# Boating Facilities – Site Analysis

***A SunCam online continuing education course***

## <u>Course Summary:</u>

A very important component of the maritime design process becomes one of properly assessing a perspective site, and advising your client as to the pros and cons associated with site suitability; and in fact must be the first step in planning any maritime facility. This continuing education program is intended to provide the design engineer with the basic essentials for performing several levels of site assessment as appropriate for the structures discussed within this text. These range from simple recreational piers to light commercial facilities. These basics are:

1. Fetch & Wave Climate Forecasting
    a. Determining Baseline Information
    b. Determination of Site Water Level Ranges
    c. Determination of Wind Stress
    d. Determination of Wave Climate
2. Assessment of Site Soil Conditions
    a. Simple & Preliminary Investigation Procedures
    b. More Advanced Investigation Methods

Each of these subjects will take the reader through the step by step process of performing that phase of the pre-design site analysis and will discuss the suitability of each for the respective level of service of the respective docking facilities. The procedures laid out herein are suitable for very simple recreational docks to more sophisticated procedures required for light commercial docking facilities. This course is a prerequisite for the other maritime courses prepared by this author, which include the other design phases of boating similar facility designs. 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 maritime facilities of the size and exposure 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 maritime design. 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 of marine structures without some oversight or guidance from someone more experienced in this field. This is especially important for design of facilities that are exposed to hurricane, high river stages, storm surge or tornado level storms. Rather it 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 the student 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 design 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; we wish to formally thank the COE and acknowledge the contributions and research done by the US Army Waterways Experimental Station, & Coastal Engineering Research Center, Vicksburg, Mississippi for there work in producing these manuals.*

## 1. Simplified Wave Analysis & Fetch & Wave Climate Forecasting:

This is by far the most important component of the site analysis – if a selected site is subject to damaging wave action more than a few times a season - its practical use for a docking facility may be not be feasible. Conversely, if the client has a compelling purpose the designer may need to consider wave attenuation in addition to the docking facility itself. This Study Course will take the designer through the process of simplified wind and wave analysis, and other site related factors, then will do a step by step design of a simple, but functional docking facility and wave attenuator. Wave height and period at a perspective site is one of the first considerations in any site selection process. Opinions on the issue of recommend maximum wave heights at docking facilities vary some what, and also depend on the size and number of boats that will be using the facility. Generally speaking however, if a site under consideration is exposed to waves in excess of twelve to eighteen inches under sustained wind conditions of up to 40 mph, the site should be considered either unsuitable, or in need of some form of wave protection.

Forecasting the potential wave conditions that will occur at a perspective site can be a very complex study – and therefore if the exposures are too complex or if the facility is subject to strong currents or vessels are large, then a more detailed analysis would be required. These more complex tasks are best left to someone who specializes in wave climate forecasting. This text will only address the more simplified methods and sites with limited exposure.

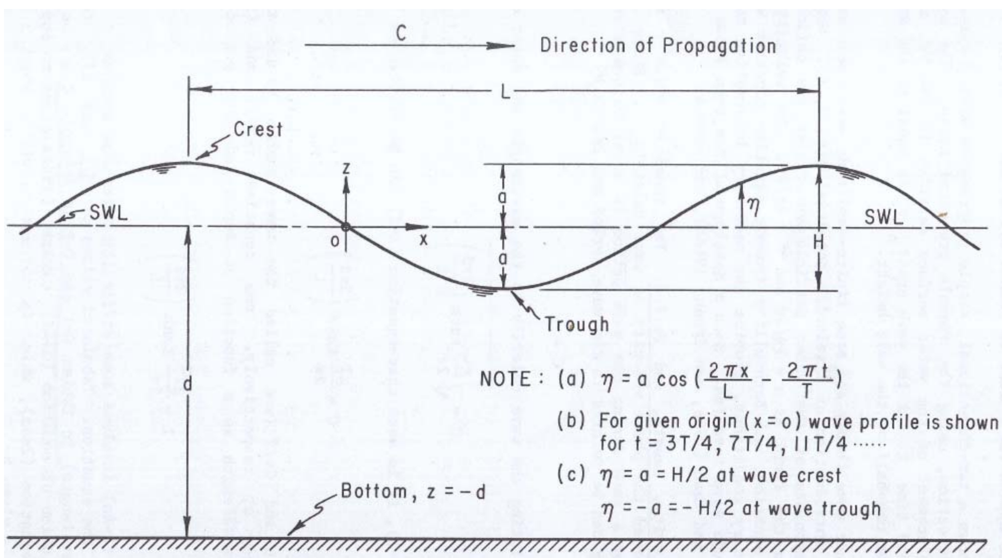



Figure 1: Wave Profile (USA COE Shore Protection Manual)

<u>a. Determining Baseline Information (Wave Form & Site Exposure):</u>
There are several components to a typical wave, as shown in Figure 1, the components that are most used as part of a wave forecast are: Mean Wave Height (trough to crest dimension "H"), Length ("L" the distance between crests), and Period (time interval between crests "P" – which is not shown in figure). Also, the water depth "d" is a significant limiting factor in wave generation, and thus is a critical factor in determining H, L and P. For design purposes of most small boat facilities the most important of these is mean wave height. In a

simplified study there are two places where the wave must be considered, the first is the open water where it develops, and the second is the wave characteristic after the degeneration that occurs as it approaches the near shore areas where the docking facility will actually be located.

Determining the open water wave conditions: This is the stage where the waves are generated by wind at some distance from the proposed docking site. Three factors determine wave height "H" (1) Wind Stress – which is a combination of wind velocity and the difference between air and water temperature; (2) Open water Fetch – the unobstructed open water distance where the waves can develop; and (3) Water depth over the Fetch Distance.

*Note: Current velocity is also a factor, but its consideration is a complex issue and is not covered by this course.*

Figure 2 below is an actual site that will be used as part of this lesson plan to demonstrate some of these characteristics, and to forecast waves for a sample boat docking facility that will be located at the shoreline where the four arrows converge.

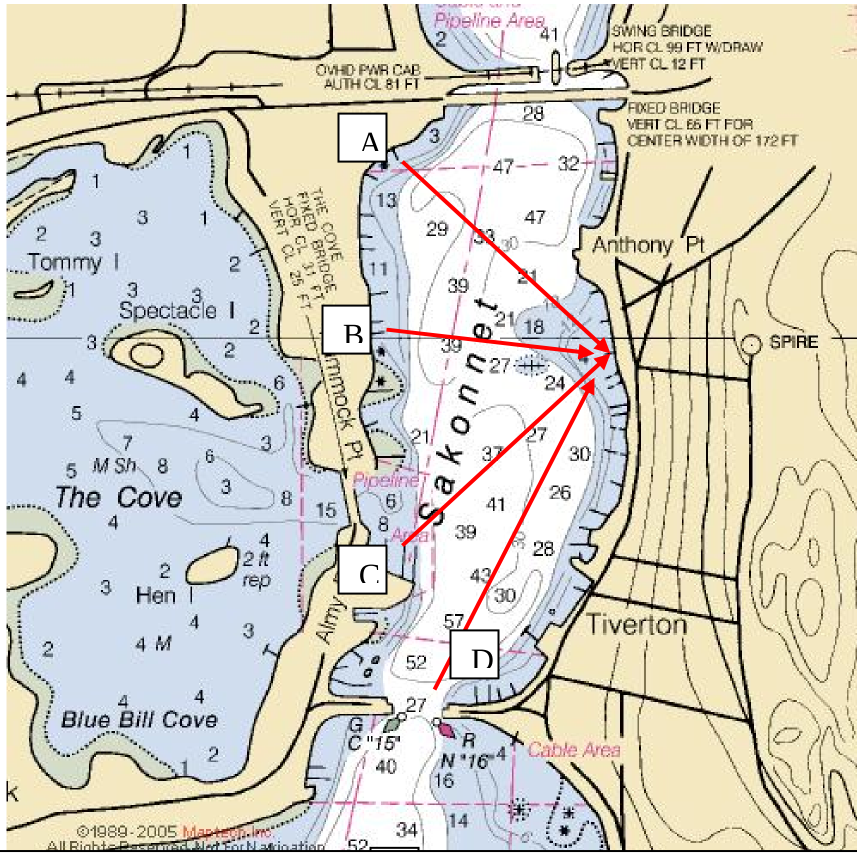



Figure 2: Sample Study Area – Wave Generation

In Figure 2, for the sample case set up for this course - four potential primary fetches are being considered. In assessing a site the engineer typically consults USGS Quad Maps, Navigational Charts (recommended), and Aerial Photogrammetric Images to independently obtain fetch and open water data. For more exposed sites it is also important that detailed near shore bathymetry be sought out – as all of the sources cited above are quite limited in the required details from that perspective.  In Figure 2 above, the red arrows are generally aligned over the “fetch distance”, which is the open water area where the most common waves are generated. The term “Fetch” is the distance from the most distant shore to the site under consideration. Note that in this case four possible fetches are shown, Fetch “A” represents a Northwesterly wind, Fetch “B” represents a due West wind, Fetch “C” represents a Southwest wind, and Fetch “D” represents a South-Southwest wind. (Please note, that if this were an actual case study, the opening in the breakwater on Fetch “D” would represent a “Complex” situation, as one would also have to also consider currents as well as the wave data generated in the outer embayment coming through the opening – as such Fetch “D” should actually be reviewed by a professional who has expertise in wave forecasting. However, for purposes of this analysis, we will presume that the opening in the breakwater does not exist, and that it is a solid barrier. For purposes of this analysis the measured Fetch Distances are as follows:

Fetch A is 2200 feet
Fetch B is 1700 feet
Fetch C is 2100 feet
Fetch D is 3100 feet(*)

The next feature that must be checked is the water depth over the Fetch Distance, since this is a simple study and the fetch distances are relatively short, and the water is generally deep, a simple averaging method will suffice for this study. Normally, if the fetch were long, a profile would be prepared for each fetch so that overall limiting depths could be analyzed, however, because the fetch distances are also relatively short (as well they should be for a small boat facility), simple averaging can be used and will yield sufficiently accurate results. Thus, each fetch will be divided into its quarter points and depths estimated at each location – as well as the starting point and one near shore termination point. These would be as follows:

Fetch A: 12' – 30' – 33' – 20' - 12' = 21.4' MLW
Fetch B: 10' – 39' – 30' – 18' – 12' = 21.8' MLW
Fetch C: 12' – 40' – 37' – 25' – 18' = 26.4' MLW
Fetch D: 50' – 57' – 30' – 30' – 12' = 35.8' MLW

<u>b: Determining of Site Water Level Ranges:</u>
Water depth at a particular site is the sum of two numbers, (1) the water depth (see "d" -Figure 1) based on some datum, and (2) the water elevation fluctuation based on tides, river stages, pool elevations and the like. One very important factor in making water depth determinations is whether the body of water being studied is tidal or subject to flooding. If the body of water is tidal – such as in the subject demonstration case - the datum used is usually on the map

source. In this case, since the data was obtained from a Navigational Chart for a tidal body of water – the water depths are in Mean Low Water (MLW) [Note: some navigational charts may be in Mean Lower Low Water, which yields slightly shallower water depths], USGS Charts are usually in NGVD which is closer to the mid-tidal elevation, however this must be checked for the local adjustment (generally the NOAA tidal web site has this information). If the potential site is located on a river, the river stages and pool elevations are most often in NGVD or NAVD on newer maps. In addition, FEMA maps should be consulted to determine the maximum flood elevation (currently being converted from NGVD to NAVD) (http://msc.fema.gov/webapp/wcs/stores/servlet/FemaWelcomeView?storeId=10001&catalogId=10001&langId=-1). Routinely the FEMA 100 year storm conditions are typically not used for routine design conditions, but rather are used to assess "Storm Survival" wave conditions, which will be addressed briefly later in this text. However, owing to more concern over increased storm activity from climate change, "storm survival" studies have become a required review condition by many regulatory agencies particularly in the Southern States.

Since wind conditions can occur at any tidal stage, and tidal conditions change on a regular cycle - calculations for water depth over the fetch distance must as a minimum take into account the highest predictable tidal (or river stage) elevation – subject to the local conditions. Normal tidal stages can be obtained readily from local tide tables or the NOAA Tide & Current site (http://tidesandcurrents.noaa.gov/). In addition to the normal predicted tides (or river stages) certain areas subjected to prolonged strong storm winds can commonly experience water elevations that rise above or fall below normal predicted levels. Such conditions are not readily available through normal sources, and the design engineer is encouraged to seek out additional advice from local mariners or the local harbor master who have personal knowledge of the local conditions.

To make this process a little more challenging, it is not uncommon to find that there may not be an exact tidal epoch (or stage) for a particular site location unless it happens to be located near a shipping channel or NOAA tidal station observation site. In the case of the subject study area, this happens to be the case. As such, a simplified study method is used to interpolate between the two or three nearest stations, this usually serves to be sufficiently accurate for facilities at the level of service being studied here.
The following is an example of how one would go about determining the tidal stages at the study site in Figure 2, the extremes of water level will be used for several aspects of the design, including the wave forecasts.

(Example): Since the study site is located in the Narragansett Bay, Rhode Island area, the NOAA tidal charts would be consulted as part of the analysis. Upon consulting the tables one would find that there is one tidal station located 0.45 miles to the north called "Anthony's Point", which is sited at 41°-38'-18" North, 71°-12'-42" West, and can easily be found on Google Earth™. The next closest point is Sakonnet which in located 11.5 miles to the south at 41°-27'-54" North, 71°-11'-36" West. The normal tidal range for Anthony's Point is 3.8 feet, and the Spring Range is 4.8 feet; the normal tidal range for Sakonnet is 3.1 feet and the Spring Range is 3.9 feet. It is important to note here that the tidal "Range" is not necessarily the same as the Mean High Water (MHW), or Mean Higher High Water (MHHW) elevation, since the lower of the ranges can, and often are negative elevations below the Mean Low Water (MLW) or Mean Lower Low Water (MLLW) datums. Further, it is important to know that these ranges are the "means" and not the extremes, which can readily be seen in Appendix 1, which is a plot of the tidal cycle for Anthony's Point for September 1, 2008. By looking at the chart one can see that the High Tide for that date was +5.0 feet MLLW and the Low Tide was -0.1 feet MLLW. It can readily be seen that the tide for this date exceeded the MHWS elevation by 0.2 feet. It is recommended that one or two years worth of data be reviewed as part of the review process, which can be accomplished quite easily using tide & current software which is commercially available. In this case we went back one year and found that the lowest predicted tide was -0.7 feet MLLW on March 10, 2008, and the highest was +5.6 on October 27, 2007, which is a classic example of the range of values. MLLW datum as compared to NGVD or NAVD itself must be obtained locally or converted using the NOAA Benchmark Web Site, many times a local surveyor will have these conversions – and since near-shore survey data is necessary for proper facility design, it is good to obtain these conversions at the time of survey. In doing this however, it is important to choose a surveyor that routinely does surveys involved with water related work, and is familiar with the local conditions, as many times the conversion data is not publically available and must be obtained through field level transfers. This would also be a good person to consult about extreme local ranges caused by weather systems.

For purposes of this course, we will assume that we have located such a surveyor, and that this party has the conversion data for MLW, or MLLW and NGVD, further since the study site is only 0.45 miles from the NOAA station we will use the tides from Anthony's Point without adjustment. However, if the study site were located further to the south we would interpolate the tidal ranges based on distance to obtain local conditions. Also we shall assume that the Local Harbor

Master has advised us that during strong southerly or northerly storms the tide can run one foot above or below the predicted tides.

For this test case we also learn that the NGVD is approximately 1.5 feet above MLLW, and from the local FEMA Map, Appendix 2, that the 100 year storm classification is A-13 (El 15) which is the approximate wave crest elevation above the NGVD 0.0 Datum Elevation. From the A-13 classification on A-13 we know that wave heights would be under two feet, thus the mean Still Water Elevation (SWE) would be in the range of +14 NGVD, we adjust this figure to MLLW by adding the 1.5 foot difference to NGVD, and arrive at a 100 year flood Still Water Elevation (SWE) of 15.5 feet MLLW, and a Wave Crest Elevation of 16.5 feet MLLW. This information will be used to determine the maximum wave height and the pier design parameters as well as storm survivability considerations.

Note: More detailed breakdowns of storm surge Still Water Elevations for 10, 25, 50 & 100 year Events can be obtained from the FEMA Flooding studies that usually accompany the Flood Map sets, or directly from the web site under “Flood Studies”.

Using this tidal/ storm surge information it is now possible to calculate the maximum and minimum design water elevations for the various conditions that could occur at the subject site. For this application we should look at the lowest tidal through the highest tidal range to observe where the most vulnerable place lies for wave impact. Additionally we also need to observe what will happen to the facility when a storm surge occurs. Based on this, the next phase of this step is to determine the average water depths over the fetch distance, which will be used for determination of wave height over the four fetches. These would be as follows:

| | Avg Depth @Base Elev | Avg Depth Hurricane @Max TideCondition |
|---|---|---|
| Fetch A: | = 21.4' MLW + 5.6' = 27.0' | +15.5 = 36.9' |
| Fetch B: | = 21.8' MLW + 5.6' = 27.4' | +15.5 = 37.3' |
| Fetch C: | = 26.4' MLW + 5.6' = 32.0' | +15.5 = 41.9' |
| Fetch D: | = 35.8' MLW + 5.6' = 41.4' | +15.5 = 51.3' |

The next step is to obtain the same information for the near shore condition in the vicinity of where the new structure will be built. This step would involve getting a field survey completed along the proposed alignment of the proposed pier location. At the same time the surveyor should also map any pertinent features that would affect the project. These would be abutting piers, shoals, rocks, shoreline structures and the general approaches to the site from landside. It is also very important that the bathymetry (underwater mapping) be indicative of the area, and not just include the proposed pier centerline, a minimum of three lines of soundings should be taken, one on centerline and another row about 25 feet on either side, as well as covering expected navigational approaches to the facility. If the area is rocky, or there are abandoned structures in the area, a diver should also be employed to check for obstructions in the area. Using this data, a preliminary plan and profile of the site would be generated; Figure 3 below is a typical profile, which will be used to simulate the sample project being discussed.

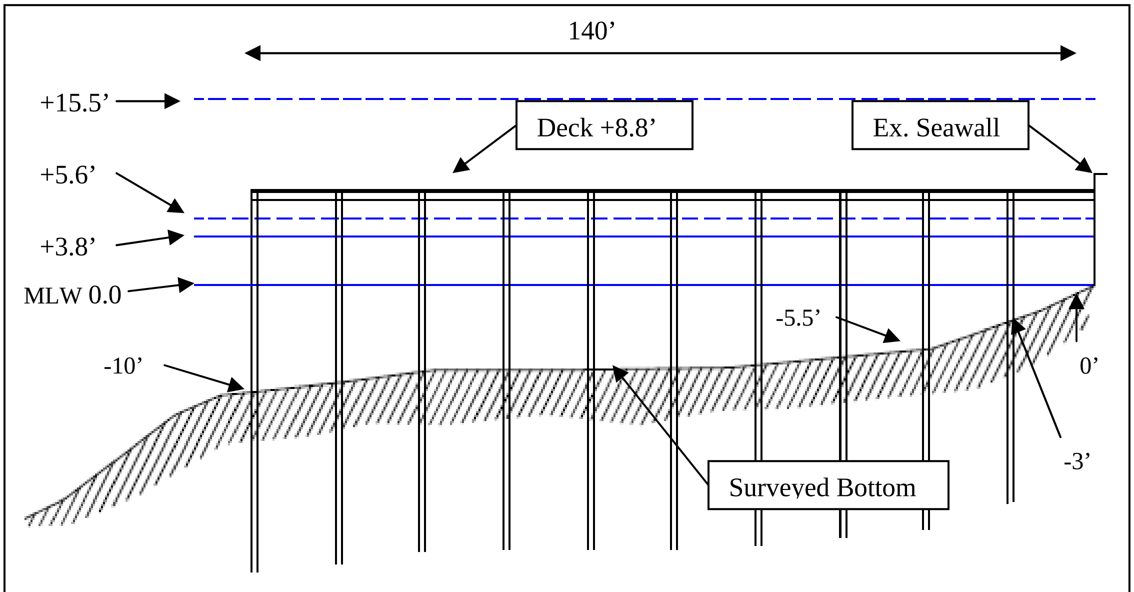



Figure 3: Profile of Proposed Pier Site

Based on the survey, water depths are determined to be 10.0' (MLW) at the outer end of the pier, -5.5' MLW at the 2nd bent, -3.0' MLW at the 1st bent, and 0.0' MLW at the seawall (shoreline). Based on this data the water depths at the various water stage conditions would be as follows:

| | Water Depth | | | |
|---|---|---|---|---|
| Location | MLW | Mean HW | Max HW | Hurricane |
| Outer Bent | 10.0’ | 13.8’ | 15.6’ | 25.5’ |
| 2nd Bent | 5.5’ | 9.3’ | 11.1’ | 21.0’ |
| 1st Bent | 3.0’ | 6.8’ | 8.6’ | 18.5’ |
| Seawall | 0.0’ | 3.8’ | 5.6’ | 15.5’ |

These figures along with the deepwater depths will be used to evaluate the potential wave conditions that would develop at the site under seasonal and survival conditions.

c: Determination of Wind Stress:

The process of practical maximum design wind speed can also be a complicated process, however for small boat facilities in reasonably protected areas one could reasonably look toward the National Weather Service for historical local severe wind conditions. In this case again, the intent is to design for exposure to reasonably predictable storm conditions that occur on a regular basis and then check that design to see that it will survive a 100 year storm(1). For instance, the example case of New England where this site is located, one could reasonably assume that severe storms would be Thunderstorms, which can produce winds up to 50 mph from any direction, but are usually short lived; Northeasters or Northwesterly Gales which can produce winds in the 50 mph range, that can last for several days; and Southeast or Southwest storms which can produce winds in the 40 to 50 mph range usually lasting only a day or two. At this site, hurricanes would be considered the 10 to 100 year storm, and one should anticipate 75 (Category 1) to 130 mph (Category 3) winds as a maximum as well as five to ten feet of storm surge. As an example, Hurricane Bob hit this area in 1991, bringing 100 mph winds and about eleven feet of storm surge above NGVD.

*(1) Note: In Northerly climates where hurricanes are rare it can be impractical to design to hurricane conditions for anything other than survival conditions, however in the Southern US where hurricanes are more common, many designs incorporate Category 1 or 2 hurricane conditions.*

There are several processes that can be reviewed in the US Army Corps of Engineers "Shore Protection Manual" or "SPM" with respect to converting winds measured over land to winds measured over water, as well as wind duration and the height of the measuring instrument. Our experience has been that while these studies are useful on larger or more exposed sites, they tend to be overkill for small recreation boating facilities. Erring on the side of caution, unless the site is particularly prone to hurricanes, tornados or water spouts, using the wind speeds cited in the previous paragraphs will suffice to produce a secure structure. That said, one should also be aware that most wave calculation formulas use a factored wind pressure known as the "Wind Stress Factor" or $U_A$. This number is largely rooted in two processes known as "Stability Correction" and "Coefficient of Drag". The calculation process is found beginning on Page 3-30 of the SPM. Once one has determined the potential for the severity of the wind for the site, additional consideration must be given to the temperature difference between the air and the water. In general the larger the difference, the greater the effect – in simple language Cold air blowing over Warm water increases Wind Stress; whereas Warm air blowing over Cooler water reduces Wind Stress. Thus one must pay close attention to the exposure of sites subjected to cold winter winds, as these tend to be the most damaging.

The correction factor ($R_T$), is a function of the difference in temperature between the air and the water and can be obtained from Figure 4 (next page):

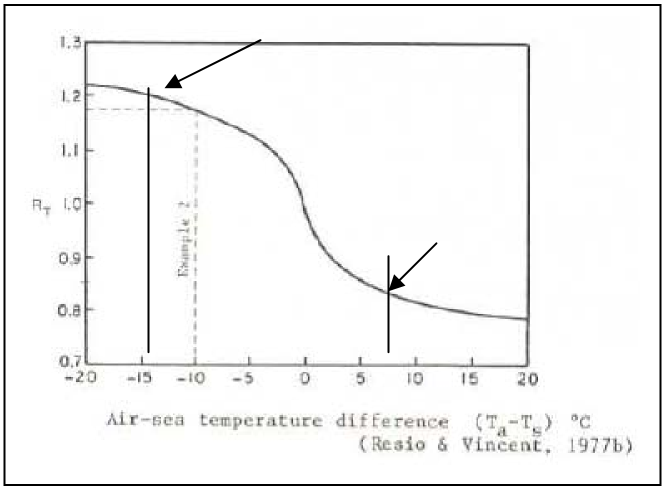



Figure 4: Chart reproduced from COE Shore Protection Manual (Formula 3-14) to find $R_T$ based on Air / Water Temperature Differential (Note that Temperatures are in Celsius)

Thus as an example assume that the air temperature is $20^o$ F, and the water temperature is $45^o$ F. First the temperature in Fahrenheit must be converted to Celsius, so $20^o$ F = $-6.67^o$ C and $45^o$ F = $7.22^o$ C. So if we enter these two values into the formula $T_{as} = T_a - T_s$ then $T_{as}$ = (-6.67)- (+7.22) = $13.89^o$ C (Use $-14^o$). From the chart – reading across the bottom line to -14, and projecting a line straight up until if crosses the curve (arrow), one reads directly an <u>$R_T$ value of 1.2</u>; this would be a common condition for a northerly wind in the Northeastern US. If it were spring in the Middle Atlantic States, and a thunderstorm wind condition occurred then the opposite outcome would occur. As an example, the relatively warm wind – say $60^o$ F, would be blowing over relatively cool water – say $45^o$ F, then $T_a$ $60^o$ F = $15.55^o$ C and Ts = $7.22^o$ C; so $T_{as}$ = 15.55 – 7.22 = 8.33. Therefore <u>$R_T$ = about</u>

0.83. To apply these examples, we will apply them to a wind speed measured about 35 feet (10M) above the ground which is the common function $U_{(10)}$ in the Wind Stress Formula of $U = R_T\ U_{(10)}$. If we apply this to the 50 mph wind speed (measured 35 feet above the surface plane) discussed in the previous section as the "Baseline" storm condition under each of the above examples we would get the following results:

Winter Condition U = 1.2 x 50 = 60 mph
Spring Condition U = 0.83 x 50 = 41.5 mph

Also as a "Survival" analysis, we would also want to check the waves for a hurricane condition, which for this analysis conservatively will be assumed to have an air-sea temperature difference of zero ($70^o$ air $70^o$ water) and $R_T$ value of 1.0. In this case we will assume a Hurricane with 110 mph winds – thus: U = 1.0 x 110 = 110 mph.

Once we have settled on a temperature adjusted U for the condition to be studied, we are ready to calculate $U_A$, which is the basis for almost all wave calculations. The formulas for $U_A$ are as follows:

$U_A = 0.71\ U^{1.23}$ (U in m/s) [Formula 1]

$U_A = 0.589\ U^{1.23}$ (U in MPH) [Formula 2]

Then to take the winter and spring wind conditions in the above examples and using Formula 2 the calculated $U_A$ factors would be as follows:

Winter condition: $U_A = 0.589 \times 60^{1.23} = 0.589 \times 153.86 = 90.62$ mph
Spring condition: $U_A = 0.589 \times 41.5^{1.23} = 0.589 \times 97.77 = 57.58$ mph
Hurricane condition: $U_A = 0.589 \times 110^{1.23} = 0.589 \times 324.27 = 191$ mph

Thus the above examples clearly demonstrate that assessing any maritime study involving waves, and especially structural evaluations, properly assessing water and air conditions that will frequent the site is a critical component.

*Note: Hurricane winds are typically classified using the Saffir-Simpson Hurricane Scale which is based on a <u>1 minute</u> gust; wind conditions used in calculations are usually based on the <u>one hour</u> wind speed. There are procedures within the SPM or CEM to make these adjustments, but typically one can approximate the one hour range by multiplying the 1 minute gust by 0.8.*

<u>Step 4</u>: Determination of Wave Climate:

The next step in this process is to determine what the wave climate will be like in the example case area and then to determine if the site is suitable for the docking of the type of boat that the hypothetical client is looking to moor, what loads will be generated on the planned structure, and as what protective measures might be needed. From Step 2 of this exercise we have determined the average water depths across the fetch distance, as well as the tidal and storm water fluctuations. We also know the water depths at our proposed pier site for these various conditions. The accepted formulae for calculating wave heights and periods are taken from the US Army Corps of Engineers "Shore Protection Manual" (SPM), or the more recent "Coastal Engineering Manuals" (CEM) these are shown below in Formulas 3 and 4:

Equation 3-39 of The Shore Protection Manual

$$H(d,F,U_A) \quad \frac{U_A^2}{g} \; 0.283 \tanh \left[ 0.530 \left[ \frac{gd}{U_A^2} \right]^{3/4} \right] \tanh \left[ \frac{0.00565 \left[ \frac{gF}{U_A^2} \right]^{1/2}}{\tanh .530 \left[ \frac{gD}{U_A^2} \right]^{3/4}} \right]$$

Formula 3: Significant Wave Height Equation (COE SPM Eq 3-39)

Equation 3-40 of The Shore Protection Manual

$$T(d,F,U_A) \quad \frac{U_A}{g} \; 7.54 \tanh \left[ 0.833 \left[ \frac{gd}{U_A^2} \right]^{3/8} \right] \tanh \left[ \frac{.0379 \left[ \frac{gF}{U_A^2} \right]^{1/3}}{\tanh \left[ 0.833 \left[ \frac{gd}{U_A^2} \right]^{3/8} \right]} \right]$$

Formula 4: Significant Wave Period Equation (COE SPM Eq 3-40)

Formula 3 (Equation 3-39) is used to determine significant wave height (H), and Formula 4 (Equation 3-40) is used to determine the wave Period (T). In these formulae $U_A$ is the adjusted wind stress factor from Step 3 of this document [(2)], "d" is average water depth in feet, "g" is gravity in ft/sec$^2$, and F is fetch in feet. These formulae can be entered into either electronic spreadsheets or programs such as MathCad™, and results derived rather quickly once the programs are set up. However the SPM has also reduced them to charts that cover $U_A$ of up to 150, and water depths of up to 50 feet; these charts are reproduced for the convenience of the reader as Appendix 3. If this were a larger facility with more exposure, we would recommend going to the effort of setting up the formulae in a packaged software program – however the exposure and scale of this site, and indeed for almost any feasible facility within the scope of this study document, the charts are more than adequate.

*(2) Note : It is very important to not here that the charts in Appendix 3, use an adjusted wind speed ($U_A$) in MPH, however if one uses the formulae given above, the units of $U_A$ must be converted to Feet per Second in order to be consistent with the units used in the formula. Also, if developing one's own packaged software system – be sure to check the results obtained with the Charts in Appendix 3, as some programs use the function "tanh" in radians, while other use degrees, which can yield confusing results.)*

First we will calculate the deep water wave conditions using these charts starting with Fetch "A", which is 2200 feet of exposed water surface. At this point we must assess which wind conditions would be appropriate for this Northwesterly fetch. Typically, winds from the west and northwest are winter winds and are much colder than winds that come from a southerly direction. Hurricane winds emanate from a tightly compressed storm system, and thus depending on whether a storm passes either to the east or west of a particular site winds could clock through virtually any direction. As such we will use the wind conditions for a northerly winter storm, or a $U_A$ of 90 mph (rounded from 90.62), and also for Hurricane conditions of 191 mph; the warm wind condition will be neglected from this case. For Fetch "A" the Maximum predicted tidal condition was 5.6 feet, and the average water depth over Fetch "A" for this tidal condition was 27 feet. To obtain approximate wave data for this condition we would go to Appendix 3d, wave forecasts for depths of 30 feet. Both scales on this chart are logarithmic for fetches from 0 to $10^6$ feet, and $U_A$ from 10 to 150 mph. To determine the wave data, first read the bottom scale and find 2200 feet; then locate a $U_A$ of 90 mph on the scale on the right side of the chart. Make a mark where the two lines intersect, then read the internal curve sets on the chart. There are curve sets for "H" significant wave height (ft), "T" wave period (sec) and "I" the time in minutes under the given wind condition for the sea to reach the given state. Starting with the curve set "H", one would read that the dot placed on the chart fell between 1.5 and 2.0 feet. Using a scale between the two curves one gets an approximate wave height of 1.7 feet. Then going to the set of curves labeled "T", again one would see that the dot falls between values of 1.5 and 2.0, again using a scale between the two curves one gets a value of “T of about 1.8 seconds. This represents the size and frequency of waves that will be approaching the pier from deeper water. Even under these severe winter conditions a 1.7 foot wave is not considered severe, but if the boat were not properly moored, it could incur damage. Just for comparison go to the bottom scale and read up to the 90 $U_A$ line from a 5000 foot fetch, now one would read waves in the 2.5 foot range with periods (T) in the 2.4 second range – these wave conditions would require very careful and strong mooring to prevent damage to a recreational boat. Thus the importance of this study quickly becomes evident. As a further point of interest, it has been the writer’s experience that when cold winds blow over moderately warm water for extended periods of time these charts tend to under-predict the wave severity and impact loads – thus erring on the side of conservatism in cooler climates is highly recommended. The next step in analysis of this fetch is to examine the conditions as the wave contacts the head end of the pier, and approaches shore, this is where breaking wave conditions must be checked, as they tend to create high loads. A general rule of thumb that will save a lot of calculation time is to assume that almost any normal wave will break when it reaches a steep rise in bottom elevation and water depth diminishes to about 1.5 times the wave height. Thus a 1.7 foot wave would break in about 2.5 feet of “Still water” elevation, (Still water =

the water depth at a given point if the seas were calm) and a 2.5 foot wave would break in 3.75 feet of water. As a check on breaking go to the charts labeled 10 feet and 5 feet of water depth. Reading up from 2200 feet to a $U_A$ of 90, one reads a wave height of 1.6 feet, which is only a small decay, and even on the 5 foot chart, the wave height at 2200 feet and a $U_A$ of 90 is only diminished to 1.5 feet. This is as far as the charts go, and in fact to calculate the breaking point using more sophisticated programs would be quite time consuming – but again considering the size of the waves at this location, such efforts are usually not necessary and are given here only to demonstrate the procedures.

One last check is required of this fetch, and that is to calculate the hurricane conditions, unfortunately our calculated wind condition of a $U_A$ of 191 mph falls somewhat off of the chart, but this can be approximated by extending the logarithmic scale up just past 150. In this case the projected storm surge would take water depths to about 37 feet, and so one would go to the 40 foot chart. Going to the bottom scale and drawing a line up from the 2200 foot fetch and projecting it part the top line of 150 about ¼", and extending the "H" line for the 3.5 foot wave; one can see that the three locations coincide in about the same place. In a similar manner extending the 2.0 and 2.5 second period lines above the chart one can see that an approximate period of 2.2 to 2.3 seconds should be expected. Since there would be about a 15.5 foot storm surge with this condition the only breaking condition that would occur would be in the vicinity of the seawall, which at some point would be submerged. There is not realistic way to judge structural conditions in weather conditions of this magnitude, since there are many other factors that enter into the picture – such drifting boats, houses and their likes. The only purpose of this part of the study is to develop a sense of survivability of the structure, so that large portions of it do not break loose and add to the damage. The most severe conditions occur as the storm surge is rising as the storm approaches, and waves begin to break on the deck of the pier. In these conditions all one can rely on is experience and common sense when it comes to building in safe guards that will contain the structure from breaking apart. These practices will be discussed briefly at the end of this study, and covered in more detail in more advanced courses.

Once one is done with the analysis of Fetch “A”, the same procedures would be carried out on fetches “B”, “C” and “D”. It should be noted that Fetch B would experience both winter and summer wind conditions, and that Fetches C, & D would only experience the summer conditions. Thus a table of all of the above conditions for this subject site would be as follows:

| | Wave "H" NW Wind | Wave "T" NW Wind | Wave "H" SW Wind | Wave "T" SW Wind | Wave "H" Hurricane | Wave "T" Hurricane |
|---|---|---|---|---|---|---|
| Fetch A: = | 1.7' | 1.8 sec | - | - | 3.6'(3) | 2.4 sec |
| Fetch B: = | 1.5' | 1.7 sec | 1.0' | 1.4 sec | 3.2'(3) | 2.2 sec |
| Fetch C: = | - | - | 1.1' | 1.5 sec | 3.6'(3) | 2.3 sec |
| Fetch D: = | - | - | 1.4' | 1.7 sec | 4.3'(3) | 2.6 sec |

(Note: 3= verified through use of formulas 3 & 4)

Step 2: Soil Conditions & Pile Supports

The work thus far in the site evaluation determines the water related environment – however the design of this next step requires some thought to the submarine environment. This part of the evaluation involves determining what conditions exist with respect to pile vertical support, pull out and lateral load resistance. This requires exploration of the site soils as the soils are what will provide the pier's support piles with respect to both horizontal and vertical resistance. With the possible exception of underlying rock formations, pile support geology in the marine environment is very rarely consistent with the conditions found near or beneath the adjacent shore itself. For example one cannot assume that because sandy or gravelly surface soils are present near the shoreline, that the underlying soils are the same or even similar in the waterway a hundred feet from shore. Coastal and riverine soil conditions have typically been in a state of flux for hundreds or thousands of years and the prudent engineer should always be mindful of this fact. As such soil exploration is a very important component of pier design that is often neglected, sometimes with very unpleasant results.

Consideration must also be given to the soil conditions where the piles will be driven, and one should never assume that they will be what we desire. Additionally - often small and recreational facility project budgets cannot bear the expense of mobilizing a floating rig to take either split spoon or vibracore samples. These explorations can be pricy depending on the availability of firms that perform this specialty from floating equipment, and costs can be further exacerbated if the site is exposed to wave action. As a practical matter however, depending on the type of project, and the loads that the piles will be subjected to - more sophisticated sampling methods may or may not always be necessary. If the project is extremely simple, some of the alternatives given herein may be considered, however if the budget will allow for it, we almost always recommend the more conventional sampling and testing methods. Whether or not the client decides to follow the engineer’s recommendations at this point is another matter altogether, and in the end we as the designer must fall back to our own judgment as to what is reasonable and prudent with respect to sub-surface exploration.

The level of need for which particular type of soil sampling method is a function of the anticipated level of service for the facility. The scope of this study is to focus on the simpler cases such as recreational facilities with minimal exposure, upwardly to light commercial facilities that have more exposure and have greater potential to carry heavier loads. Conversely, heavier timber piers for public or commercial use, and load carrying structures such as travel lifts and ferry berths fall outside of this classification. To determine where the individual example case lies along the spectrum takes a measure of engineering common sense as well as a hard look at the anticipated end use of the structure. With that said, we would also say that one should also never undertake a project of this nature without some form of soil information, as the marine environment does have a tendency to conceal many unpleasant surprises in that respect. Another trap that should be avoided is making the assumption that one can take a soil boring on land, near the shoreline using a conventional soil boring rig and assume that the conditions found there on the shore would be similar to those found further from shore. Such assumptions will more often than not yield very inaccurate results.

The types of soil exploration that are most commonly used in the marine environment are as follows – discussed here in an order from the least to most sophisticated:

1. "Probing" the soils from a boat is relatively fast and inexpensive; this method of soil exploration is suitable for light duty structures with limited exposure, or as a supplement to a split spoon sampling program. <u>This method should not be used where piles are subjected to heavy vertical or lateral loads,</u> or where the facility will be used for public access. Additionally, the procedure should only be

also be performed by personnel experienced in doing probes of this nature. Good field notes, recording of tides during the process and accurate positioning of the probe locations are critical to obtaining suitable information. Typically for light duty structures the probes are advanced about 20 to 25 feet into the soils. The probing process uses a ½" pipe or steel rebar, marked in one foot increments. The probe is first carefully lowered until one "feels" the bottom, then it is allowed to penetrate under its own weight – the depth and thickness of this layer is then recorded. Then the probe is gradually advanced using more and more pressure, while the person doing the probe listens and feels the probe for the heavy or light "crunching" that would indicate the presence of sand or gravel. Generally speaking the soils in the first layer that the probe penetrates under its own weight should not be considered as structurally suitable, likewise any soils that allow penetration with anything less than a hard "bouncing" of the probe using its own weight to obtain advancement. Soils that are sandy and through which penetration is obtained only by hard "bouncing" or "stabbing" of the probe are usually suitable for moderate lateral loads - as the pile driving process tends to consolidate these layered soils in the immediate vicinity of the pile. When practical refusal is reached this is an indication that a suitable bearing soil layer has been reached at the tip of the probe. From this point, a second ½" to ¾" pipe probe is used with a water pump and hose attached to the upper end. This is known as "jet probing", a process that uses the velocity of water flowing through the pipe to clear the sample hole. The probe is advanced by jetting, then the water is stopped and the probe is pushed and "bounced" into the bottom of the probe hole, noting the amount of resistance and again listening for the crunching sound indicating sand or gravel. Stiff clay could also be encountered, and is generally found suitable, and will not yield a crunching feel, but will feel very stiff and the probe will have a tendency to "stick". In clay soils, sometimes pieces of clay adhere to the probe itself when it is withdrawn. When "wash probe" refusal is reached, one will need to determine if the refusal was caused by skin friction on the pipe, very compact soils or gravels, or hard rock (which most often produces a ringing sound).

2. Vibra-core sampling is another method of obtaining soil data, and is usually faster and therefore less expensive than split spoon sampling; however it also has several limitations. First, it can tend to homogenize the sample and may give misleading results, secondly – it also does not produce a blow-count record, so soil classification and load bearing assumptions are strictly based on visual examination and

bench testing of the samples once they are removed from the sample tube. This type of soil evaluation should only be done by an individual experienced in marine soil geotechnical evaluation. Properly evaluated and tested, vibra-core samples can give a general indication if the soils are suitable for providing lateral support for piles, however they will only give limited information with respect to the pile bearing support value.

3. Driving a “test” pile can also give good and practical information and can be used as a confirmation of any of the soil exploration methods discussed herein. It is generally advisable to drive the pile with an impact hammer in lieu of a vibratory hammer, as the impact hammer allows blow counts to be recorded in the process. This is an important factor in determining the “un-supported length” which is important in determining the allowable lateral loads that the pile will be able to withstand. In addition real lateral support data can be obtained by performing a lateral load test in the field after the pile is driven. This consists of moving the pile driving rig off to one side of the freshly driven pile then applying a horizontal pull using a winch and crane scale. (Note: <u>It is very important that these lateral load tests be conducted only individuals experienced in these types of operations, and with appropriate safeguards in place - as they have the</u>

potential to be very dangerous to the inexperienced). As the load is applied the deflection of the pile is recorded and a curve produced that plots deflection versus load. Because the pile can fail at any point in the testing process it is also advisable to use a sacrificial pipe pile of about the same diameter as the pile that will be driven, this way if the pile fails during the test it can still be extracted from the bottom.

4. Split Spoon Sampling is the preferred method of soil exploration, and is conducted using one of several types of sampling rigs which are commonly available; however finding a firm experienced in borings in the marine environment can also be quite challenging. The most common methods range from installing a conventional sampling rig truck on to a suitably sized barge, to something as simple as mounting a tripod type drop hammer sampling rig on a pontoon boat or raft. Again, this type of sampling should only be attempted by individuals with experience in marine geotechnical experience.

What is equally important in split spoon sampling and is often neglected is having good horizontal positioning available to locate the sample locations on the site map, as well as methodical and accurate recording of tides, correlated to the time each individual sample is started and finished. As an example, if it takes 20 minutes to drive a split spoon 24”, and the tide rises four inches during this sampling period and is not corrected, both the length of the core sample and the depth that it was taken will have error introduced into them. This is extremely important where tidal exchange exceeds more than two feet, and/or where slow driving or coring conditions are encountered. If rocky conditions are encountered – we also recommend having rock-coring equipment available and coring at least a few feet into any rock that is encountered. This will help define the rock quality as well as help differentiate between finding true bed-rock or simply hitting a boulder.

Figure 7: Example of simple floating split-spoon tripod sampling rig

The location of the borings, the depths and the number of samples required is subject to the individual site evaluation. Where the site is reasonably undisturbed and the structure is simple one or two sample locations may suffice, sites that have been disturbed by prior construction (especially dredging), and site explorations for heavier structures require more investigation. Many times this is a factor that is determined in the field, if a few samples yield consistent results then fewer samples are required; conversely inconsistent results and complexity of the project in turn requires more samples. This factor is again more one of experience and local knowledge than anything else.

Once soil information is obtained, it is important to have it reviewed by an individual experienced in marine geotechnical analysis to determine the minimum pile embedment length as well as the lateral support characteristics. This information must them be weighed against the service level of the anticipated structure, obviously the more severe the service and exposure the more important this information becomes in the design process. Part II, which is the next level of this course will apply all of the above logic this into our test case for a light duty commercial pier with a fixed wave break (wave wall).

Course Recap:

In Part I of this course we have learned the basic steps for obtaining baseline information with respect to determining site suitability for Recreational and Light Commercial Piers and other equivalent structures of Maritime usage. Upon completing this course the Engineer should have a basic understanding of the six most important components of design for these basic level marine structures, these are:

Fetch & Wave Climate Forecasting

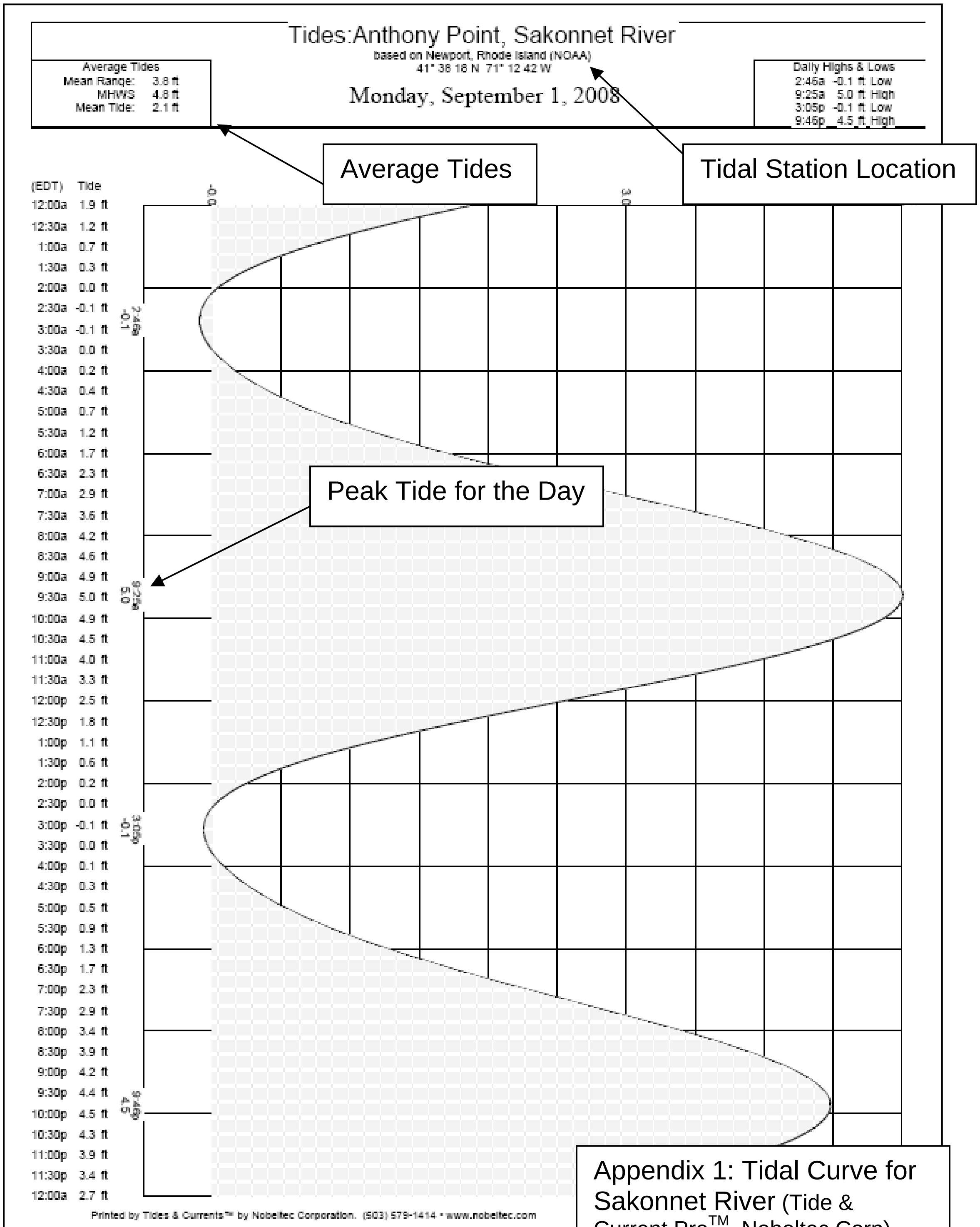
a. Determining Baseline Information
b. Determination of Site Water Level Ranges
c. Determination of Wind Stress
d. Determination of Wave Climate

Assessment of Site Soil Conditions

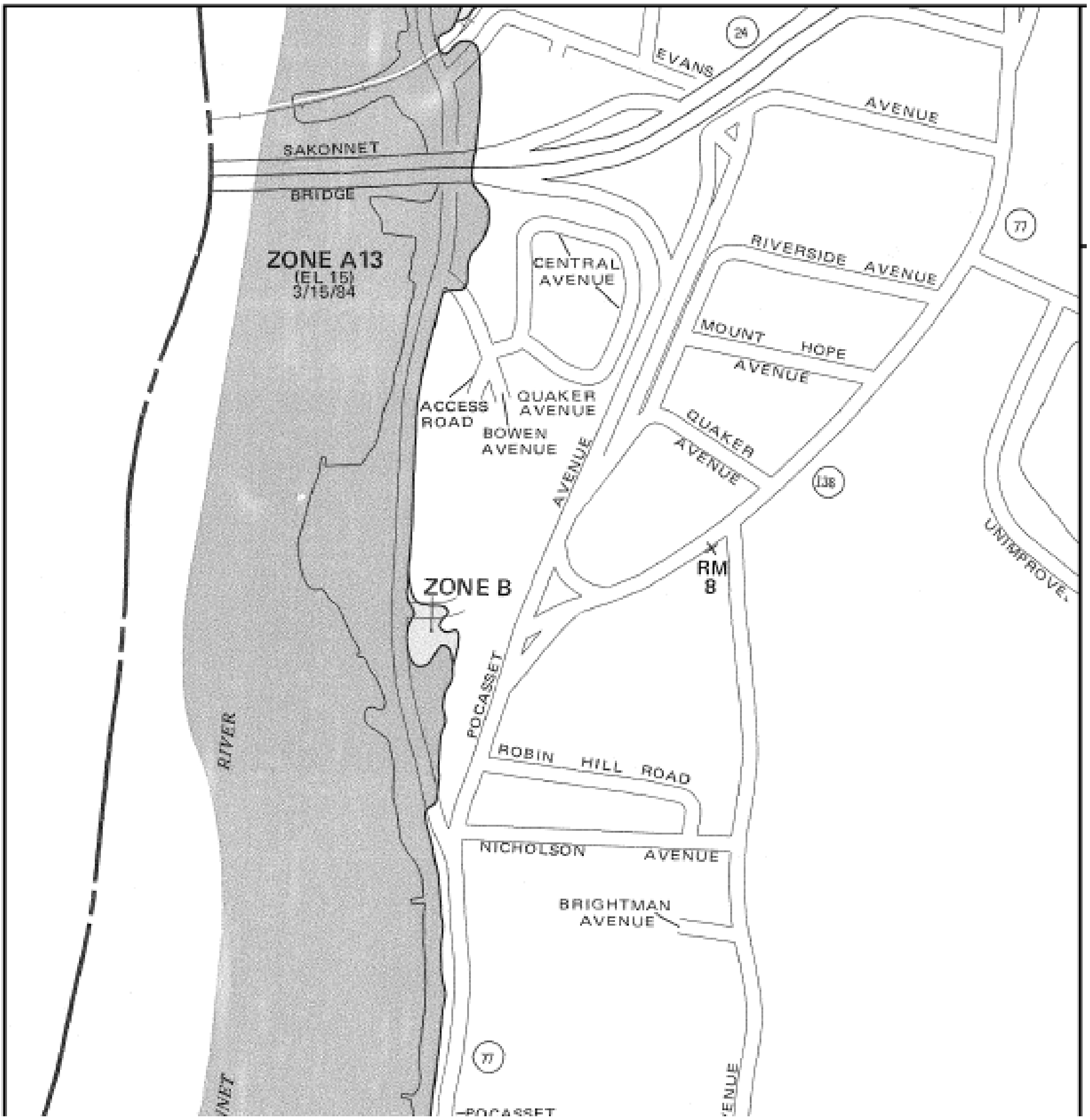
a. Simple & Preliminary Investigation Procedures
b. More Advanced Investigation Methods

Once the Engineer has developed an understanding of these components, he or she should be in a position to go on to study the next levels of maritime design. Part II of this course will go on to undertake design of a basic light commercial pier and wave break, which will use almost all of the basic site analysis procedures covered by Part I. In addition, the understanding of Part I will also allow the individual to take continuing education courses in other areas of maritime design, such as layout of maritime facilities, design of floating docks, wave attenuation, coastal revetments, and bulkheads, as well as more advanced subjects such as design for storm survivability.

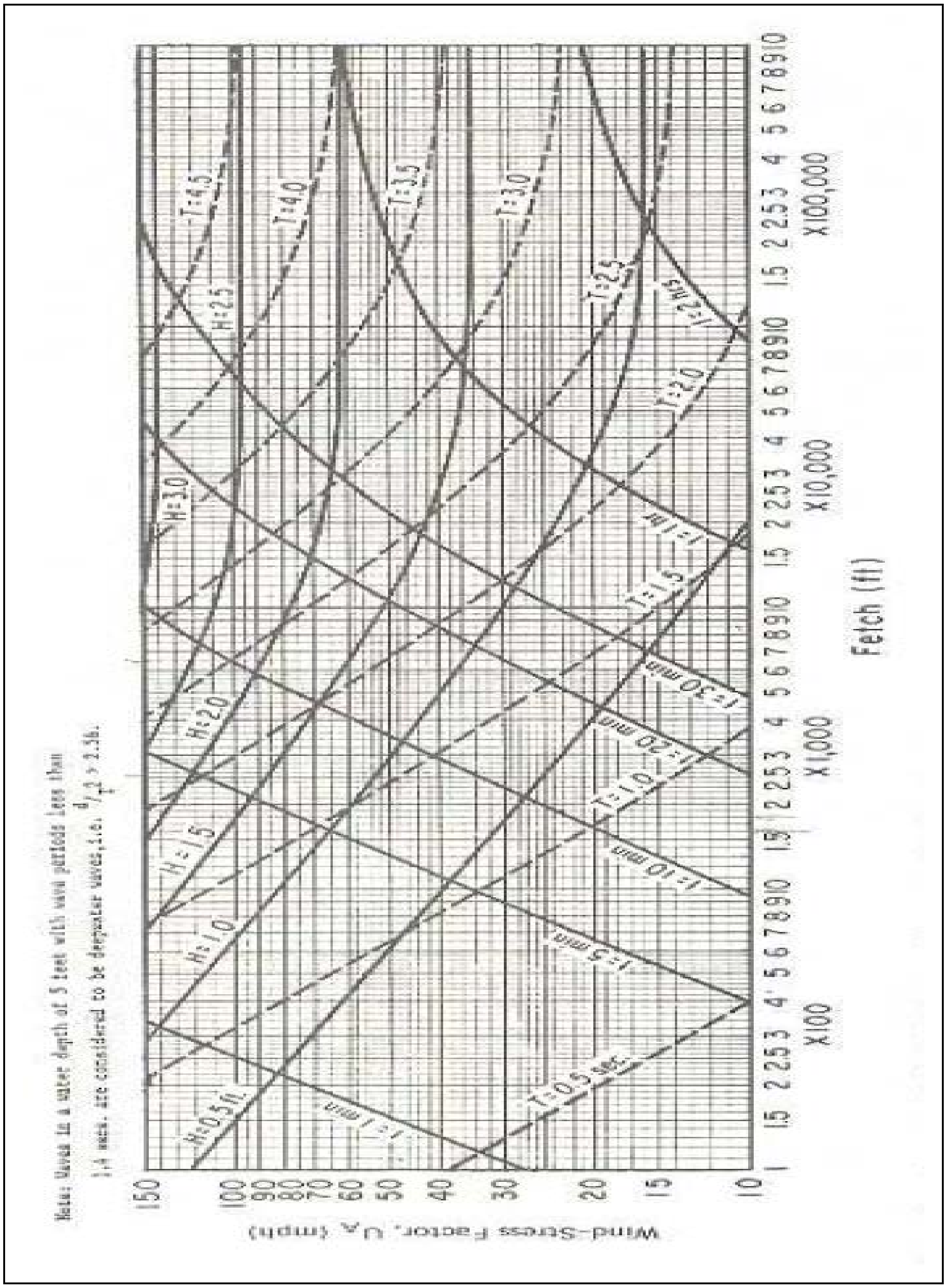
# APPENDICIES



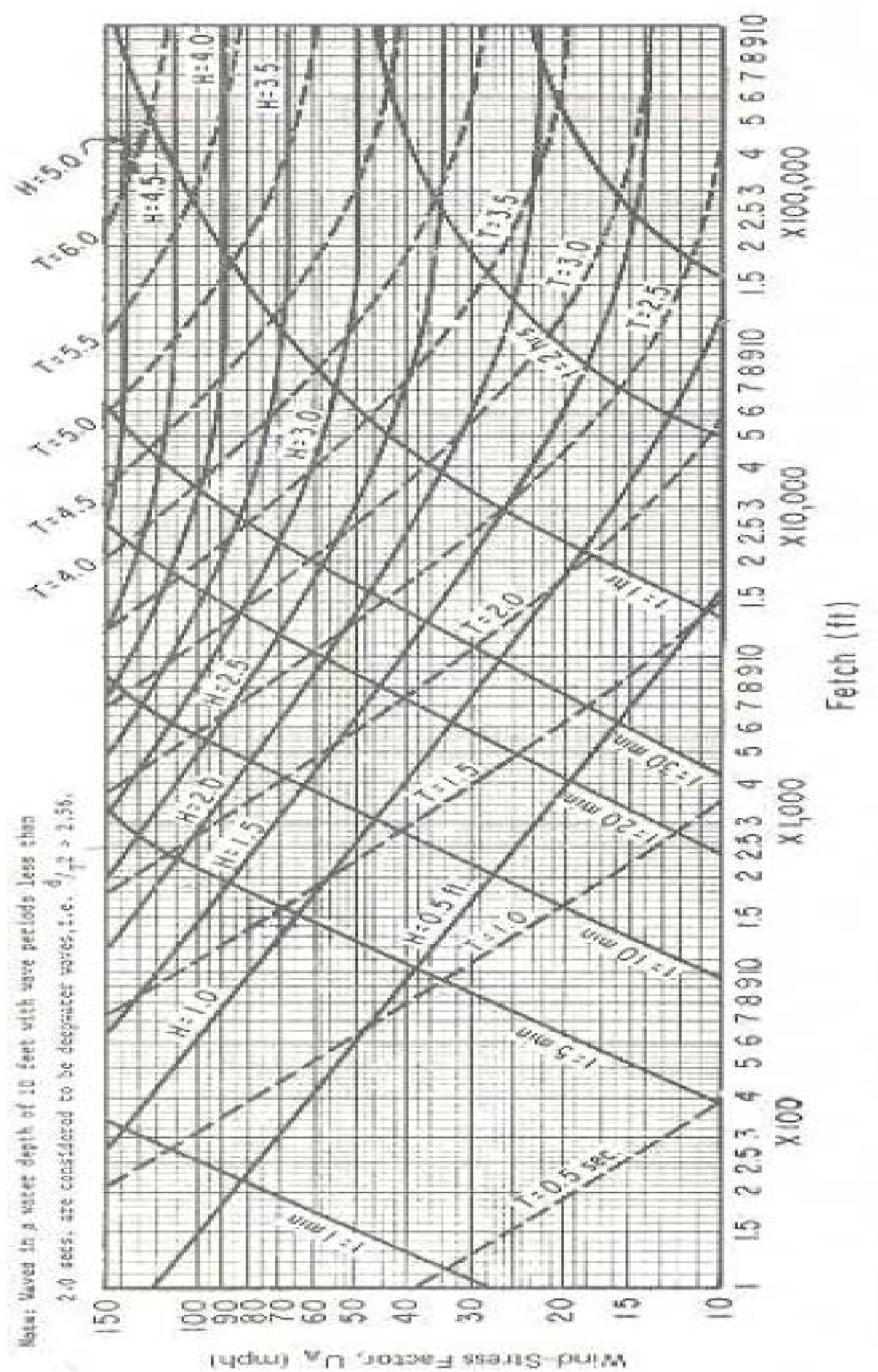

Appendix 1: Tidal Curve for Sakonnet River (Tide & Current Pro™, Nobeltec Corp)



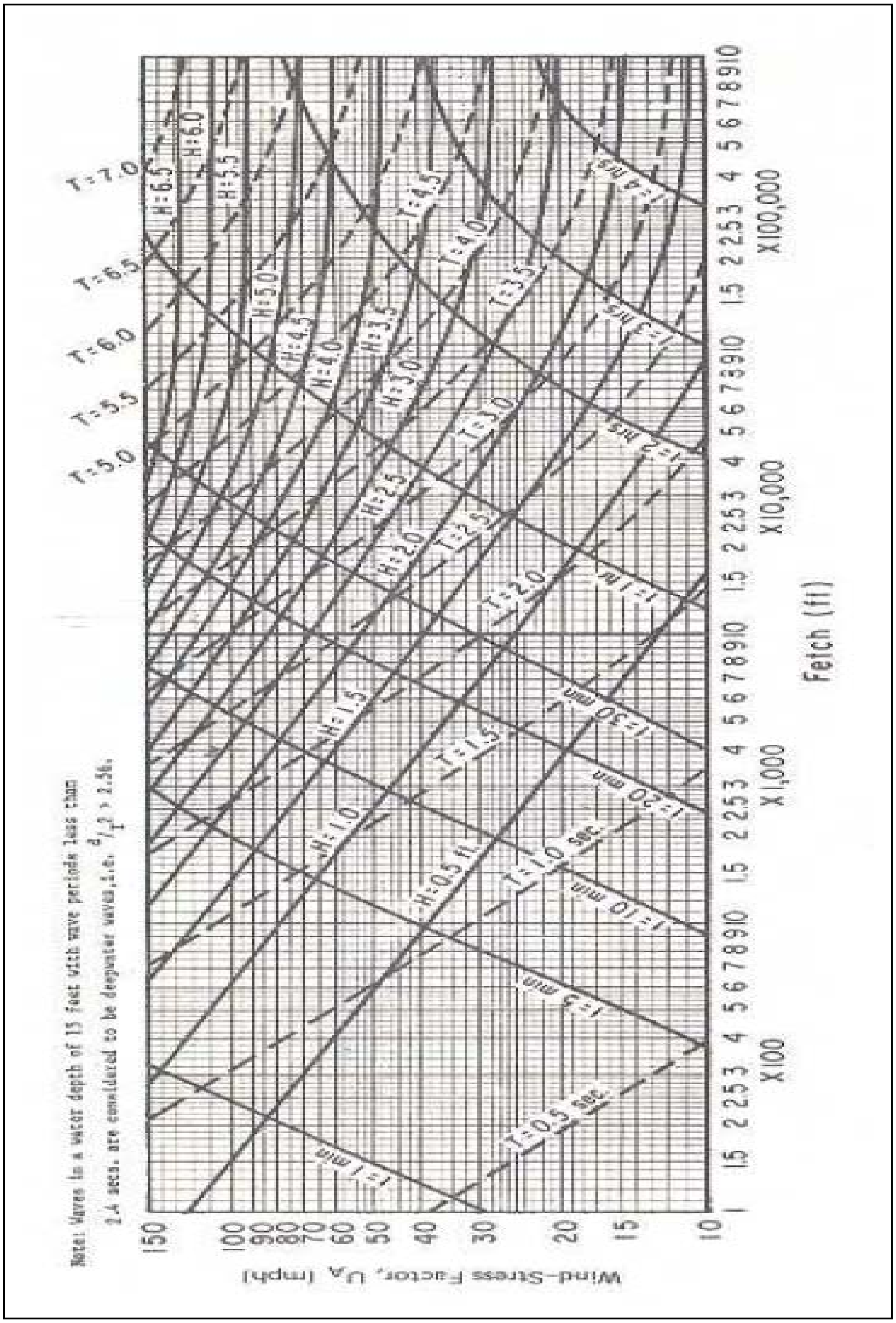

Appendix 2: Portion of FEMA Flood map, Tiverton, RI



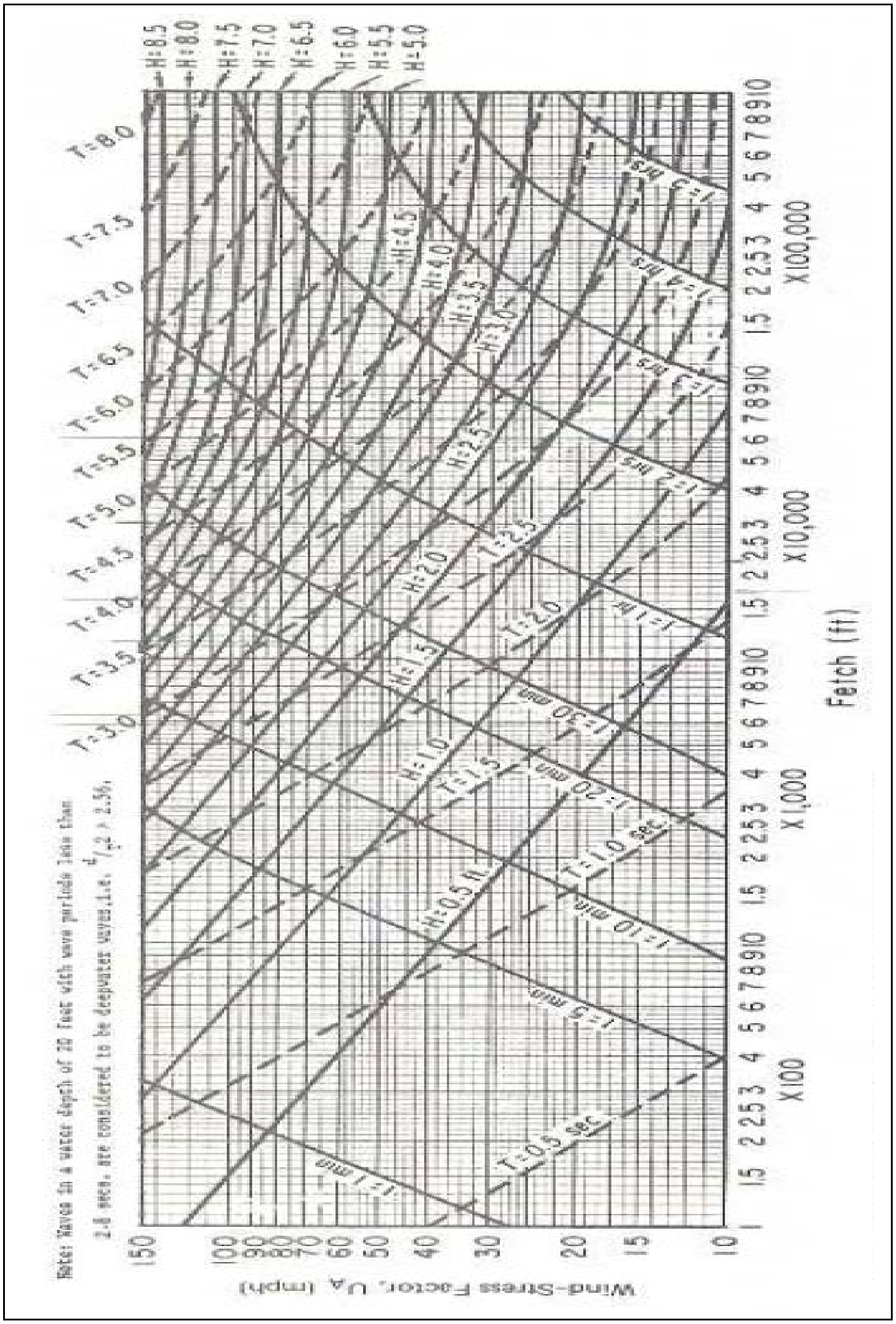

Appendix 3a: Wave Forecasting Chart for water depths of 5 feet (USA Shore Protection Manual)



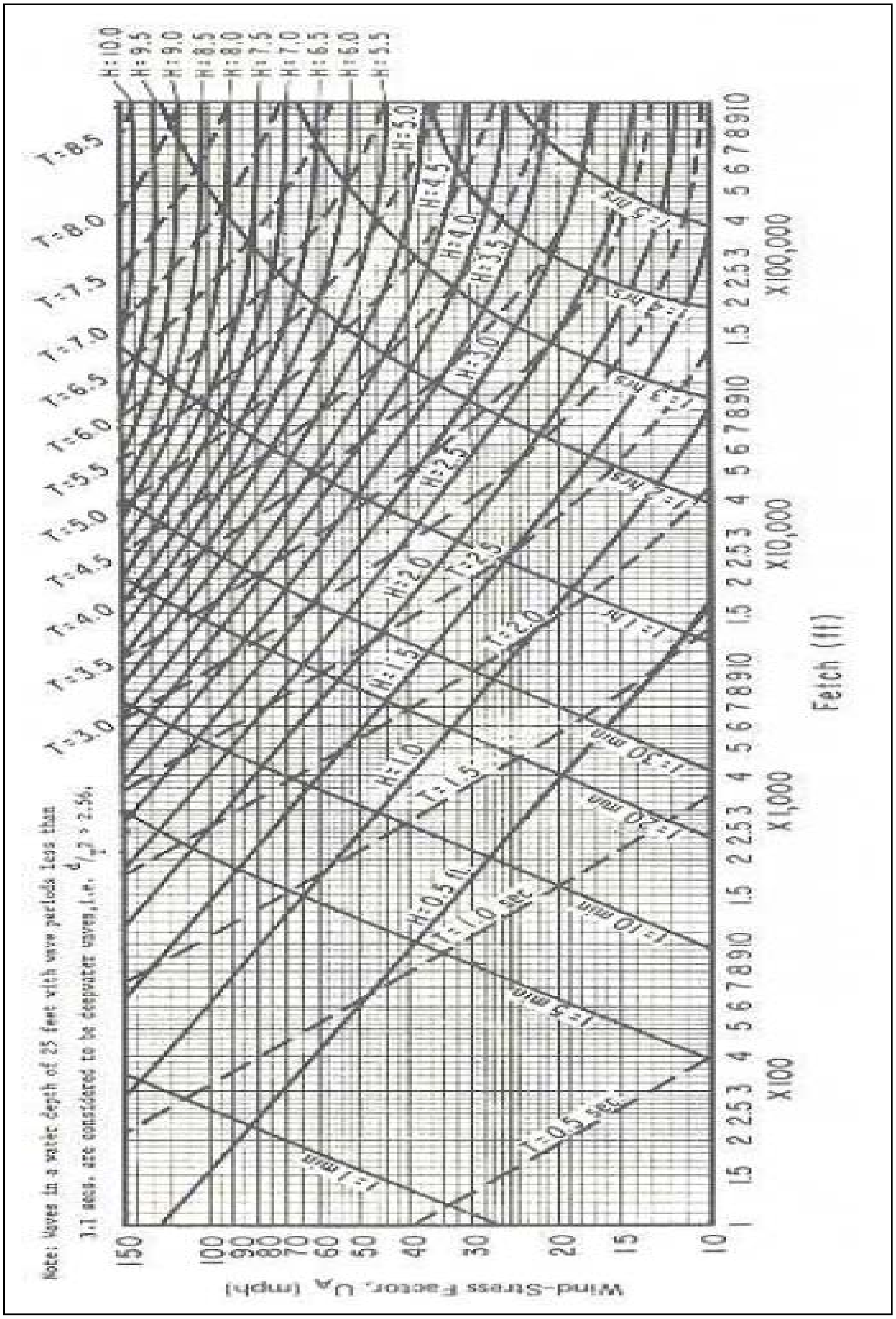

Appendix 3b: Wave Forecasting Chart for water depths of 10 feet (USA Shore Protection Manual)



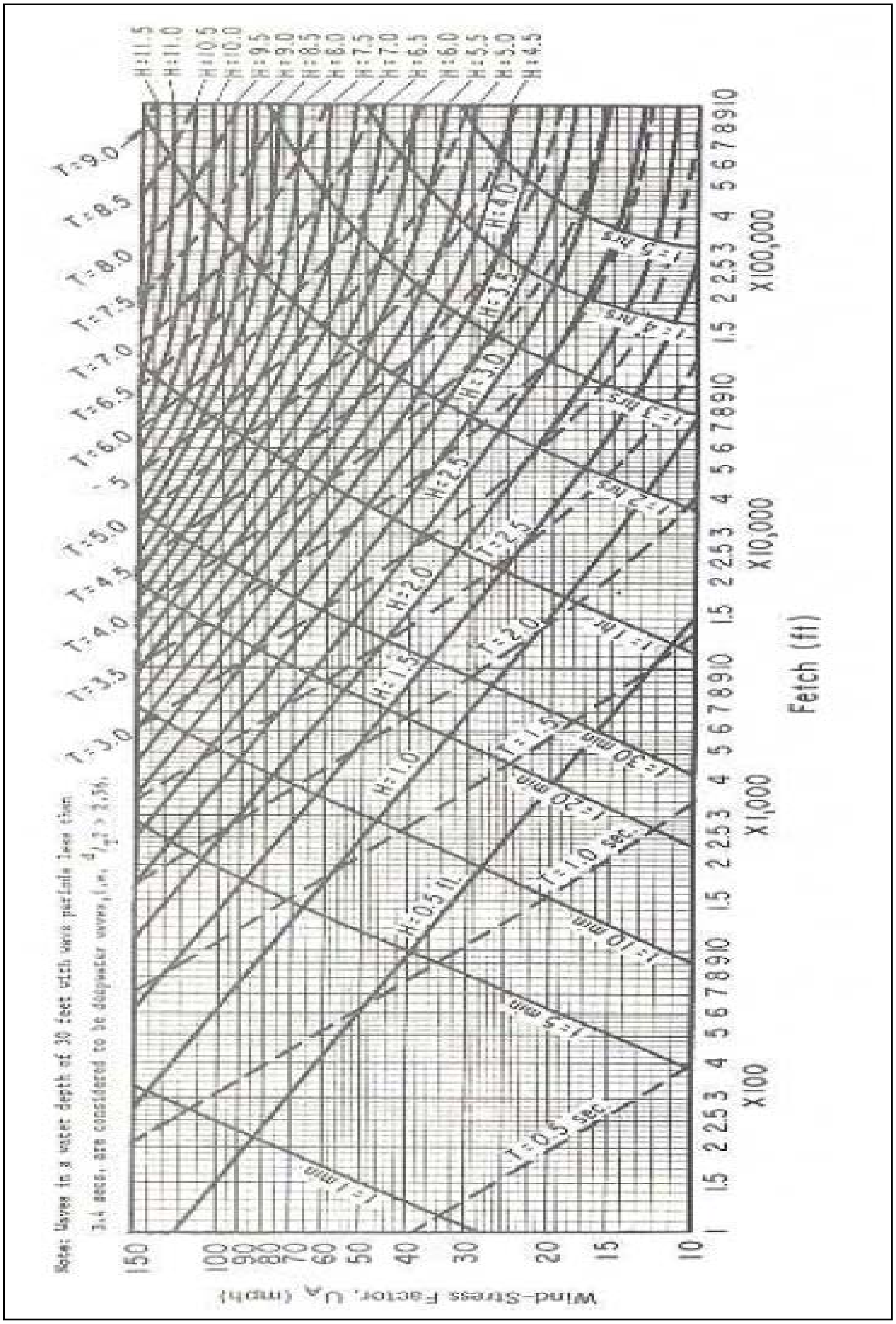
Appendix 3c: Wave Forecasting Chart for water depths of 15 feet (USA Shore Protection Manual)



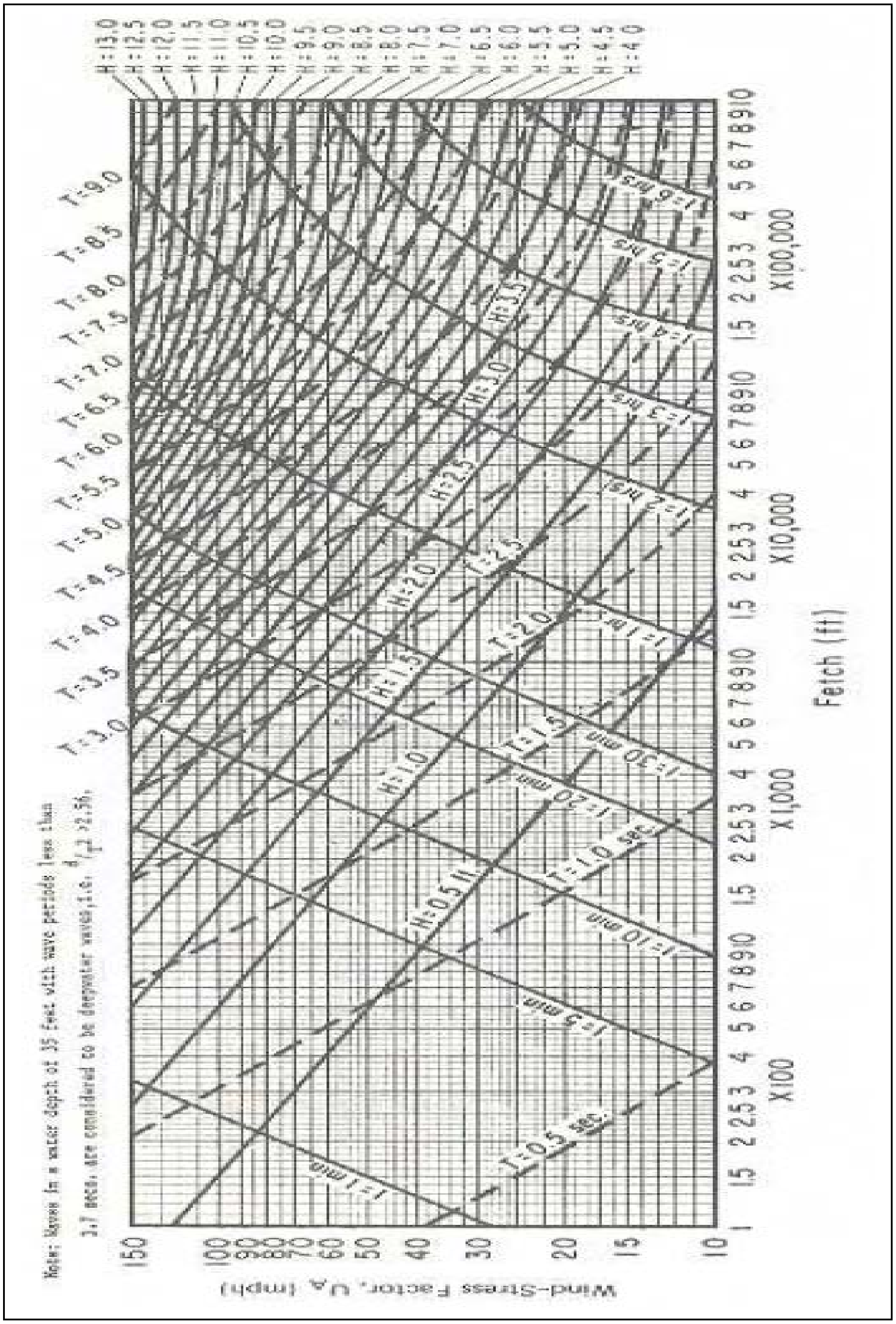

Appendix 3d: Wave Forecasting Chart for water depths of 20 feet (USA Shore Protection Manual)



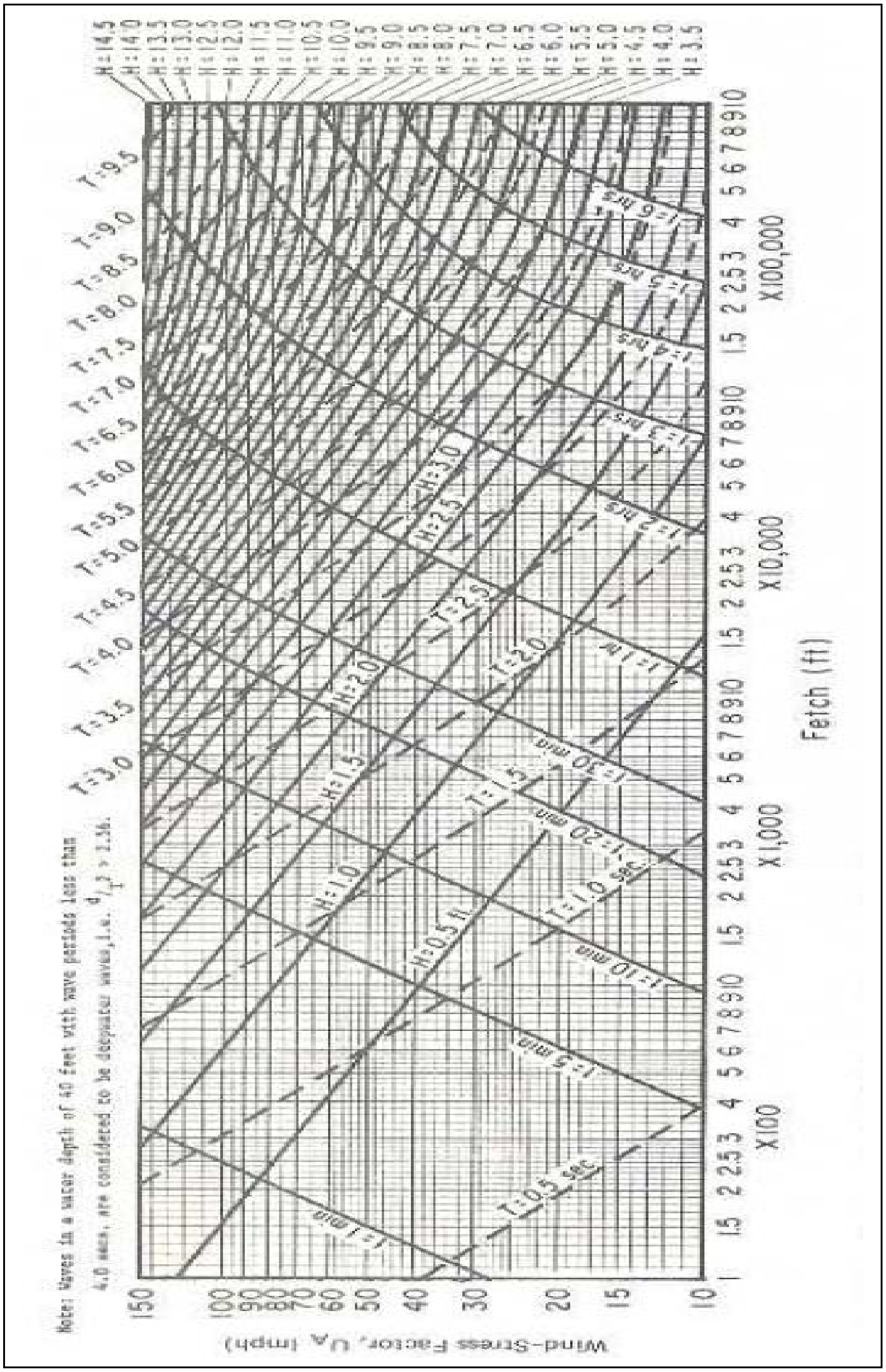

Appendix 3e: Wave Forecasting Chart for water depths of 25 feet (USA Shore Protection Manual)



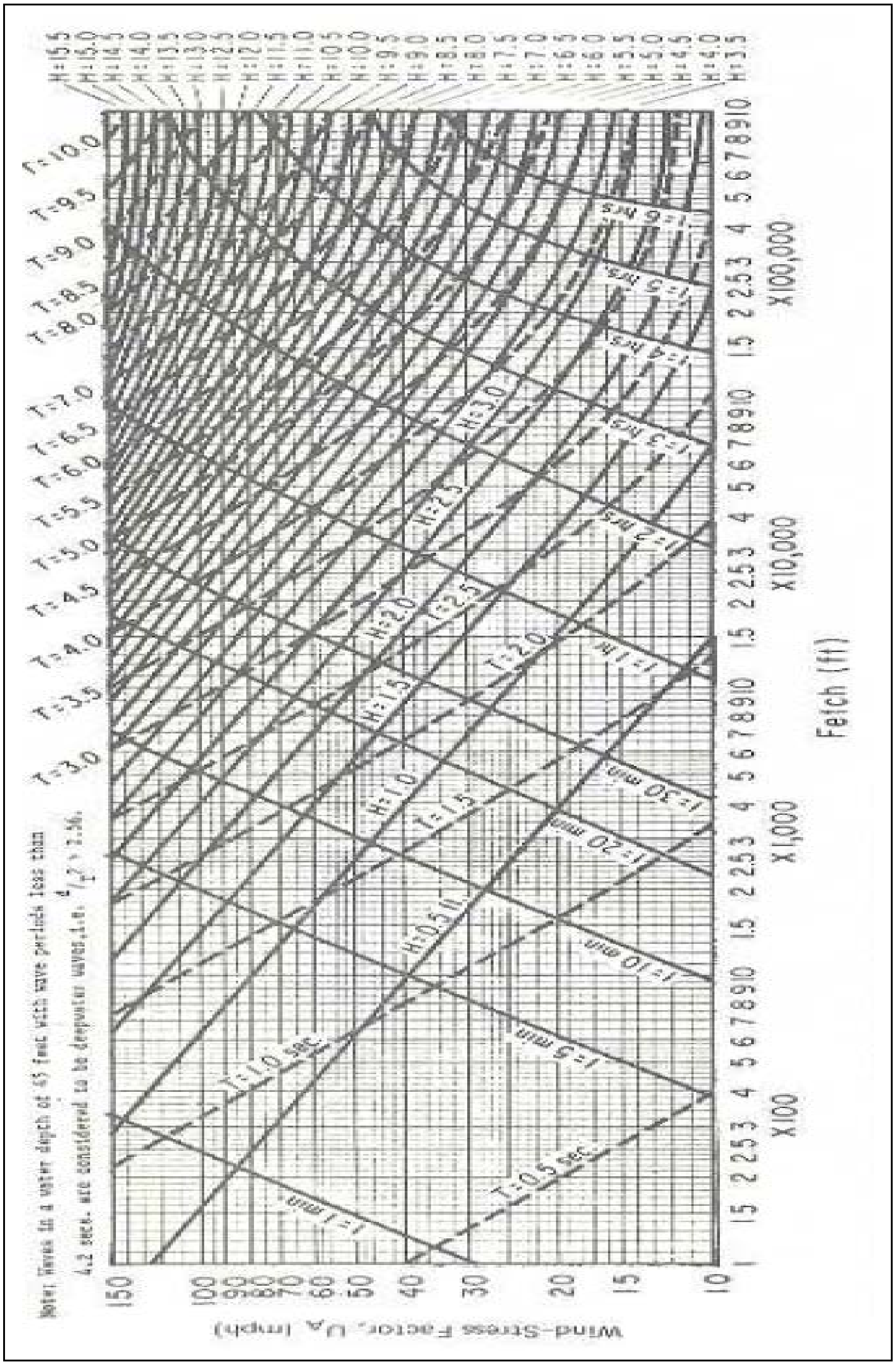

Appendix 3d: Wave Forecasting Chart for water depths of 30 feet (USA Shore Protection Manual)




Appendix 3e: Wave Forecasting Chart for water depths of 35 feet (USA Shore Protection Manual)




Appendix 3f: Wave Forecasting Chart for water depths of 40 feet (USA Shore Protection Manual)




Appendix 3g: Wave Forecasting Chart for water depths of 50 feet (USA Shore Protection Manual)