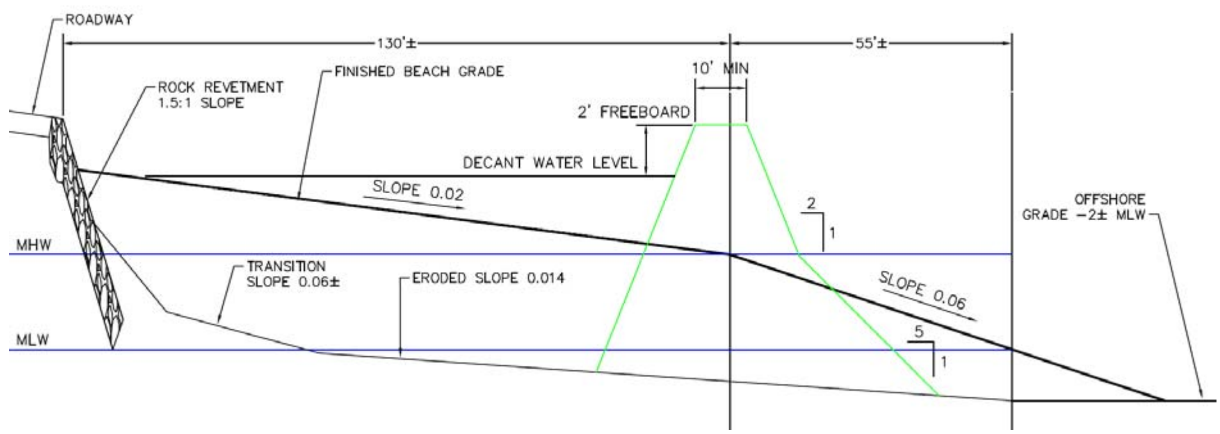

Figure 7: The outboard dike is shown in green, note 2:1 slope on the outboard side above high water line, and 5:1 slope in the intertidal zone. Placement of the dike should be reasonably close to the outside limit of the beach fill area.

Note that in the Figure that the 5:1 slope is shown all the way to the bottom, this is because normally when the fill is first placed under water it will hold a 2:1 slope – but as the intertidal slope erodes – the eroded material will eventually fill out the 5:1 slope below the water as shown. Above the MHW the sand should again hold a 2:1 slope (depending on wave exposure), note that the minimum recommended top width is 10 feet, and that there is a two foot freeboard allowance over the maximum interior water level. These are precautionary measures that are taken to help prevent the dike from “blowing out” during the filling process – which is the term used when the dike fails.

At the conclusion of the beach filling – the dikes are generally regraded to desirable levels with traditional earthmoving equipment and final graded. The buried portions of the dike are usually incorporated into the fill (providing they are built with compatible material – as suggested above). The grading process follows the same procedures as upland earthwork – however it is advisable to use “low ground pressure” tracked equipment in the intertidal zone and further offshore – as freshly placed sand can retain a high liquid softness for a while and the low ground pressure equipment is less likely to become stuck. In addition, particular attention needs to be paid to the area just offshore of the low water line (Figure 8).

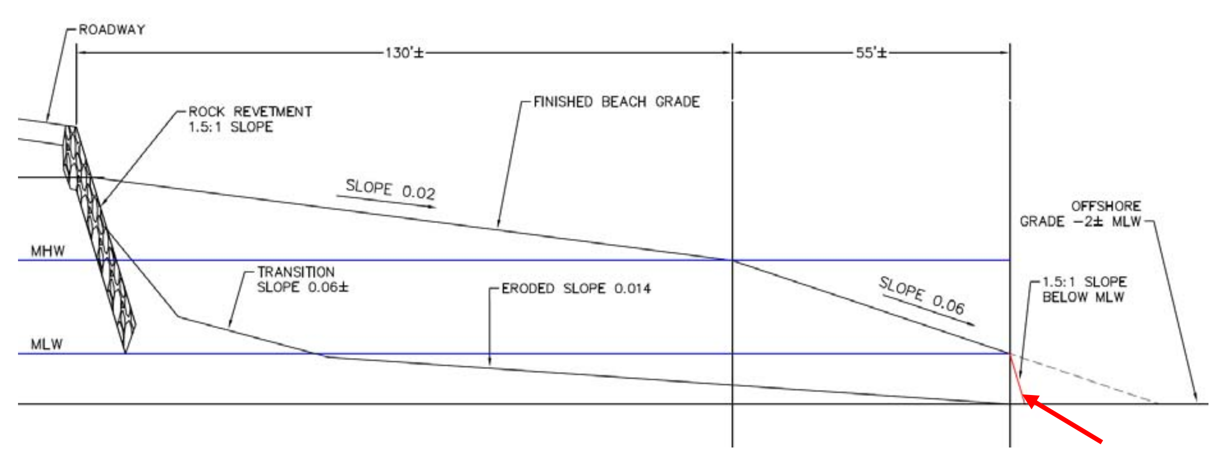

Figure 8: The completed fill template, showing the offshore areas that usually need to be manually graded to meet the uniform offshore grade.

The red line denoted by the red arrow in Figure 8 may seem like an unimportant detail, but it is quite the opposite. As noted in the paragraph on diking, the underwater portion of the fill just past the low water line tends to naturally hold a steeper slope than the intertidal area. In some cases this natural slope can be as steep as 1.5:1, and although it may not sound terribly steep it can became a trap for the unwary bather walking along the shore (as they tend to do). It is very easy for the unwary passer by to slip and fall off of these steep underwater drop-offs, and depending on the depth of the drop off, they could be seriously injured. Most contractors do not like to run their earthmoving equipment in this area – as it accelerates the wear on the tracks and seals. As such - it has historically required additional diligence during inspections to make sure that the slope is properly graded all the way to the interface with the natural bottom. The grade verification process usually requires having a surveyor in waders cover the area with a level rod – and it takes some extra but essential time to make sure that this part of the work is done right.

<u>Dune Construction:</u>

Dune construction or nourishment tends less complicated than beach nourishment, however there are some important aspects with respect to the design. Generally speaking the slopes of dunes should be initially graded between 4:1 and 6:1, depending on the quality of the sand being used. Unlike beaches – they are not normally subject to wave erosion, but they depend on wind erosion to function properly so gravelly fill in this area should be avoided. With the exception of Florida, beach nourishment projects are normally performed during the colder months when the beaches are not in use – however the winter and shoulder months also tend to be windier. Wind, especially the colder westerly winds can erode unprotected sand quite severely in a short time. Beach grass and snow fencing are the most critical (wind) erosion deterrents, so it is important to get these protective work items in place as soon as possible after the grading is completed. Standard practice for dune construction does not require much in the way of grading – in fact the simpler they are the better, Figure 9 shows a simple cross section.

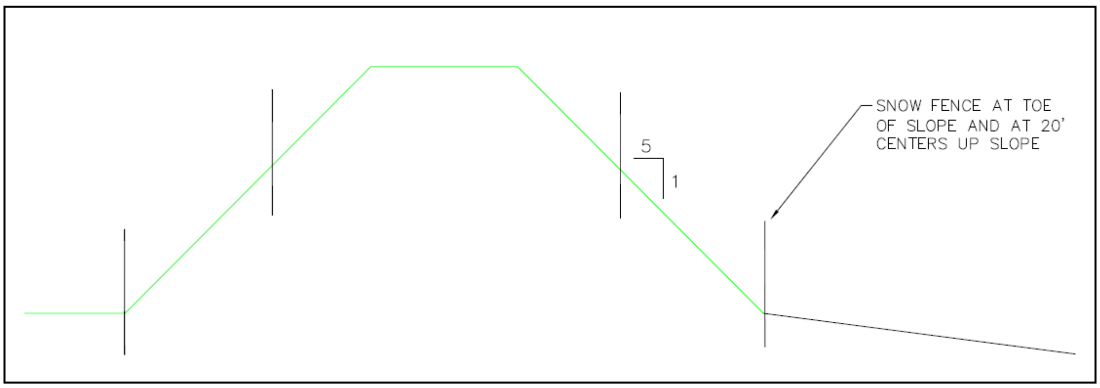

Figure 9: Basic dune cross section, with 5:1 slope shown. Also snow fence at toe and up slope at 20 to 30 foot centers

While opinions may vary, the Corps of Engineers studies indicate that dune lines should be generally straight or gently curved. The figure above shows a 5:1 slope, this is not a bad starting point – however the dune will generally take on its own form – with slopes as steep as 2:1 or as flat as 10:1 depending on the configuration of the fencing and dune plantings. While dunes can be built to slopes steeper than 5:1 once they dry out it becomes difficult for grading equipment to operate on anything much steeper than 4:1.

Providing access through dunes is very important, as once they are planted, random foot traffic can wreak havoc on new plantings. Figure 10 shows a simple cross path, these can be as shown or circuitous depending on the nature of the site. If budgets allow, it's a good idea to install dedicated board walks, as they tend to be more effective at keeping people on the paths.

When it comes to planting, it is best to consult with a professional who is familiar with the local conditions and has experience with dune plantings. While to the untrained eye, all dune grass may look the same, there are at least 20 different types of plantings that can be effectively used to stabilize dunes.

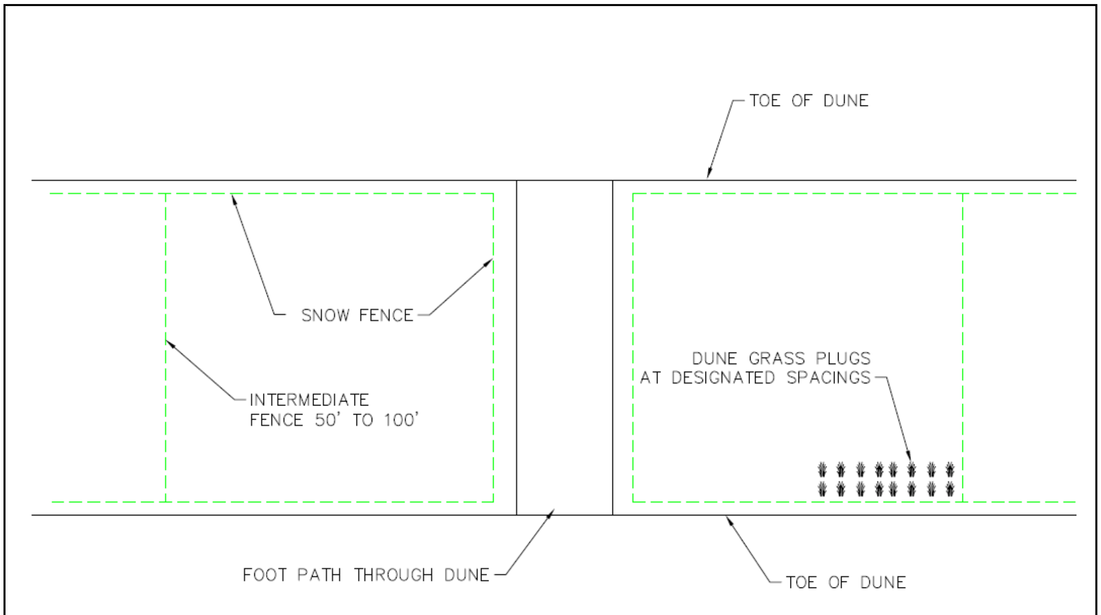

Figure 10: simple dune layout, with cross path, snow fence lines and planting.

The most common varieties are available as plugs and are usually planted in patterns. There are also several types of bush, such as bayberry and rosa rugosa which then to be more hardy, and if planted along the sides of walkways, also help keep traffic off of the planted dune.

Part 2: Habitat Restoration and Mitigation (Wetland & Nearshore):

Overview:

Estimates vary widely as to how much coastal wetland is lost every year, but no matter whose figures one chooses to listen to – the fact remains that despite all best efforts these resources are still on the decline. There are basically two types of wetland loss; the first is coastal erosion where the action of wind and waves cause the exposed “coastal bank” of the estuary to break up and collapse into the water body, and the second is settlement of the land mass through compression of the underlying soils – to a point where inundation takes over and the wetland no longer functions as it once did. This course will not focus as much on the method of loss as it will on how to build or rebuild an already lost or damaged wetland habitat that borders on a body of water or a waterway through the use of dredging. The procedures discussed herein are also applicable to mitigation projects, habitat enhancement projects, and/ or the construction of wildlife islands.

Figure 11: Eroding coastal marsh bank – showing a section of recently collapsed bank (arrow)

Planning:
Any habitat restoration project will necessarily need to review the history of a site and in so doing determine as best possible the cause of the wetland loss, so that the restoration design can potentially be more durable in nature.

One of the most common causes of loss is shoreline erosion – and the marsh habitat areas that front on water bodies are among the most susceptible. These systems depend on the cohesive nature of their underlying soils, combined with a heavy root system to protect their bordering banks from wave and current action. However, the additional pressures of pollution, wakes from passing boats, and development in general weaken these banks and cause them to collapse. The question that arises is – how does one go about developing an engineering based "coastal bank" that will withstand the increased pressure of the new environment, yet maintain the function of the original condition. This is not an easy question to answer, nor are there any "off the shelf" solutions. The US Army Corps of Engineers has done more in this area than any other single entity – and their solutions tend to follow two trends. The first of these methods being the establishment of a new, substantial beach system to absorb wave energy at the seaward edge of the reconstructed coastal bank (where space and bathymetry permit). The second method is the creating of an appropriately "hardened" bank (i.e the use of rock or other erosion resistant material) to act as a shock absorber for wave energy. They have also done a number of studies on the possible use of "soft" solutions and to date have had little documented success to date. As of this writing the only successfully durable version of a "soft" coastal erosion system has been an integration of rock armor, with its voids filled with soil and plantings. With this method – or in fact any "soft" method, the biggest functional problem has been finding a way to protect the newly planted areas until the root systems become established – something that normally could take several years.

Thus knowing the first design challenge, a suggested approach would be as follows: The first undertaking should be an overview of the area to make a rough determination of what outside forces are acting on the site. The second task, which goes hand in hand with the first – would be a gathering of available topographic and bathymetric data over the area of consideration. With the ease of access to online aerial photogrammetry and navigation charting - this task is much easier than has been in the past. As a sample case - Figure 12, below is a section of Jamaica Bay New York, where the loss of intertidal islands (red arrows) has been severe over the past decades. This is a particularly difficult site, and the continual loss of these islands has been the subject of several large studies; yet the entire reason for their degradation remains more speculation than knowing.

Figure 12: Aerial view of Jamaica Bay, New York – showing several eroded intertidal islands

One thing is obvious however, and that is the fact that as these islands have eroded away, the fetch distance (especially to the Northwest) between them has in turn increased dramatically over the past 20 or so years (light blue arrows). This condition is further exacerbated by the presence of a deep water channel (yellow lines) immediately to the Northwest – that is at least 25 feet deep. The straight fetch distance from the far shore is between 1.3 and 2.0 miles, however stronger winds – especially over water don't always blow in a straight line – rather they tend to bend laterally to fit the shape of the waterway as demonstrated by the blue lines in the figure. Thus in this case the routine winter winds that frequent this site from the Northwest might have an actual fetch of over 3 miles – thus easily generating a very steep and damaging chop. This type of wave is particularly damaging to shorelines and if one looks at the historic aerial photos of this area, it is very evident that wave action has been a major component of the wetland degradation. Further compounding the problem are the wakes that are generated from passing boats during the warmer months. Boat traffic emanates from a number of large marinas immediately to the east of the bridge shown in the upper right portion of the photograph.

One of these islands was recently restored using the concept of constructing a sandy beach to the windward extremes of the eroded island – and followed by heavy planting of the intertidal zone. The object of the beach was to provide a gentle slope that would act to absorb wave energy and thus protect the plantings further up the beach. The problem with this concept was - that by their very nature, these wetlands need to become inundated during the high tidal cycle or they will not function properly. Since wave height and energy is a direct function of water depth no matter what the originating source (wind or wake), at the highest tides the newly planted areas were subjected to damaging waves, and thus suffered considerable loss. At present the sponsors of this project are investigating the possibility of installing temporary offshore, floating wave attenuating devices (floating breakwaters – which are discussed at the end of this course), to protect the new shoreline until the plantings become established. This may turn out to be the workable solution; however wave attenuation structures are not without their own set of issues, not the least of which is their propensity for becoming attractive nuisances.

<u>Sample Case Studies:</u>
Since the subject of this document is to focus on the “how-to” of habitat restoration, or replication – only the more proven techniques will be discussed in detail, and the development of soft solutions for shoreline erosion will only be discussed from a limited perspective. This course will cover the determination of the wind/ wave environment, the types of possible design, and the positive and negative aspects of each – all based on the currently accepted coastal engineering practices. For purposes of this instruction we will take the eroded island from Figure 12, and apply one possible method for designing new wetland planting area as well as considering several coastal bank protection schemes for a practical wetland reconstruction.

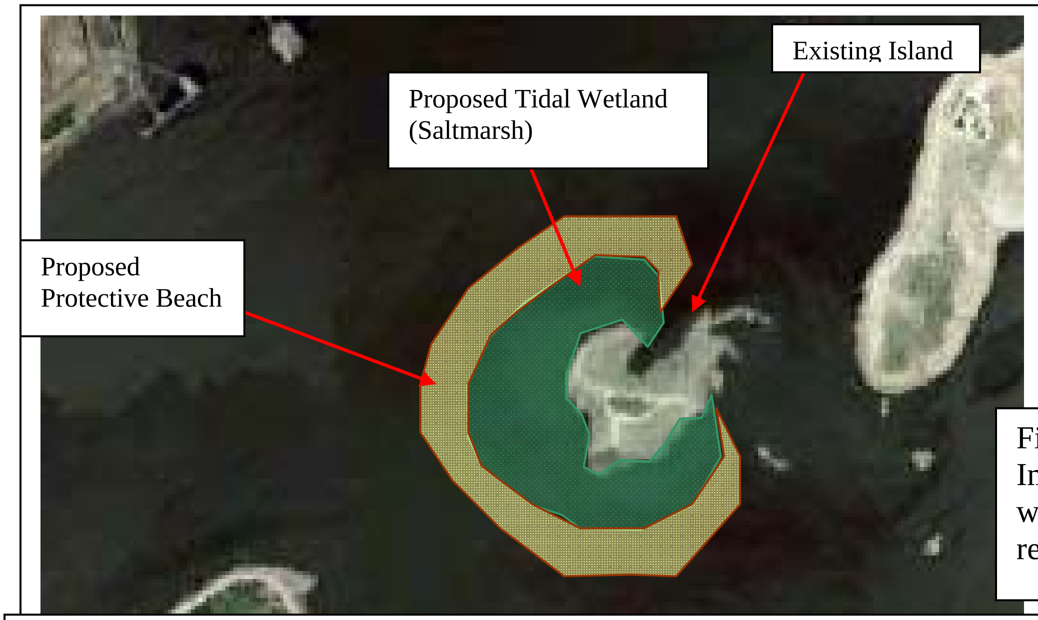

Figure 13: Aerial Image of proposed wetland island restoration

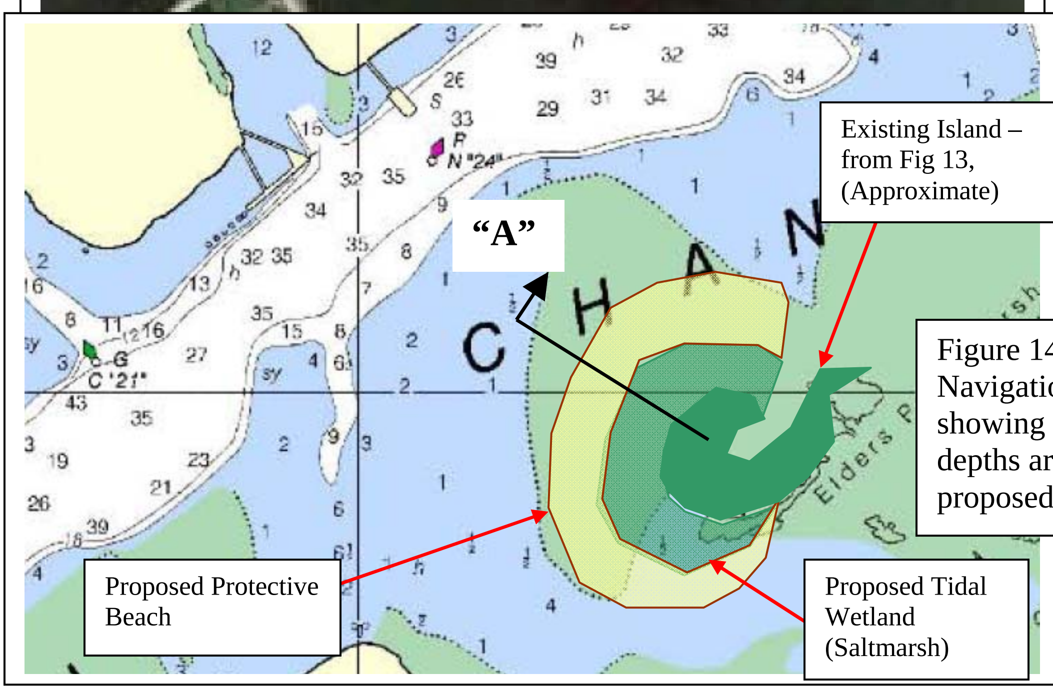

Figure 14: NOAA Navigation Chart showing water depths around the proposed restoration

Let us start by assuming that the overall restoration scheme is shown in Figure 13. Figure 14 is a NOAA Navigational chart with essentially the same proposed layout shown in Figure 13. Note that there are essentially two components of the restoration – an intertidal wetland saltmarsh (planting area), and a beach to help protect the windward limits of the restoration area from wave action. Note also that the project attempts to stay within the shallowest surrounding waters – which for purposes of this dissertation will be assumed to be the approximate historic limits of the original island.

Design Process:

The following is a general outline of the process of developing a restoration plan and the design process. However, before the hard design begins it is essential for the designer to understand the need to get a wetlands and wildlife habitat expert, as well as permitting expert involved early on to set the parameters for the overall restoration plan. Such an individual or individuals should have local first-hand knowledge of the project area, and they will need to be involved throughout the project on issues of habitat selection, plantings, grading schemes and project oversight to best assure the proper functioning of the project once it is completed. This is especially important when it comes to plantings – as most wetland plants require specific soil elevations as well as periods of tidal inundation to thrive. This means that developing a plan that will establish the proper final grade for each type of planting is critical to the success of the project. With that said - translating plans to a successfully constructed project is a bit more difficult than it sounds, as hydraulically placed fill tends to settle over time, so this brings a geotechnical component into the design mix to assess not only how much the newly placed fill will settle, but also how much the underlying soils could compress under the weight of the new fill.

Assuming that the wetlands/ wildlife/ permitting members of the team will furnish the needed design parameters of what the final site should look like – the task then falls on the design engineer to develop a cost effective and sustainable design plan to bring these plans to fruition. The following are the field related components of the design phase that will be needed to accomplish that end:

1. Obtaining current topographic and hydrographic information for the entire footprint of the site.
2. Investigation and location of a suitable fill source for the project.
3. Obtaining geotechnical borings and evaluation of both the fill site and the fill soil borrow area.
4. Obtaining tidal and current information for the area.

These components are all discussed at length in the first part of this course (Beach Nourishment & Restoration) and they do not differ significantly from the needs for this project, so they will not be repeated here. Instead we will begin with a conceptual plan of what the finished project will look like (Figure 15) which is a typical section (denoted as Section "A" in Figure 14) through the existing shoreline – overlaid with the proposed restoration profile. Note that like the beach nourishment project in part one of this course - there are several components to this project. Examining Figure 15; to the extreme left there is an area that represents the remaining saltmarsh at the existing exposed coastal bank. Note that this saltmash is tidal, and the soil level is between 0" and 8" below the Mean High Water (MHW) line (Note: this elevation will vary depending on the project location and planned planting types). Immediately to the right of the existing saltmarsh is the eroded bank, which is usually quite steep at the top – as it is naturally stabilized with the root structure of the plants. Most saltmarshes tend to erode by breaking off in large pieces and slide away from the bank, where they eventually break up and disperse (Figure 11). At the toe of the bank there is usually some form of transition zone that is flatter than the bank yet a somewhat steeper grade than the outwash or adjoining flat.

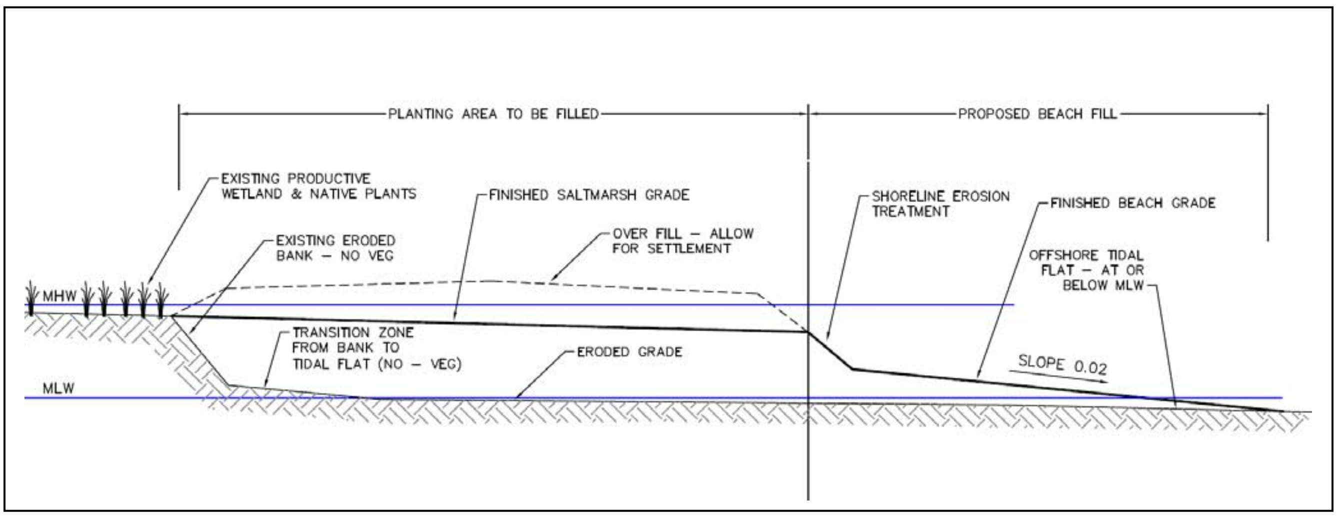

Figure 15: Conceptual profile of existing & proposed wetland restoration fill.
(Please note that the scale of Figures 15 to 17 is warped to a ratio of about 5v:1h

From the toe of the transition zone to the extreme right side of the figure is the tidal flat. Note that this just happens to be the case example that we are using, and that the eroded outwash area can be considerably deeper.

There are also components to the proposed restoration. The features shown are but a few of the many potential features that might be incorporated into a project such as this. Other features might be tidal pools, circulation channels, nesting islands and the like. However from the designer's perspective, these are all just a function of final grading, and cover soil selection which can all be accommodated by the fundamental methods that will be discussed herein.

Note the heavy black line that runs for the top of the existing bank to the right extreme of the exhibit – this is the proposed finished grade. It consists of three basic parts, the upper flatter planting area; a steeper "coastal bank" at some predetermined coastal bank location (that will require some form of stabilization), and the near shore beach – that extends to meet the existing grade at some point offshore. Also note that above the heavy finished grade line, there is a dashed line that represents the initial fill line – which is the grade to which the new fill must be initially placed in order for it to settle to the proper grade. There will be two parts to this settlement, the settlement that occurs as the dredged fill dewaters, then dries and solidifies, and the settlement of the underlying support soils from the additional weight of fill. Fine grained fill (i.e. silts and marine clay) require considerably more expertise to analyze than sandy fill (or underlying soil), as sand tends to be much more stable by its very nature.

Soil Types, Best Uses & Settlement Issues

Depending on the nature of the wetland restoration project that the designer is undertaking there are a variety of soils that are suitable for reclaiming wetland habitats. Generally speaking almost any dredged material that is allowed by the regulatory authorities for aquatic or near shore disposal can be used as long as reasonable screening parameters are observed. The limiting parameter for the soil material is usually the type and level of in-situ contamination found during the exploratory soil sampling process. These limitations are usually spelled out very carefully by the State Water Quality regulations where the proposed project is located. Generally speaking the allowable levels of contamination allowed in such fills are quite low; most commonly falling somewhere above the levels allowed for open water disposal, and below those requiring secure upland disposal. In some cases there are also specific limitations on the allowable contaminant levels in soils used for wetland habitat restoration. The reasons for the limitations lie in the potential for these materials to leach their way back into the waterway either by erosion, groundwater leaching or uptake of the contaminants by plants, where they can be introduced into the wildlife food chain.

Generally speaking there will be two types of soil commonly available for the construction of the restoration project, those being sand – which may or may not be available in sufficient quantities; and fine grained marine silt which is usually abundant. Exploring, classifying and quantifying these soils is covered in Part 1 of this Course, and will not be repeated here – except to say that with a project of this nature there may be more potential sand resources available – which will be discussed later in the course. The sand component would have two uses; one is for fill as the protective beach shown in the sample case – the second would be as a "top-covering" for the higher saltmarsh segment of the proposed fill. On the ideal project, it would be desirable to construct as much of the fill as possible out of sand or soils that were predominantly sand – this is because sand usually has an inherently lower level of contamination, settles less after placement and consolidates much more quickly than marine soils that contain higher percentages of silt. In addition, the sand is a much friendlier soil when it comes to the planting component of the restoration, this is primarily because it can be machine graded and walked on much more quickly and it is much easier for young plants to take root in. Thus if sufficient quantities of sandy soils can be obtained within a reasonable distance from the fill site – there is little need to consider dealing with the settlement issues common with fine grained soils. In the more common situations however, sand is usually not an abundant resource, and such is the reasoning behind the "top-covering" methodology suggested earlier. Generally speaking in an area where sand is a premium commodity the lowest parts of the restoration area can be filled with marine sediment, which is usually available in nearby navigation channels or anchorages - then covered with several feet of sand as an exposed planting surface. Using this method of restoration – the lowest level of fill (using fine grained soils) is placed to some elevation that will allow for at least two to four feet of sandy fill to be placed on top (allowing for settlement). After the lower fill of silty soil has stabilized, the top level of sandy fill is "floated" on top and left at some grade where once it has stabilized and settled it will be at the proper grade for planting.

In cases where sandy soils are in very short supply more innovative approaches may be appropriate, however the designer must be aware that departing too far into the realm of innovation potentially brings with it a new degree of challenge with respect to permitting. Thus before venturing too far down this path, it is always wise to consult with the permitting specialists in your team as well as applicable regulators for feedback.

Author's note: These "innovative" solutions which act as burial "cells" for maintenance dredging projects are admittedly less than ideal means of restoration, however in such cases one needs to weigh the benefit of the restored wetland against the realities of available funding. This is to say, that if funding were not an issue – the best possible solution would be one that disturbs the underlying geology the least, and replicates the original conditions of the wetland as closely as possible. With that said, in most areas where wetlands need restoration or reconstruction – the costs and/or time constraints required can be quite high, and since most restoration projects are publically funded, or are constructed as mitigation for public projects – the unlimited budgets and time-lines that would be required are more often than not unrealistic. As such – the designer and/or the design team normally need to consider alternative design methodologies that are multi-purposed; i.e. Wetland Habitat Restoration and Navigation Channel Dredging as combined projects which can take advantage of multiple funding resources.

Since many areas where wetland restoration projects are needed face the issue of limited sand availability, let us assume for the purposes of this discussion that the sample project is challenged with a shortage of available sand. We will also assume that there is a nearby navigation channel in need of maintenance dredging – and the primary material within the maintenance dredging template is silt, with less than 20% sand. We will also assume that the sediment to be dredged within the channel has low levels of contamination that are "borderline" with respect to open water disposal. At the same time the contaminants are within the allowable limits for placement in "near shore" areas – that is – those abutting waterways. Let us also assume that the soils that underlie both the wetland restoration area as well as the beach are sandy in nature. This is a very common situation found on the Eastern Seaboard, and one that engineers involved in the dredging field face on a recurring basis. If the restoration site in Figures 13 and 14 were faced with the challenges listed above, one potential solution would be something on the order of the conceptual level design shown in Figures 16 & 17.

In Figure 16 we see the proposed wetland restoration project shown in Figures 13 & 14 (and in section view in Figure 15); in this Figure the proposed restoration and notes have been removed, and the existing conditions are shown with a "Phase I" preparation completed. In this case the area where the proposed saltmarsh will be constructed has been excavated several feet deep and a "disposal cell" has been created.

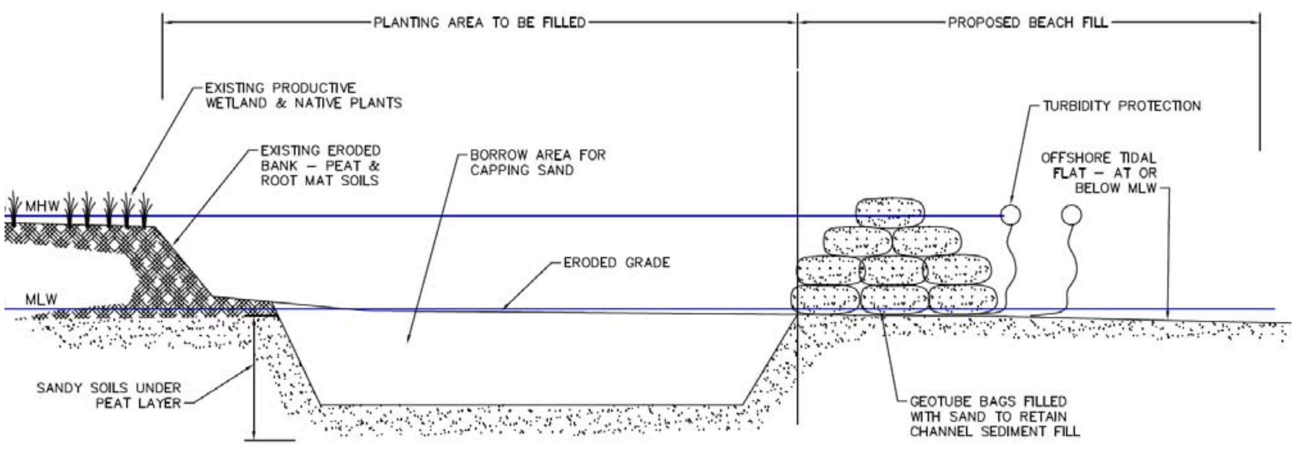

Figure 16: The same proposed wetland habitat shown in Figure 15 with the finished wetland & beach construction removed. The limits of the proposed restored wetland and beach are shown at the top of the figure for reference. The eroded marsh is primarily peat and root mat, but is underlain with sandy soils.

As the Figure shows – the soils underlying the saltmarsh fill area are primarily sand, thus the sand from the excavation of the disposal cell has been removed and stockpiled offsite for this phase; possibly on a nearby beach (preferably the timing would be the "off" season). Since the project will take some time to complete properly, and the sand will be stockpiled for some time – it will be important that the stockpile has appropriate erosion control in place. The next step in this preparatory Phase I would be the placement of an underwater berm to contain the soft sediment from the navigation channel that will be pumped into the cell. The berm will prevent the sediment from migrating away from its intended disposal location. Since the area of this "core" fill is intertidal, it was necessary to build the containment device to a level above the high tide elevation in order to allow the construction of a compartmented spillway – which would be used to control the turbidity of the effluent water created by the filling operation. The final elevation and thickness of the berm will be a function of the local currents and site exposure to wave action; i.e. the more of both or either – the more the need for protection of the fill until it is capped. The best material for construction of this temporary containment are "geotube" bags, which can be placed and secured at low tide, then filled with sand from the dredge excavation of the cell. From a constructability viewpoint, the dredge would have to start digging on the incoming tide – as soon as it had enough water depth for floatation – and dig through the high tide cycle - creating a large enough “starter hole” so that it would not ground out before the tide receded. In the case that it were not able to do so – it would simply wait for the next rising tide and continue. Once the disposal cell excavation was completed and the geotube bags filled, and the compartmentalized spillway constructed - silt containment booms would be placed around the periphery of the fill area to contain any stray turbidity that might otherwise occur from the spillway or the filling. One of the requirements of the Water Quality Certificate would most certainly be that the water leaving the fill area around the perimeter not exceed background turbidity by more than “x” NTUs (“x” will vary by permit and location of project). If the site were extremely sensitive – the permits might require that no water be allowed to overflow the spillway at all – in which case it might be necessary to place a second dredge (or booster) in the fill area while it is being filled – to pump the excess water back to the navigation channel site where the fill material was originating from. (Note: This is not an inexpensive option – and it should be avoided if at all possible)

Once the “core” fill approaches completion the process of   off the fill becomes extremely important. Since the “core” fill is primarily silt, it will retain a considerable volume of water, and it will take time for the silt to dewater itself and consolidate. If this consolidation must occur under the weight of the silt only and the processes of nature, it will normally take a year or more for the silt component of the fill to stabilize and consolidate. Most projects would not be able to accommodate a work requirement that left a site such as this in a state of partial completion for a year, much less the expense of multiple dredge mobilizations and maintenance – and as such measures would have to be taken to accelerate the consolidation process. There are two ways to accelerate the consolidation of the silt fill, one would be the addition of polymers to the dredge effluent while the filling was occurring – but this also could be very expensive. A second option would be to carefully “float” the sand cap component on top of the silt while it is in a semi-consolidated state (See the Suncam Course “Dredging and the Environment - Part 2a “Advanced Remediation Issues” for details on the sand cap placement and the process of “floating” or gentle placement of sand on top of soft sediment). The additional weight of the sand would add sufficient weight to accelerate the consolidation process and the porosity of the sand would allow the excess water to bleed off. The process of this “early” sand capping is not without its risks however – and as such certain precautions must be taken to assure that the capping works properly.

The greatest risk is the somewhat fickle consolidation nature of silt with high moisture content, and this is where the input of an experienced soils expert is critical. As the process of placement of the silt “core” component of the fill is nearing a close – it is advisable to start tracking silt the consolidation process by taking sample cores at several locations around the fill site. The freshly place silt will be very “loose” and potentially fluid, thus the procedures outlined in “Dredging and the Environment - Part 2” for obtaining soil samples in very soft soils should be followed: This is to assure that the analysis is performed on the samples is a true representation of the in-situ condition, and will reflect the state of consolidation at that point in time. From this data the soils expert should be able to generate calculations on how much long term consolidation of the silt core should be expected. The importance of tracking this factor is that it allows for adjustment of the initial core fill levels so that there won’t be any surprises at the end of the project – once the sand cap is in place. It is also important to note that some flexibility must be built into the design with respect to the finished “tolerances” of sand cap thickness and final top of sand cap elevation. This is because the predictability of silt consolidation at this high state of moisture content, as well as the ability to get truly accurate in-situ samples is still more of an art than a science. Also, the amount of settlement will vary from place to place within the fill area depending on where the dredge discharge pipe is placed; i.e. the sand component of the dredge slurry will tend to settle over a relatively small radius around this pipe – and thus these areas will tend to settle less.

The next most critical aspect is how the sand is placed on top of the freshly placed silt core (again refer to “Dredging and the Environment – Part 2” for the general procedures for accomplishing this phase). With that said, there are things to be considered here – the first is that the fill in the example case (which will be typical of most such projects) is inter tidal – that is to say - portions of the core fill, if not all of it will be exposed at the lowest cycles of the tide, and during these times when the sand is above the tide line – its weight will increase by 64 pounds per cubic foot, which when one considers that the underlying silt will be about the consistency of soft pudding – is huge. As such, it is extremely important that at least the first foot of sand cap be placed hydraulically using the “spreader” methods in the above reference, and that the placement layers not exceed six inches per lift. Further, it is best if the sand cap only be placed at the periods of highest tide, and that the spreader be moved continuously by cables rather than boats. The most important thing to keep in mind is that if the spreading operation is lax and allows sand mounds to build up – this will result in the generation of sand pockets and "mud waves" as the tide lowers and the pressure on the soft underlying sediment increases. This in turn will result in areas of the cap that are extremely thin, or exposed areas where there is no cover over the silt, which in turn will make final grading and planting difficult.

All of these factors are important to keep in mind keep in mind in the design process as well as the on-site monitoring of the project. Generally speaking – the more closely the above steps are followed, especially with respect to progressive sampling and analysis as the sediment is placed, the more successful the final project will be. Figure 17 is a simplified section drawing of a completed fill for a wetland restoration (less the shoreline protection and plantings). This is but one of any number of possibilities – that are usually driven by the type of restoration being undertaken, as well as availability of materials.

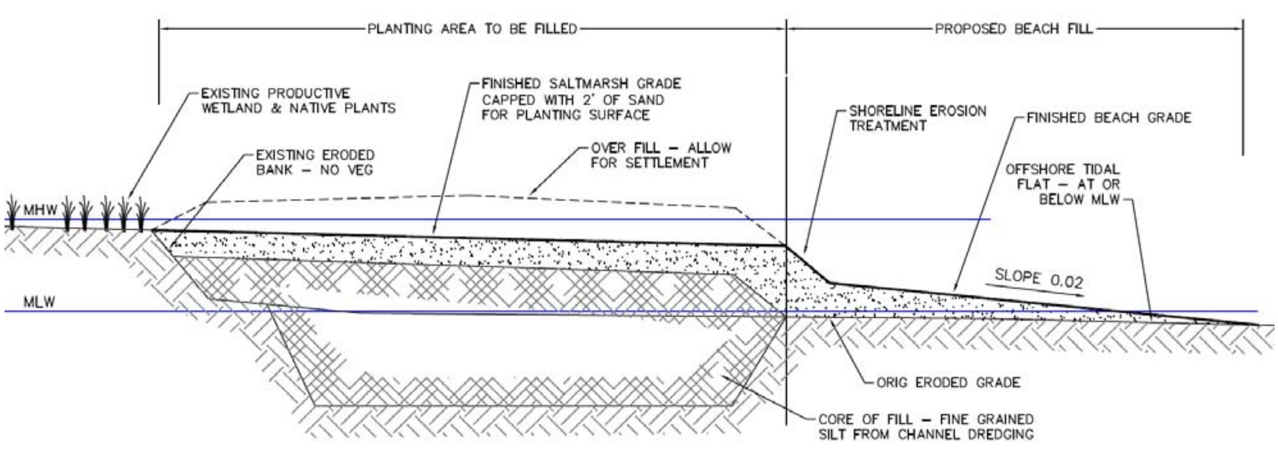

Figure 17: Completed wetland restoration – using "core fill" and sand cap method. Note over fill to allow for core settlement (Shoreline Erosion protection not shown). Approx. 5:1 warped scale.

Shoreline Protection Structures:

Author's note: With all due respect to those who may be taking this course who do not like "hard" shoreline protection measures, or those who must obtain environmental permits in the face of opposition to "hard" shoreline protection systems, the unfortunate fact of life is that in areas were wetland loss is the heaviest "soft" shoreline protection systems just do not have a significant history of long term survival. This is especially true of systems that depend on development of root systems to act as the erosion deterrents. With that said – feedback from readers who have experienced documented success of "soft" solutions is welcomed – and they are encouraged to contact the author regarding such information.

Almost every wetland in need of restoration borders on some body of water that was most likely one of the underlying causes of its degradation. Thus the next critical step in the design process is the development of a shoreline protection scheme that will protect the newly restored wetland either in perpetuity or at least until the root systems of the plantings can become established enough to help with the protection process. The systems that will be considered as part of this course are Rock Revetments, Bulkheads, Sand filled Geotextile Bags and Temporary Offshore Floating Breakwaters; other less durable systems will be presented here in brief, however they are only recommended with the understanding that the client fully understands that such systems typically do not have a history for longevity.

This phase of a project requires some background in wave climate and current analysis, this subject is covered at length in the Suncam course 023, "Marina Site Analysis", the application of which will not be required for completion of this course, but study of that course is recommended at some point for those who wish to understand more about the marine environment and the wave propagation process. In addition to the four primary initial investigations listed at the beginning of this section, the typical additional calculation processes required for analysis of a shoreline protection systems are generally as follows.

1. Wave and current analysis
2. Rock sizing and revetment design
3. Simplified bulkhead design
4. Wave attenuation characteristics and horizontal wave forces for floating breakwaters

Taking the sample case that is being used for this course, and assuming a winter wind and wave condition as represented in Figure 12 – and considering the bending effects of the wind following the shape of the waterway the following wave generation factors will be assumed for the design process:

Winter air temperature = 10^o F = -12.2^o C
Winter water temperature = 30^o F = -1.1^o C
Fetch Distance = 3.0 miles = 4.83 km
Water depth over fetch = 25 feet = 7.62 m
For a winter typical cold front – instantaneous wind speed of 30 mph
Calculated significant wave height = 1.8 feet = 0.55 m
Calculated significant wave period = 2.5 seconds
Calculated significant wave length = 32 feet = 9.76 m

For a winter typical cold front – instantaneous wind speed of 45 mph
Calculated significant wave height = 2.8 feet = 0.85 m
Calculated significant wave period = 3.0 seconds
Calculated significant wave length = 44 feet = 13.4 m

Note: There are a number of ways to generate these figures – and there are many factors affecting propagation of waves, these figures were generated using a more complex analysis using the Shore Protection Manual. For the reader's convenience, a simplified methodology is offered in Suncam Course 023 (Marina Site Analysis); also see Appendix 1 for a brief calculation layout of the methods used to compute the above.

The above represent conditions that are typical for maritime sites in most of the northern states; southern states will have proportionally milder conditions and would generate somewhat smaller waves. This analysis is shown to demonstrate typical conditions for the test case being used in this course, if the reader wishes to apply the principles of this course to other areas of the country – it would require research, available from the National Weather Service's web site regarding seasonal as well as storm condition wind and air/ water temperature conditions. In preparing an analysis such as this, it is best to determine a typical "harsh" wind/ wave condition – this is to set parameters for the more common conditions that the wetland project will have to endure. Then the next step would be to prepare a more typical "severe" weather condition that might occur only once a year, or once every few years – the object being to establish design parameters for both the “routine” condition and the “survival” condition of the shoreline structure. Taking the above data into consideration the designer should then look at the available shoreline protection systems and assess what is the most appropriate for the project. It should be noted here that more often than not there are many other factors in play than pure structural analysis. Thus the best and most durable solution may not meet the project design goals or the policy standards of the regulating agencies – as such numerous design approaches are commonly required during the permitting process and the final design is usually a negotiation between engineers, project proponents, stakeholders, environmental groups and regulators.

<u>Rock Revetments:</u>

The most common and oldest form of shoreline protection is the rock revetment, which normally consists of a layer of rocks or other high mass, durable objects (such as concrete blocks) that are placed or layered in the area of highest wave energy dissipation. Figure 18 is an overall section view of what the sample project finished wetland restoration might look like utilizing a rock revetment as the shore line protection system.

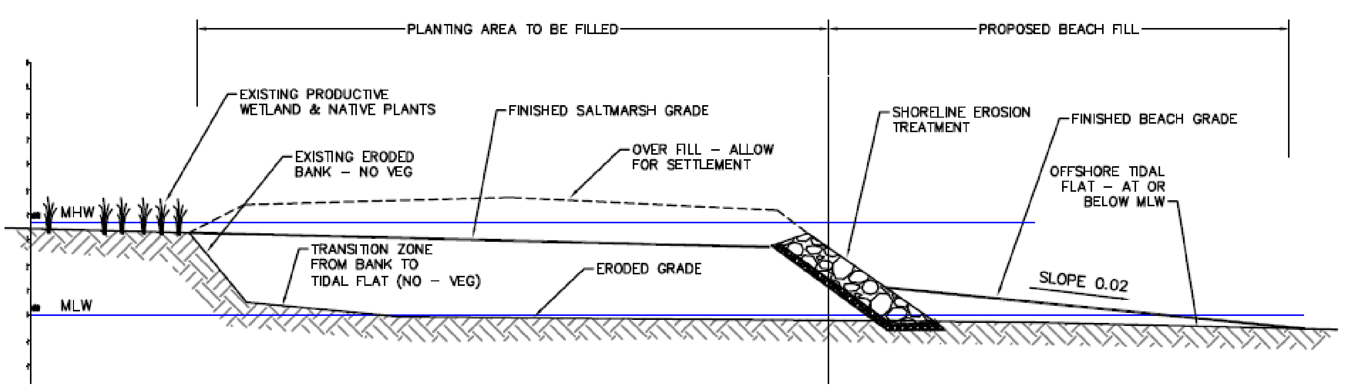

Figure 18: Cross section of the course example wetland restoration project using a rock revetment as a protection system in the “coastal bank” area of highest erosion and wave energy dissipation.

The area of the transition zone form the beach or other “outwash” areas to the higher wetland is typically some form of coastal bank; which is commonly described as an area where the slopes are considerably steeper than either the beach (or other outwash) and the higher wetland or upland areas that it adjoins. These areas are most commonly referred to as “coastal banks” and are naturally formed by the erosion of the higher land mass by waves. In the example shown the area of highest wave energy extends from the low and of the beach at the extreme right of the figure, to the top of the revetment shown on the right center. Technically speaking, at some point in the tidal cycle the wave impact area could focus on any area of the cross section, thus one could say that the potential wave erosion zone extends all the way to the left end of the exhibit because any area covered by water at any tide is capable of allowing waves to generate. However, since wave height is a function of water depth, and the water covering the new wetland would be quite shallow even during the highest tides – the resultant waves would be quite small (usually 0.7 x water depth maximum). Further, once the wetland grasses become established – they would in turn inhibit wave generation.

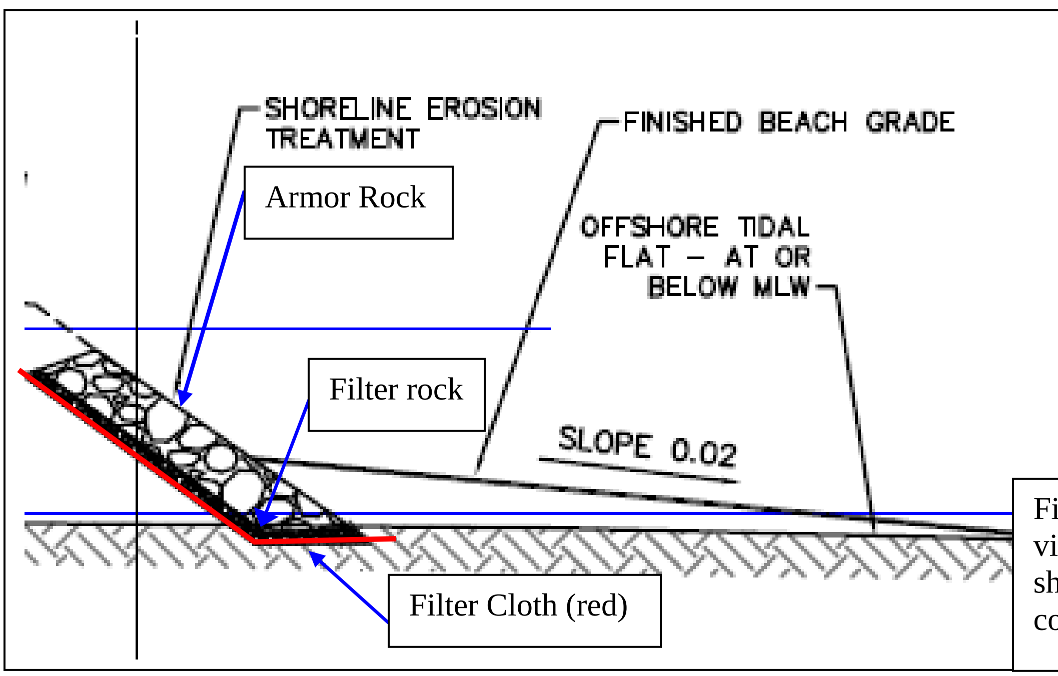

Figure 19: Closer view of revetment showing details of construction.

In the beach nourishment section of this course we discussed how erosion in the beach area was directly affected by the steepness of the beach slope. That is to say, the flatter the beach – the more slowly it eroded – but that the beach erosion process would over time flatten the beach until it approached the grade at its deeper water “toe”. The same is true of coastal banks of almost any description;

however the process is somewhat different in that what normally happens is that as the beach erodes and flattens, the wave energy tends to focus itself on the toe of the coastal bank (where it meets the beach). This in turn usually undermines the toe of the bank until the steepness of the bank causes it to become unstable – whereupon it collapses onto the beach and becomes part of the beach building/ erosion process.

Figure 19 shows a closer view of a typical modern revetment, which has a number of features designed to make it more durable. First note that the “toe” of the revetment is founded well below the proposed beach line (the deeper the better), and also extends some depth (at least a foot – but preferably more, to allow for future erosion) below the pre-existing grade. The area of the interface between the rock of which the revetment is constructed and the underlying natural materials (usually sand) is covered by a geotextile filter fabric. The purpose of the fabric is to prevent the wave energy from “working” the rocks into the underlying sand, thus causing the revetment to settle. The first structural layer of rock is then placed on the filter cloth, and it is recommended that this first layer be comprised of smaller rocks sometimes referred to as a “filter layer”. The purpose of the filter layer is to act as a cushion between the larger “armor” rocks and the underlying filter cloth. Basically the filter layer of rock helps distribute the concentrated weight of the larger armor rocks over a larger area; without the “filter layer” of rock (usually 2” to 6” screened rock, 6” to 12” thick) the wave energy could cause the larger rocks to tear the filter cloth over time, thus greatly reducing the system effectiveness. The last layer of rock is the “armor” layer, which may be one or more layers of rock, depending on the anticipated wave energy. The most common slope for a revetment is one foot of rise to 1.5 feet of horizontal projection. Revetments can be designed with flatter slopes, and each has its own affect on the wave energy dissipation – which generally depends on the wave shape.

Appendix 2 of this document is a simplified process for determining the rock size and number of rock layers needed in a revetment. More information on this subject can be found in the US Army Corps of Engineers “Shore Protection Manual” (SPM), and “Coastal Engineering Manuals” (CEM). Generally speaking the methods used in the SPM utilize wave height as a determining for factor rock size and thickness. One note of caution – the wave heights generated by the formulas in Appendix 1 as well as the CPM are <u>average</u> significant wave heights;

if the subject site is exposed to large longer period waves an additional factor of safety (SF) is recommended. It is suggested that a SF of at least 20% to 50% be added to the wave heights calculated using the SPM methods depending on site exposure. For the case being used in this course 20% is adequate – if the site were exposed to large bodies of water the SF should be closer to 50%. Thus if the severe storm average wave height is calculated to be 2.8 feet, then applying a safety factor of 1.2 would bring the peak wave to about 3.36, which would be rounded off to 3.5 to 4.0 depending on other exposure issues. Using the mathematics from the example in Appendix 2a, and using Case W-1, which assumes that the rocks on the structure trunk are randomly placed angular quarry stones, and the face of the revetment is sloped at 1.5 (H) to 1 (V), the formula produces a required rock size of about 1300# each for a four foot wave. A 1300# rock divided by 150#/ cubic foot would be about an 8.7 cubic foot rock; taking the cube root of 8.7 yields a side dimension of about two feet on each side of the rock were perfectly cubical (which they obviously are not). But this exercise gives an approximation of about what size the rocks would be – using the logic that the average size of these rocks would be about 2.5 feet – and one layer of the rocks would be required, the minimum armor on the slope should be about 2.5 to three feet thick. Taking all things into consideration (exposure, intended use, need to protect upland, etc.), this analysis is sufficient for the intended use of this revetment. Worth noting - if this were a more exposed site, with permanent structures involved a much more rigorous analysis would be advisable.

Also “ground truthing factor” worth noting – if one were to substitute even the lowest wave condition in Appendix 2 (1.8 – 2.0 foot wave height), and no factor of safety factor - the minimum rock size still calculates to be in the range of 160#, which works out to be about a one cubic foot rock (Appendix 2b). Taking this factor into consideration, (that is to say comparing a minimum rock size of 160# to fiber mats or newly planted soil) the designer can easily see from an engineering perspective why less formidable “soft” protection systems would not hold up in the long term in such an environment.

In addition to the features of the structure discussed thus far, the following should be considered as part of the finished design – again depending on the wave exposure. The first consideration would be the addition of smaller rocks to the armor layer – these should be fitted in the voids between the larger rocks. The low end of the weight range for filler rocks on the subject project should be no less that 20% of the mean weight of the larger armor stones (Note that filler stones should not be incorporated in such a way as to be supportive of the primary armor rocks). While these smaller rocks may become displaced during storms – they are an important component in helping to dissipate wave energy, and they also serve to protect the underlying filter rock. The second consideration would be the possible addition of an overtopping apron at the head of the revetment. These aprons are basically the “cap” or “head” stones designated in Appendix 2a, and 2b which tend to be 20% heavier than the slope stones. The design of the apron is rather complex as it involves locating the end point of where the wave breaks over the top of the revetment. For purposes of this simplified design it was omitted because the exposure does not really warrant its expense – however for more exposed sites an apron should definitely be considered. For those readers wishing to learn more about this subject – either the SPM or CEM should be consulted.

There is a functional advantage of the rock revetment that none of the other shore protection options cited in this course exhibit – and that is the characteristic that the rocks have with respect to wave energy dissipation. None of the other systems used in general engineering practice come close to the efficiency of rock revetments – which is probably the reason that they are the most commonly deployed shore protection system. The next closest system would be the concrete mattress systems which are commercially available almost anywhere in the US. Concrete mattresses are basically articulated concrete blocks, tied together with wire rope – or some other linkage system. They come in two or three thicknesses, and come assembled in large mats that look somewhat like bed mattresses (thus the name). Their most common usage is as coverage systems for submerged pipelines, and they are used in many areas of the US where quality rock is not readily available. The major weakness of these systems with respect to shoreline protection is the corrosion of the cables or linkage systems which are usually made of steel. The installation would be similar to the rock revetment, substituting the mattress for the rock layers. Most of the manufacturers have both installation details as well as charts for sizing the concrete blocks for various environmental conditions.

Bulkheads:

Another possibility for a shore protection system would be a wood bulkhead. They would not be the best choice for the Sample Case application as the flat surfaces tend to reflect waves, which in turn would accelerate the erosion of the new beach. Figure 20 is a possible typical installation of such a bulkhead. Bulkhead would however be more suited for wetland or wildlife island construction where the underlying soils are weak and soft; or in almost any case where movement or settlement is a significant issue; in fact in such cases they might be the only workable system.

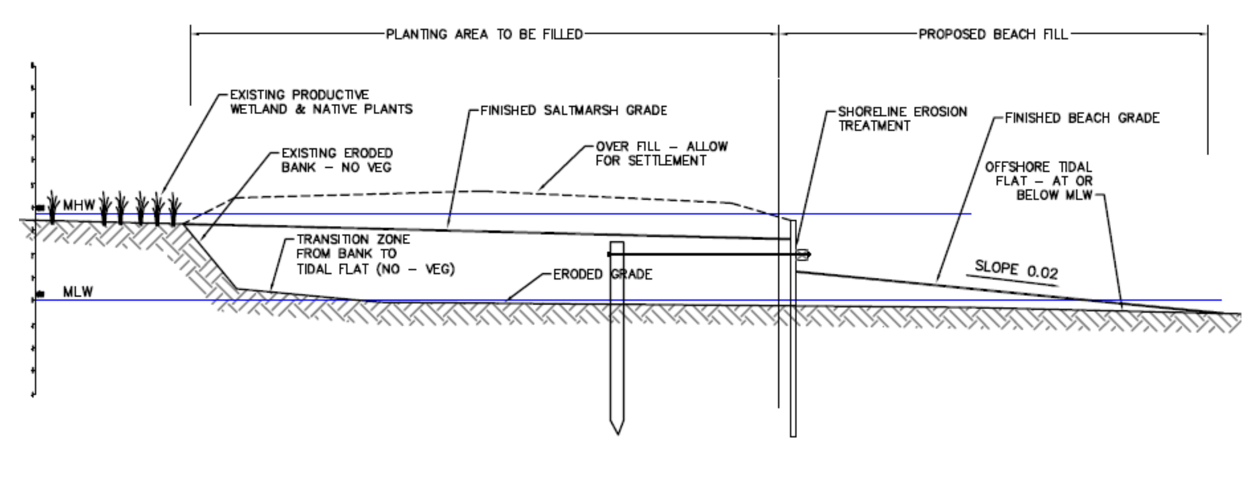

Figure 20: Wetland restoration shown in previous examples utilizing a wood bulkhead as an erosion protection safeguard.

While the design of a bulkhead such as the one depicted is not complicated – the space allowed for this course does now allow a proper explanation of the engineering required for detailed design. There are other good sources for design of simple bulkheads of up to four feet of exposure, such as the “Pile Buck Manual”, however for larger exposures a specialist should be considered. In addition Suncam will also offer a more detailed course on bulkheads and revetments some time in the near future.

Besides offering stability in soft soils - if the shore protection structure is to be “temporary” – that is to say – only to be in place until the wetland root mat could be re-established, then the bulkhead has one other possible advantage. If the bulkhead were to be constructed with untreated wood, it would naturally decompose in the intertidal zone within five to ten years and would thus potentially create a more natural looking coastal bank. Worth noting is that the totally buried portions of the bulkhead would not decompose – thus they would form a limited line of defense for the toe of the bank, which is the most vulnerable area to erosion or overturning (slip out).

Geotubes:

If rock is not available, or if local regulatory policies prohibit its use, sand filled "geotubes" have been substituted for revetments with some degree of success (Figure 21).

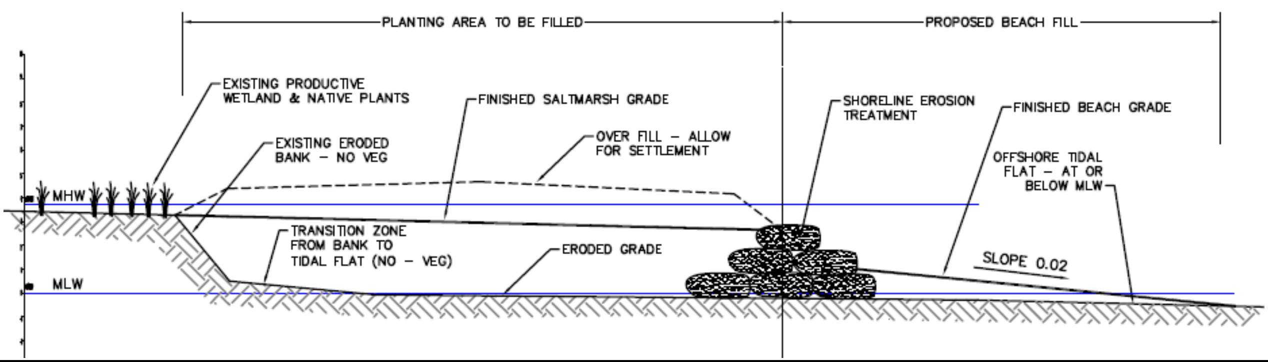

Figure 21: Wetland restoration shown in previous examples utilizing sand filled "geotubes" as an erosion protection safeguard.

"Geotubes" are basically heavy duty geotextile fabric sewn into long tubes that can be virtually any diameter that can be handled. The bags are usually placed on the ground (in this case at low tide), then filled with sandy soil using a hydraulic dredge or other hydraulic pumping device (such as an "air lift"). Many agencies and advocacy groups consider this a more suitable "soft" solution to shoreline protection, however they have several drawbacks. Since the geotextile fabric is essentially a form of "plastic" it tends to decompose over time when exposed to the sun, thus allowing the bags to eventually weaken. When this occurs they are prone to breakage or tearing, thus releasing the sand filler, and rendering them more and more ineffective for protection as time goes on. When the bags tear apart, the strands tend to fragment – and they eventually enter the water column, where they decompose at a much slower rate. Thus these loose fabric strands or patches of cloth become a potential nuisance substance that could be ingested by water fowl or marine life. The other disadvantage is that the smooth surface that they present does not absorb wave energy nearly as effectively as random placed rock – thus overtopping and increased erosion from wave reflection become problematic in some applications.

Offshore Breakwaters:

Another potential aide in shoreline protection is the floating offshore breakwater. These systems have been in sporadic use for wetland protection for several years – and they appear to be moderately effective. The principal behind them is to deploy them a few hundred feet offshore of newly rebuilt coastal features so that they will reduce wave height during higher wind events, and thus will control erosion of newly planted wetlands projects or "soft" shoreline protection systems.

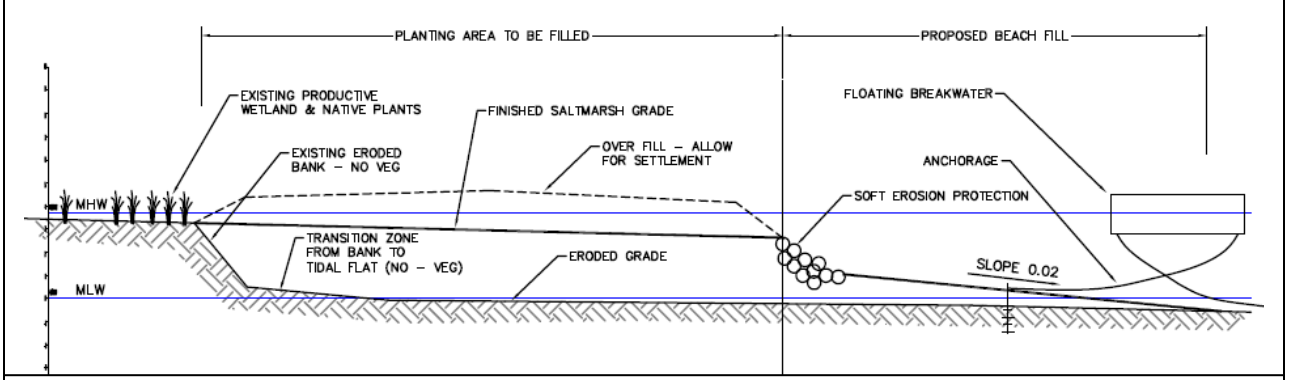

Figure 21: Wetland restoration shown in previous examples utilizing an offshore floating breakwater as an erosion protection safeguard. Also note the used of a "soft" shoreline protection system at the new "coastal bank".

It should be noted that (depending on exposure) in addition to the breakwater some form of coastal bank protection is still required. There are several reasons for this – first, no floating breakwater is 100% efficient and depending on the design and wave exposure may only reduce the wave severity by a percentage. The range can be as little as 10% reduction to as much as 70%, and the percentage is highly dependant on the tested efficiency of the breakwater design as well as the period of the incoming wave. Very few such systems have significant effect on waves exceeding when the wave period exceeds 3.0 seconds. In designing a project that depends on a floating offshore breakwater for protection - it is important to verify the wave attenuation characteristics with the manufacturer. In addition it is equally important that the breakwater manufacturer have actual field tested and documented wave attenuation performance charts; it is very important that the designer not rely on “brochure data” that is not backed up by documented, actual full scale testing.

Once the breakwater type and size has been determined the next step is to determine the height of the residual attenuated wave, which should be obtained from the manufacturer’s test data (Note that wave attenuation does not change the wave period). To this the designer must also consider the wave height that will rebuild in the space between the floating breakwater and the shoreline, as this can be a considerable factor that is often overlooked. Using the fetch distance between the new breakwater and the new coastal bank and water depth at the highest tide – the designer can obtain wave height and period data using either the Suncam course 023, “Marina Site Analysis”, or the SPM. While the calculated wave period will most likely not be significant for the short fetch, the wave height must be considered in light of the attenuated wave height. That is to say, the wave height generated by wind in the space between the breakwater and the shore must be added to the residual attenuated wave height that bypasses the breakwater. This is because the periods of the two waves will be different, and that every so often the crest of the attenuated wave will match the crest of the newly generated wave, resulting in a higher wave. The height of this larger wave should be the wave height used in the calculations to determine the suitability and/or sizing of the shoreline protection system.

Some manufacturers are also knowledgeable in the required anchoring of the system for floating breakwaters, while others are not – this is an issue that should be addressed by a professional who is experienced in the anchoring of floating breakwaters. Anchorage systems can consist of Helix anchors (www.chance.com) that are literally screwed into the bottom, or mushroom anchors, which dig into the bottom soil, or concrete blocks that rely on pure weight and to some extent partial burial. Generally speaking the Helixes are the most reliable except in areas where the underlying sediments are deep, and soft. In those cases either mushroom anchors or concrete weights are more reliable but exhibit much lower holding capacity than Helixes in corresponding firm soils such as sand or clay. If the anchoring system were to fail a drifting floating breaker would create a navigational hazard, thus it is recommended that whatever anchor system is deployed – a field testing program be considered after installation - using appropriate measuring equipment such as calibrated strain gauges.

Other factors to consider in selecting a floating breakwater for shoreline protection are the depth of water offshore of the newly constructed habitat being protected. In the sample case being considered by this course an offshore breakwater would not be a good choice because it would ground out at low tide, causing a number of problems – not the least of which would be damage to shell fish and their habitat in the immediate underlying footprint. Other potential negative issues to consider are their propensity to collect debris, in the form of both bird droppings, or from recreational boaters that might tie up to them for any number of reasons.

Course Recap:

In Part 3 of the "Dredging and the Environment" course we have learned the basics and a few of the complexities involved in the Dredging of contaminated sediment. Upon completing this course the Engineer should have an understanding of the following:

1. The fundamentals of designing a beach nourishment project, including the field and sampling necessary for planning such a project.
2. The basic methodologies for constructing a beach nourishment project.
3. The fundamentals of designing a wetland restoration/ reconstruction project in a marine estuary including the field and sampling necessary for planning such a project.
4. The basic methodologies for constructing a wetland restoration/ reconstruction project.
5. Basic methods for protecting the shoreline transition from the newly constructed wetland to the adjoining waterway.

Once the Engineer has developed an understanding of these components, he or she should be in a position to go on to study other levels of dredging design.

Appendix 1

Wind Wave Calcs

Wind Speed 1 Minute Time Duration $SSt := 1\cdot 60$ $SSt = 60$ seconds

Wind Speed 45 mph $Ws96 := 45$ mph for 1 minute

Wind Speed 60 mph $Ws117 := 60$ mph for 1 minute

From Figure 3.13, Page 3-29 of SPM for 1 minute to 20 minute duration

One Minute factor = 1.245

20 Minute factor = 1.03

Wind speed adjustment = 1.245/ 1.03 = 1.209

Use 20 minute avg

$Ut96 := \frac{Ws96}{1.209}$ $Ut117 := \frac{Ws117}{1.209}$

$Ut96 = 37.221$ mph $Ut117 = 49.628$ mph

Adjust U to Ua for temperature differential
Assume Winter Storm (cold air) 10 deg F & water 30deg F

Convert temps from F to C $TaF := 10$ $TaC := -12.22$

$TsF := 30$ $TsC := 1.11$

$T\Delta as := TaC - TsC$ $T\Delta as = -13.33$

From Fig 3-14 USA - SPM $Rt := 1.1$

$U96 := Ut96\cdot Rt$ $U96 = 40.943$ mph

$U117 := Ut117\cdot Rt$ $U117 = 54.591$ mph

From Formula 3-28b USA - SPM

$Ua96 := 0.589\cdot U96^{1.23}$ $Ua96 = 56.636$

$Ua117 := 0.589\cdot U117^{1.23}$ $Ua117 = 80.681$

Average time required to build 3 - 4' wave = 20 minutes

Based on Chart 13-3 SPM; 1 min (60 second wind speed) adj to 20 min (1200 sec)

Adjustment factor = 1.01/ 1.24 Wind speed averaged to 20 minutes = 0.81 of 1 minute gust

$WS96\_20 := .81\cdot Ws96$ $WS96\_20 = 36.45$

$WS117\_20 := .81\cdot Ws117$ $WS117\_20 = 48.6$

Wind Wave Calcs

COE - Shore Protection Manual
Wave Forcasting Formulae

Fetch 1 Winter Storm (45 mph)

	Ft per sec	Ft of depth	Ft of fetch	Ft per Sec	
Wave Ht "H1a"	$g1 := 32.2$	$d1a := 25$	$F1 := 16000$	$U1a := \frac{Ua96 \cdot 5280}{3600}$	$U1a = 83.067$

From Formula 3-39

$$Hs1a := 0.283\tanh\left[0.530\cdot\left(\frac{g1\cdot d1a}{U1a^2}\right)^{.75}\right]\cdot\tanh\left[0.00565\,\frac{\left(\frac{g1\cdot F1}{U1a^2}\right)^{.5}}{\tanh\left[0.530\cdot\left(\frac{g1\cdot d1a}{U1a^2}\right)^{.75}\right]}\right] \qquad Hs1a = 0.012906$$

$H1a := \frac{Hs1a\cdot U1a^2}{g1}$ $H1a = 2.766$ Wave Ht in ft

$$Ts1a := 7.54\tanh\left[0.833\cdot\left(\frac{g1\cdot d1a}{U1a^2}\right)^{.375}\right]\cdot\tanh\left[0.0379\cdot\frac{\left(\frac{g1\cdot F1}{U1a^2}\right)^{\frac{1}{3}}}{\tanh\left[0.833\cdot\left(\frac{g1\cdot d1a}{U1a^2}\right)^{.375}\right]}\right] \qquad Ts1a = 1.129$$

$T1a := \frac{Ts1a\cdot U1a}{g1}$ $T1a = 2.912$ Wave T in Seconds

$$L1a := \frac{g1\cdot T1a^2}{2\cdot\pi}\cdot\sqrt{\tanh\left(\frac{4\cdot\pi^2\cdot d1a}{T1a^2\cdot g1}\right)} \qquad L1a = 43.415$$

Appendix 2a

Rock Sizing Calculator

W1 = The minimum rock size in pounds

wr = Unit weight of wetted rock in #/ cu foot $wr := 150$

Ht1 = wave height at site in feet $Ht1 := 4.0$

Sgr = Specific Gravity of Armor Unit relative to local water $Sgr := \frac{150}{64}$

ww = Unit weight of Water in #/ cu ft $ww := 64$

θs = angle of the structure slope in degree from horiz $\theta deg := 35$ $\theta r := \theta deg \cdot .017453293$

Radian Conversion Required $\theta s := \theta r$ $\theta s = 0.611$

Kd = Stability Coefficient (Table 7-8) Guidelines only - refer to Table

Random Quarry Stone Structure Trunk, Breaking Wave	$Kd1 := 2.2$	
Random Quarry Stone Structure Trunk, Non-Breaking Wave	$Kd2 := 2.5$	
Random Quarry Stone Structure Head, Breaking Wave	$Kd3 := 1.9$	(1.5 : 1)
Random Quarry Stone Structure Trunk, Non-Breaking Wave	$Kd4 := 3.2$	(1.5 : 1)

Minimum Rock Sizes

$$W1 := \frac{wr \cdot Ht1^3}{Kd1 \cdot (Sgr - 1)^3 \cdot \cot(\theta s)} \qquad W1 = 1259.273$$

$$W2 := \frac{wr \cdot Ht1^3}{Kd2 \cdot (Sgr - 1)^3 \cdot \cot(\theta s)} \qquad W2 = 1108.16$$

$$W3 := \frac{wr \cdot Ht1^3}{Kd3 \cdot (Sgr - 1)^3 \cdot \cot(\theta s)} \qquad W3 = 1458.106$$

$$W4 := \frac{wr \cdot Ht1^3}{Kd4 \cdot (Sgr - 1)^3 \cdot \cot(\theta s)} \qquad W4 = 865.75$$

Appendix 2b

Rock Sizing Calculator

W1 = The minimum rock size in pounds

wr = Unit weight of wetted rock in #/ cu foot $\quad wr := 150$

Ht1 = wave height at site in feet $\quad Ht1 := 2.0$

Sgr = Specific Gravity of Armor Unit relative to local water $\quad Sgr := \frac{150}{64}$

ww = Unit weight of Water in #/ cu ft $\quad ww := 64$

θs = angle of the structure slope in degree from horiz $\quad \theta deg := 35 \quad \theta r := \theta deg \cdot .017453293$

Radian Conversion Required $\quad \theta s := \theta r \quad \theta s = 0.611$

Kd = Stability Coefficient (Table 7-8) Guidelines only - refer to Table

Random Quarry Stone Structure Trunk, Breaking Wave $\quad Kd1 := 2.2$

Random Quarry Stone Structure Trunk, Non-Breaking Wave $\quad Kd2 := 2.5$

Random Quarry Stone Structure Head, Breaking Wave $\quad Kd3 := 1.9 \quad (1.5 : 1)$

Random Quarry Stone Structure Trunk, Non-Breaking Wave $\quad Kd4 := 3.2 \quad (1.5 : 1)$

Minimum Rock Sizes

$$W1 := \frac{wr \cdot Ht1^3}{Kd1 \cdot (Sgr - 1)^3 \cdot \cot(\theta s)} \qquad W1 = 157.409$$

$$W2 := \frac{wr \cdot Ht1^3}{Kd2 \cdot (Sgr - 1)^3 \cdot \cot(\theta s)} \qquad W2 = 138.52$$

$$W3 := \frac{wr \cdot Ht1^3}{Kd3 \cdot (Sgr - 1)^3 \cdot \cot(\theta s)} \qquad W3 = 182.263$$

$$W4 := \frac{wr \cdot Ht1^3}{Kd4 \cdot (Sgr - 1)^3 \cdot \cot(\theta s)} \qquad W4 = 108.219$$

