DETERMINATION OF MAIN DESIGN PARAMETERS FOR USE IN COASTAL DEFENCE

Dominic Hames
School of Computing and Technology, University of East London
d.p.hames@uel.ac.uk

Abstract : Although the fitting of univariate distributions to environmental parameters are a relatively straightforward task, the fitting of a joint or multivariate distribution, where the different variables have a degree of correlation is more of a problem. This paper considers the fitting of a joint distribution to the two main parameters considered in coastal defence, namely waves and high sea levels. The resultant range of wave height and high sea level combinations is explained, together with how this range or 'curve' of values is interpreted and used in coastal defence.

1. Introduction:

A coastal defence structure, like most structures in the field of civil engineering, cannot be designed to withstand any force of nature that is thrown at it. Coastal defence structures are therefore built to a design standard, typically defined as (for example) the 50 year return period event. This means that on average, the structure built should be able to resist the forces of nature for an average period of 50 years, but would on average be expected to fail once every 50 years. More stringent design standards are usually applied to more 'valuable' property. Central London for example is protected to a design standard of 1000 years.

When designing a coastal defence structure, the two main parameters that need to be considered are the damage to the structure caused by wave impact, and the water overtopping the structure caused by a combination of the waves and the height of the sea level. Although the determination of the marginal wave height and the sea level matching the required design standard is a relatively straightforward task; by fitting a univariate distribution to recorded or hindcast data sets, wave heights and sea levels are linked. The combined or joint probability of the two variables therefore needs to be considered.

For uncorrelated variables, this is a relatively straightforward task – multiplying the probability of one event by the other to give the required probability. However, as wave heights are caused by the wind, which also causes a rise in sea levels above the predicted tide, this gives a degree of correlation between wave heights and sea levels. This is exacerbated by shallow water depths, which is almost certain at a coastal defence structure, where wave heights can be reduced due to wave breaking.

2. Design standards in Coastal Defence:

Damage to a structure is mainly as a result of the height of the wave that impacts the structure. With coastal defence structures built in the nearshore zone, the largest wave heights approaching a structure are usually restricted in height by the depth of water. Wave heights too large for the depth of water carrying them will break, and reform after breaking at a smaller wave height. This causes the white water effect seen on breaking waves at the shoreline. Obviously the higher the sea level, the greater the depth of water that the wave travels in. This means that waves are less likely to break, or break to a higher level at higher sea levels.

Overtopping of a structure is caused by wave heights running up the structure face, and over the crest of the structure. Generally the larger the wave height, the more overtopping that occurs. Also, the less distance that a wave height has to travel up a structure face, the more overtopping that occurs. This means that the greater the wave height or the sea level, the greater the overtopping.

As it is impractical and uneconomic to design and build coastal defence structures so that no damage or flooding occurs, an allowable amount of damage and overtopping is allowed for. This is known as the design standard. This design standard, usually of 50 or 100 years, limits damage and overtopping from storms to a level that will be reached or exceeded on average once during a period matching the design standard. Storms in excess of this design standard would be expected to result in flooding or damage in excess of that designed for.

3. Extreme Value Estimates:

In determining the wave height and sea level in the design of a coastal defence structure, the univariate, or marginal distributions of each of these variables needs to be considered. Standard distributions can therefore be fitted to these data sets, with results extrapolated to more extreme events.

With the increase in freely available digitally recorded sea level data sets around the world (particularly the UK), extreme estimates of sea levels can be determined with a high level of confidence. However, few records of recorded wave heights exist, and wave height records usually have to be determined based on hindcasting of wind records, or wave transformation modelling of offshore modelled or recorded data. Wind records can be obtained from (for example) records held at airports, however, these are affected by nearshore land forms, therefore wind records from offshore Met. Office locations are more appropriate. These can then be used to estimate wave records at a site by applying an appropriate spectrum to the wind records such as a JONSWAP for deep water or TMA for shallow water spectrum, see for example (Bouws et al, 1985). These spectra were specifically developed to describe the characteristics of irregular seas from wind records based on fetch limited seas. Met. Office modelled wave data can also be transformed nearshore using a wave transformation model such as REFDIF, (Kirby et al, 1994).

The recent establishment of the WaveNet program of recorded wave data around the English and Welsh coastlines, (CEFAS, 2007), will in the future greatly assist in the prediction of wave heights, although these are for offshore locations, and transformation modelling would still be required for nearshore locations.

The extrapolation of (high) sea level data for prediction of extreme events is demonstrated in Figure 1 for Holyhead in North Wales, where over 30 years of recorded data has been analysed. In this figure, a Generalised Pareto Distribution (GPD) has been fitted to the upper 2.5% of high sea levels (533 records). This follows the methods described by (Davison and Smith, 1990), where the cumulative distribution function of the high sea level records is given by equation 3.1. The chosen threshold (in this case the upper 2.5% of data) is based on numerical techniques which gives the most confidence to the fit to the data, and therefore the extrapolation to extreme values.

$$F(X) = \varepsilon\left\{1-\left(1-\frac{k(X-\zeta)}{\theta}\right)^{1/k}\right\}+(1-\varepsilon) \qquad (3.1)$$

where:

X	=	sample being considered
k	=	shape parameter
ζ	=	location parameter
θ	=	scale parameter
$1-\varepsilon$	=	threshold

The return period is given by the Weibull plotting position formula, given by:

$$\text{Return Period} = \frac{n+1}{706r} \qquad (3.2)$$

where:

n	=	number of events
r	=	rank of event
706	=	number of high tide events per year

Extrapolation of wave heights follows the same procedure, and Figure 2 demonstrates this for a location in Southern England where equation 3.1 has been fitted to the upper 5% of wave heights (429 records).

4. Joint Probability of Wave Heights and Sea Levels:

Although as demonstrated in Section 3 the marginal distribution of wave heights and high sea levels are relatively easy to define, their joint distribution is more problematical as the joint probability of two variables that have a degree of correlation cannot be defined. Design parameters also consist of a range, or curve of values not a single high sea level, wave height combination. This can be appreciated by considering the definition of a 50 year return period event. This is defined as the level which is reached or exceeded on average once every 50 years. Therefore, for the marginal extreme high sea levels given in Figure 1, this is given by 3.71m@OD. However, for the combination of high sea levels and wave heights (slwh), the 50 year return period event has to reach or exceed this slwh combination for both parameters. Considering the lowest high sea level, then this includes all wave height records, and the wave height will equal the corresponding marginal return period of the wave height. However, if you consider a higher high sea level, then you will only consider wave heights that occur at this high sea level or greater. For higher high sea levels, less and less wave heights are considered. This means that as the high sea level increases, the corresponding wave height in the slwh combination must reduce to give the same return period.

This is demonstrated for a location on the Cumbrian coastline by Figure 3, which has been adapted from (Hames, 2007). The joint probability curves have been determined in this case by assuming that the upper tails of the wave height and sea level data sets follow a mixture of two bivariate normal probability distributions, with data below these thresholds assumed to follow the general distribution of the input data. This is based on proposals by (Coles and Tawn, 1990). This figure, which consists of 13.2 years of modelled data, shows the 5 and 100 year joint probability curves superimposed on the modelled slwh data combinations. Considering the 5 year joint probability curve only, with 13.2 years of data 2-3 slwh combinations would be expected to exceed each point on the joint probability curve. This is shown on Figure 3 for the slwh combination of (4.6m@OD, 3.59m), which indicates 3 larger combinations as shown, which corresponds to the 2-3 that would be expected statistically. However, significantly more or less combinations could occur. This is indicated for slwh combinations of (4.5m@OD, 3.62m), (5.1m@OD, 3.36m) and (5.8m@OD, 1.94m) which are exceeded 2, 7 and 2 times respectively. It should also be noted that within any time series data set, certain combinations would be expected to have return periods far in excess of the length of the data series. This is clearly indicated in Figure 3 where 2 slwh combinations are noted to have return periods in excess of the 100 year joint probability curve, one of which has an estimated return period in excess of 200 years.

5. Application of Joint Probability Curves to Design:

With the joint probability curve defined, these can now be used to determine design conditions. As has already been stated in Section 2, two factors need to be considered in design. These are damage to the structure and maximum allowable overtopping.

The maximum damage to a structure will normally occur when the maximum wave height for the joint event considered impacts the structure, (Hames, 2007). However, this will not be the maximum wave height from the joint probability curve as wave heights are limited by depth. Figure 4 demonstrates this for a structure toe at 2m@OD, using the joint probability curves from Figure 3. Assuming a simplistic maximum wave height 78% of the depth (see Southgate, 1995), this figure shows the maximum wave height relative to sea level that can impact the structure. Where this depth limited condition meets the joint probability curve gives the maximum wave height that can impact the structure under design conditions.

The maximum level of overtopping is determined by considering levels of overtopping for each slwh combination. The largest overtopping determined gives the maximum that can overtop the structure under design conditions. This is also shown on Figure 4, where the overtopping is determined using the method of (Owen, 1981). Typically maximum damage to a structure and maximum overtopping occur at different slwh combinations, with overtopping occurring at a slwh combination associated with a higher sea level. Different design parameters therefore have to be considered when looking at damage and overtopping.

6. Conclusions:

This paper looks at the fitting of a joint distribution to the two main parameters considered in coastal defence, namely waves and high sea levels. Using published results in (Hames, 2007), it is shown how design parameters consist of a curve of high sea level and wave height conditions, where any one of these could result in the maximum damage to a structure, or the worst case of overtopping. With wave heights limited by depth, this paper indicates how to determine the largest wave height that impacts the structure, and which one of the high sea level, wave height combinations gives the worst case overtopping.

It is indicated that typically the worst case for damage and overtopping is not as a result of the same parameters, and different parameters have to be considered for each design condition.

5. References:

Bouws E., Gunther H., Rosenthal W., Vincent C., "Similarity of the Wind Wave Spectrum in Finite Depth Water", *Journal of Geophysical Research,* 1985, Volume 90(c1), pp. 975-986.

CEFAS, Centre for Environment, Fisheries and Aquaculture Science UK Wave Network (WaveNet), [internet], Available from : http://www.cefas.co.uk/wavenet/. [cited 24 May 2007].

Coles S.G., Tawn J.A., "Statistics of coastal flood prevention", *Philosophical Transactions of the Royal Society of London Series A,* 1990, Volume 332, pp. 457-476.

Davison A.C., Smith, R.L., "Models for exceedances over high thresholds", *Journal of the Royal Statistical Society Series B,* 1990, Volume 52, Number 3, pp. 393-442.

Hames D.P., "The Joint Probability of Waves and High Sea levels in Coastal Defence", 2nd IMA International Conference on Flood Risk Assessment, University of Plymouth, 2007.

Kirby J.T., Dalrymple R.A., *Combined Refraction / Diffraction Model REFDIF-1. Version 2.5.,* Documentation and User's Manual, CACR Report 94-22, Center for Applied Coastal Research, University of Delaware, 1994, 122 pp.

Owen M.W., *Design of Seawalls allowing for Wave Overtopping*, HR Wallingford Report EX924, 1981.

Southgate H.N., "Prediction of wave breaking processes at the coastline. In: Rahman M. (Ed.)", *Advances in Fluid Mechanics*, Volume 6, Computational Mechanics Publications, Southampton, 1995.