OPTIMISING A BREAKWATER LAYOUT USING AN ITERATIVE ALGORITHM

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Due to the remote and exposed coastal location of most liquefied natural gas (LNG) exportation terminals, a breakwater is often required to reduce the wave energy at the vessel berth. Best practice and research techniques for developing the cross-section and sizing armour stone are well established however, little research exists for developing economical breakwater layouts. This paper describes the methodology behind a new piece of software which can be used to quickly develop breakwater layouts that simultaneously minimise berth downtime and capital cost. This is achieved on the premise that for a given level downtime, there exists a corresponding breakwater length. This allows the berth availability to be considered as a function of the breakwater layout, which can therefore be solved through an iterative approach. A random wave transformation model is used to transform the waves to, through and around the breakwater. The diffracted and transmitted wave energies are summed using the root mean square (RMS) approach before the 'effective wave' is used to determine whether the vessel mooring threshold is exceeded. Performing this for a time series of waves gives an indication of the percentage of downtime that will be incurred with this breakwater layout. An Iterative search is performed using Brent's algorithm to find the layout which offers the desired level of berth availability. This methodology is applied to a detached rubble-mound breakwater, though could easily be transposed to a berm, caisson or breakwater connecting to the shore. A case study comparing this approach to the results of a front end engineering design is used to demonstrate the effectiveness of this methodology in developing accurate breakwater layouts in a very short time-frame, highlighting that a simple approach can deliver accurate concept designs quickly. A list of the notation used in this paper is included on page 11.

Keywords: *Breakwater, Concept Design, Optimisation, Root-Finding, Wave Transformation, Berthing*

1. Introduction

As more countries are looking to capitalise on national gas resources through exportation, many new Liquefied Natural Gas (LNG) terminals are in development. LNG exportation terminals are often situated in locations where adequate natural protection is unavailable, therefore requiring an artificial breakwater to reduce wave conditions at the berth to within acceptable limits (figure 1). The most common type of breakwater used in an LNG terminal is the rubble mound breakwater armoured with rock or concrete armour units. Caisson breakwaters are sometimes used, although they only usually become cost effective in depths > 15m.

Figure 1 - Breakwater Protecting two berthed LNG vessels

The breakwater is primarily used to reduce wave energy transmitting through the core, diffracting around the breakwater and overtopping the breakwater. In typical circumstances, overtopping only occurs when wave conditions approach design conditions, although wave energy will be transmitted through the core and around the breakwater on a continual basis and is therefore of paramount importance when developing a layout.

Rubble mound breakwaters are flexible structures. Their design is based on the concept of tolerable damage and acceptable displacement of armour stone. Careful consideration must be given to frequent and extreme events as the former affects the wave climate behind the structure and the latter affects safety (BSI, 1999).

As waves diffract around the breakwater, they lose a significant amount of energy . However, wave energy reaching the berth may still be above acceptable limits. In a typical berthing study, a target level of downtime may be 5% which gives a berth availability of 95%. The percentage of time that the wave climate in the breakwater shadow is below acceptable limits is used as the criteria to find a suitable length of the breakwater.

1.1. Concept Design of a Breakwater

The conceptual design stage of any engineering project is arguably the most important as up to 80% of the project resources are committed and as time progresses, changes become harder and more expensive to implement (Kicinger *et al.*, 2005) (figure 2). It is essential that good decisions are made in the early design stages as these have great influence on the final design.

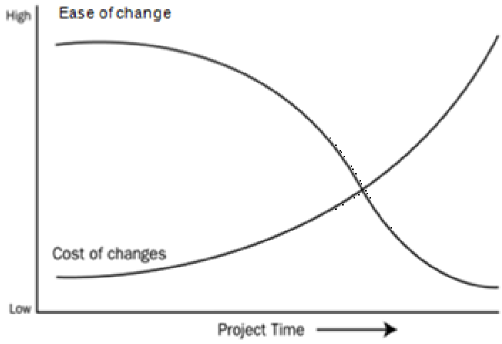

Figure 2 - Cost and Ease of Change over Time

Regarding the design of a breakwater, there are two major aspects. The first is the design of the cross-section, and the second is designing the shape of its footprint. Design of the cross-section (figure 3) is a well-established field of practice, the foundation of which involves calculating the crest height R_c which is usually designed to limit the volume of overtopping discharge; selecting the front and rear slope angles θ_{front} and θ_{rear} (usually as steep as possible to minimise material volume) and determining the crest width B which is largely determined by usage factors such as whether the pipe trestle will be built on the crest or vehicular access is required. A common approach to determining the armour size of a breakwater is to use the 'design wave' which is a single value wave height (often H_s or $H_{1/10}$) which has a low probability of exceedence during the design life (BSI, 1999; Goda, 2010). Wave period, direction, spectral energy and whether the waves are breaking is also important; longer period waves transmit more energy through the breakwater (CIRIA and CUR, 2007) which may require wider cores to reduce transmitted energy. The rock armour size is typically calculated using van der Meer's slope stability formulae for rock armour (van der Meer, 1988), or using Hudson's stability formula (Hudson, 1958) for concrete armour units such as Accropodes. The size of the armour has only a minimal effect on the berthing conditions, more important are the cross-sectional dimensions.

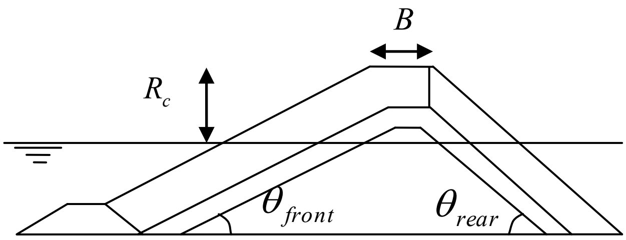

Figure 3 – Breakwater cross-section showing main variables

Initial breakwater layout concepts are often based on engineering judgement in order to obtain an

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approximate breakwater layout, length and ultimately cost. Figure 4 shows two breakwater layouts with waves diffracting around the breakwater roundheads to a specified location (the vessel berth). The breakwater on the left provides a higher level of protection as the angle of diffraction is smaller than that of the one on the right, although it will also cost more to construct. Judging the right length is a primary concern for the engineer.

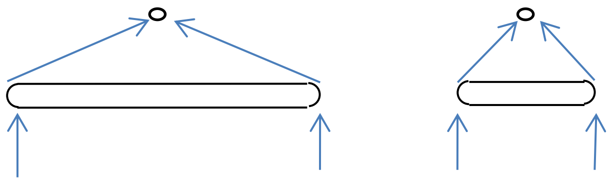

Figure 4 – Two breakwaters offering more (left) and less protection (right) to the specified point.

Numerical and physical modelling is often used during the latter design stages to optimise a design, although these techniques are not appropriate for the conceptual design stage as several concepts may be in development as these modelling techniques are expensive and time-consuming. When performing a concept layout 'desk study', common practice is to review wave tables of the proposed location and create a 'design wave' which is often used in conjunction with tools such as Goda's random wave diffraction diagrams (Goda and Takayama *et al.*, 1978). Through this approach, an indication of the length of breakwater can be calculated, although this is based on a single or few waves which summarise potentially decades of wave data and is therefore less accurate than considering the entire time series.

1.2 Aims of this Paper

This paper outlines a method for developing detached breakwater layout concepts using a simple software model. The model integrates random wave transformation and vessel mooring simulation models within a mathematical algorithm used to find the root of an equation. This allows a breakwater layout to be generated, wave conditions to be transformed to the berth simulating the effects of shoaling, refraction, diffraction and transmission where the vessel mooring threshold is tested. Performing this for a time series allows an estimation of the percentage of berth availability to be made and the breakwater length is either increased or decreased until the breakwater layout providing the desired level of berth availability is found. This method is fast and more accurate than using judgement alone and can be automated to test multiple berth locations and layouts.

> **Note:** *Root-finding in this context is a mathematical term where an iterative algorithm is used to solve f(x) = y, not the 'root' of the breakwater where the breakwater meets the shore.*

2. Wave Transformation

In most cases, wave data will not exist exactly where the engineering work is proposed; wind or wave data is often only available from offshore survey buoys. This means that the processes of wave refraction, shoaling and diffraction must be simulated to provide wave conditions at the breakwater. Refraction is the bending of water waves due to variation in celerity across the wave crest; shoaling is the increase in wave height proportional to the decrease in wave celerity occurring as the wave enters shallower water and diffraction is the bending of waves as they pass an object.

Although the random nature of sea waves was understood by some engineers in the early 20th century, it wasn't until the 1950's, that theories started to emerge. Sea waves can be analysed as an infinite spectrum of smaller waves of varying heights, frequencies and direction, most pronounced around the peak values (Goda, 2010). This is often represented as a *directional wave spectrum* when designing for waves.

2.1. Refraction

Fast and accurate wave height predictions considering the wave spectrum can be made with Goda and Suzuki's method (Goda and Suzuki, 1975) where:

$$(K_r)_{eff} = \sqrt{\frac{1}{m_0}\int_0^{\infty}\int_{-\pi}^{\pi} S(f,\theta).K_r{}^2(f).K_r{}^2(f,\theta).d\theta.df}$$

In which: m_0 is the representative value of total wave energy, $S(f,\theta) = S(f).G(\theta|f)$ and is the combined directional spectral density function and directional spreading function.

2.2. Shoaling

A nonlinear, random wave nearshore shoaling theory was published by Shuto (1974) using the Bretschneider-Mitsuyasu spectrum and has been expressed in terms of the shoaling coefficient K_s by Goda (1975) which is used in this study to simulate shoaling of random waves employing the following equation:

$$K_s = K_{si} + 0.0015\left(\frac{h}{L_0}\right)^{-2.87}\left(\frac{H_0'}{L_0}\right)^{1.27}$$

2.3. Diffraction

Penney and Price (1952) published a seminal transcript on a methodology for calculating the diffraction coefficient K_d for monochromatic waves due to interaction with a semi-infinite breakwater. In 1962, Weigel published the so-called "Weigel Diagrams" for calculating wave diffraction. Goda et al. (1978) developed random wave diffraction diagrams using the Bretschneider-Mitsuyasu spectrum. Kraus (1984) published an approximation for random wave diffraction based on Goda's method which can be calculated without integrals; this is implemented in this study:

$$K_d(\theta) = \sqrt{0.5\left[\tanh\frac{S_{max}\theta}{W} + 1\right]}$$

Where: $W = -0.000130S_{max}{}^2 + 0.27S_{max} + 5.31$

and θ is in radians.

3. Optimisation Techniques

Many optimisation routines make use of evolutionary based algorithms such as genetic algorithms (GA's). Elchahal et al. (2013) used a genetic algorithm to optimise the shape of a detached breakwater within a port. This is an interesting approach though the algorithm took over 13 hours to run and 'optimal' designs were often quite unrealistic and would likely have been redesigned by the contractor for a more cost optimal solution highlighting the importance of solid concept design.

3.1. Root-Finding Algorithms

Root-finders are a class of algorithms which iteratively estimate a value of x until $f(x) = 0$ is found. Figure 5 shows a function y=f(x) where the root can clearly be seen as y = 0. Root-finders are simple to implement and have been used successfully for millennia.

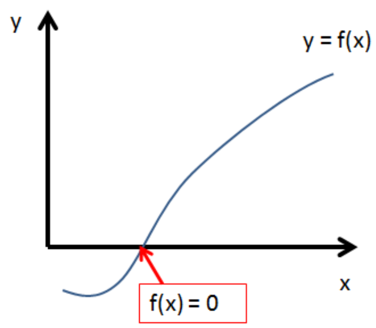

Figure 5 - Graph of *f(x) = 0*

The first root-finding algorithm is thought of to be the Secant Method ~2300BC which uses a succession of roots of secant lines to find an approximation of $f(x)$. The Secant Method only works in select circumstances which limits its generic applicability. The Newton-Rhapson Method is a Method for approximating $f(x)$ where the function is iteratively divided by its derivative until an acceptably accurate root is found. Though the algorithm is fast, the Newton-Rhapson Method occasionally fails to converge making it less suitable. A more robust method is the Bisection Method where a positive (a) and negative (b) estimate of $f(x) = 0$ is made and the error is computed. The interval between (a) and (b) is then halved iteratively until the approximation is found. Though robust, the Bisection Method is often slow to converge, especially in comparison to the Newton-Rhapson Method. Another important method is the Inverse Quadratic Interpolation algorithm. This algorithm is generally fast to converge if the current approximation is close to the actual root though slow in other circumstances. In response to these shortcomings, Brent developed Brent's Method (Brent, 1973) which implements the Bisection Method, the Secant Method and Inverse Quadratic Interpolation, selecting which is most appropriate for the current iteration. It has the robustness of the Bisect Method and the speed of the Newton-Rhapson Method making it a useful and appropriate tool for this study.

4. Methodology

In order to find the breakwater layout which offers the required level of berth protection, the model requires a bathymetric data set, a wind and wave time series and the tidal range as well as the coordinates of the desired berth location as shown in figure 6.

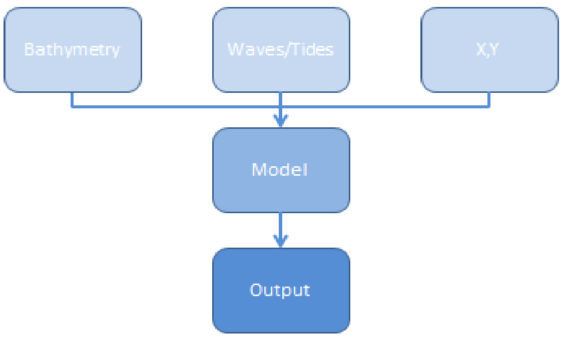

Figure 6 - High Level Flow Diagram

Figure 7 gives an overview of the software algorithm that is used to calculate the layout of the breakwater. The following sections will briefly discuss each of the processes labelled in Figure 7 from **a** to **h**.

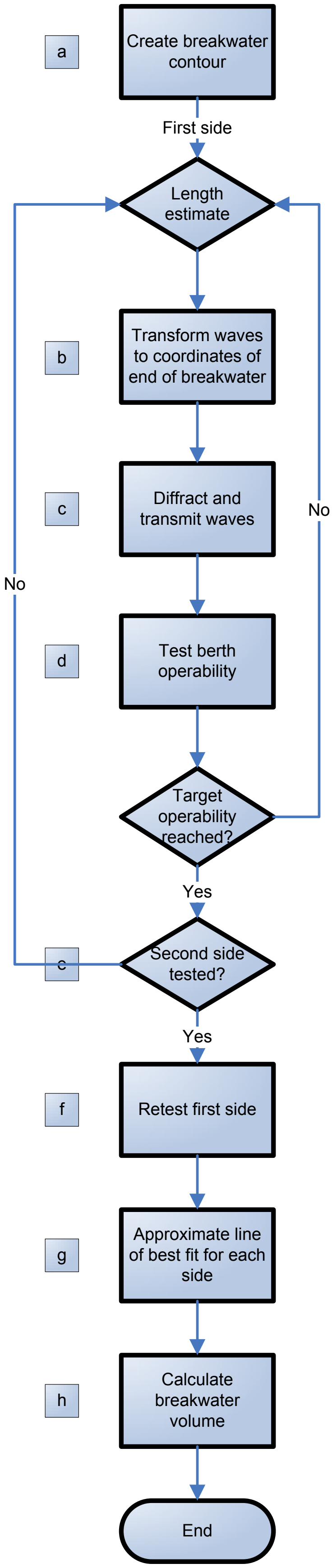

Figure 7 – Flow diagram of the breakwater layout algorithm

a. Creating the Breakwater Outline

With the berth location selected, the centre point of the breakwater found by extruding in the direction of the seabed slope θ_{seabed} (which should align with the direction of the transformed significant wave due to millennia of wave-seabed interaction). From this point, two lines are extruded at θ_{seabed}+ 90 and θ_{seabed}-90 which gives the potential breakwater layout as shown in Figure 8 where the black circle represents the berth location, the yellow line is the direction of the contours, red 'X' is the mid-point of the breakwater and the dark red lines are the potential lengths of each side of the breakwater.

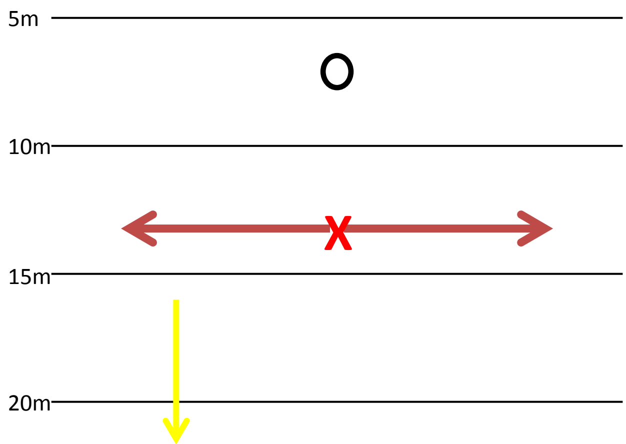

Figure 8 – Topographical view of potential breakwater layout

Now that a potential breakwater layout has been found, Brent's Algorithm can be used to find the length which corresponds to the layout which offers the desired level of berth operability.

b. Transforming the Waves to the Breakwater

The wave height at the berth is a product of the three major wave components and can be approximated as the root mean squared (RMS) of their total wave energy:

$$H_{eff} = \sqrt{{H_1}^2 + {H_2}^2 + {H_3}^2}$$

Where the value of H_1 is the refracted, shoaled and diffracted wave travelling around the LH side of the breakwater, H_2 is the refracted, shoaled and transmitted wave coming through the breakwater core and H_3 is the refracted, shoaled and diffracted wave travelling around the RH side of the breakwater as shown in figure 9.

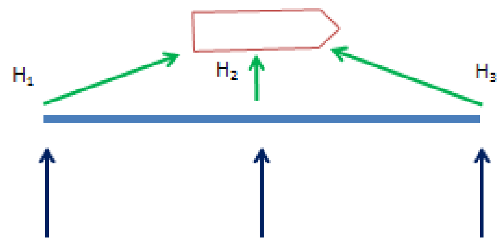

Figure 9 - Simplified Diagram of the Major Wave Energy Components

The water level is calculated from the observed tidal level dataset which is inputted into the model. Figure 10 shows sea level variations based on tidal motion. In absence of such data, approximations can be made if the highest and lowest astronomical tides are known using basic trigonometric functions to simulate the gravitational pull of the sun and moon simultaneously.

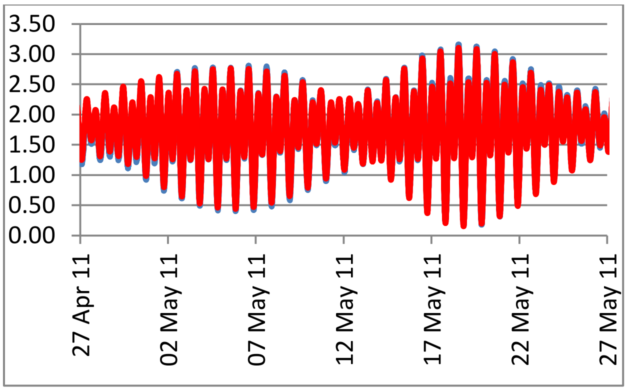

Figure 10 – Sea level variation through tidal motion

The offshore wave time series is refracted and shoaled to the mid-point of the breakwater and the transformed wave data is stored in an array. Only one breakwater side length can be found at a time using a root-finder so each side of the breakwater must be considered independently at first. As the effects of the diffracted wave emerging from the other side are unknown as a length has not been estimated for that side yet, the effective wave height equation reduces to:

$$H_{eff} = \sqrt{{H_1}^2 + {H_2}^2}.$$

For the first side, Brent's algorithm is used to select a breakwater length from which the corresponding coordinates are easily found. The offshore wave time series is transformed to the coordinates corresponding to the length which has been chosen by the algorithm, taking into account the water level variation due to tidal motion.

c. Diffract and transmit waves to the berth

The diffraction angle is calculated for each of the transformed waves from which the diffraction coefficient can be found using Kraus' Method for H_1. The following equation is used to calculate the coefficient of wave energy transmitted through and over the breakwater cross-section for H_2:

$$C_t = -0.4\frac{R_c}{H_s} + 0.64\left(\frac{B}{H_s}\right)^{-0.31}\left(1-\exp\left(-0.5\varsigma_m\right)\right) \quad : \frac{B}{H_s} < 10$$

$$C_t = -0.35\frac{R_c}{H_s} + 0.51\left(\frac{B}{H_s}\right)^{-0.65}\left(1-\exp\left(-0.41\varsigma_m\right)\right) \quad : \frac{B}{H_s} \geq 10$$

The total wave height at the berth is now computed as a product of the diffracted and transmitted wave heights as shown in figure 11.

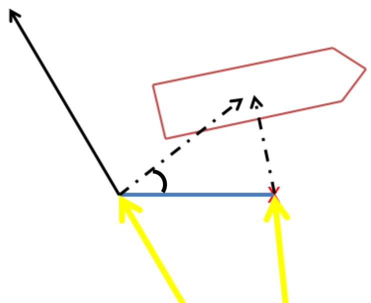

Figure 11 - Diffracted Wave Angle

d. Test berth operability

To test whether the mooring threshold of the berthed vessel has been exceeded, equations, linking the peak period Tp to the significant wave height Hs are used, a graphical representation of which is shown in figure 12:

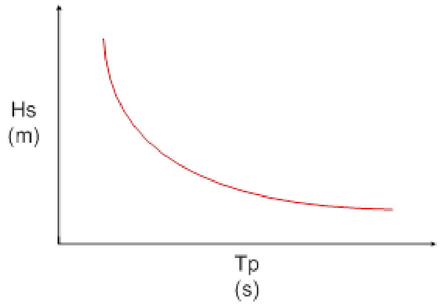

Figure 12 - Mooring Threshold Curve

These equations are based on full dynamic mooring simulations that have been conducted at HR Wallingford. Equations for 75,000m^3, 138,000m^3 and 210,000m^3 capacity vessels for waves which are head-on, quartering, and beam-on have been developed (figure 13).

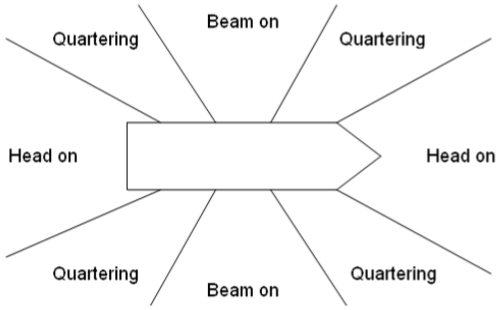

Figure 13 – Relative wave direction to the vessel

The diffracted wave is the largest contributor to H_{eff} so the direction of this component relative to the vessel is used to select the correct equation. The wave period from the current step in the time series is used in combination with H_{eff}. If the combination is below the threshold then a pass is given for that step. The operability for the berth $f(x)$ is then calculated as the percentage of waves which were given a pass. The error in $f(x)$ is used to estimate a closer value of x. This process continues until a suitably accurate value has been found for x. Generally speaking, smaller vessels are more susceptible to small period waves and large vessels are more susceptible to longer period waves as the peak frequencies are closer to their natural frequencies. Larger wave heights and longer wave periods carry more energy so there is a trade-off between these which can be approximated with the threshold curves.

e. Calculate Length of Other Side

Steps **b**-**d** are now repeated for the second side. As the length of the first side is now known, waves emerging from both sides can be considered as shown in figure 9 using the equation from section **4.b**.

f. Recalculate first side

Once the second side has been calculated, the procedure is performed on the first side once more, this time considering the waves emerging from the second side also. This process could go on for several more iterations, though it has been found that after this iteration, the percentage of length change is negligible.

g. Approximate line of best fit

In locations where waves are large, the model at this stage may produce a breakwater length that seems too large as shown in figure 14.

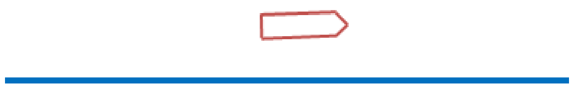

Figure 14 – Non-optimised breakwater layout

A routine has been included that reduces the length of the breakwater if this is the case. By ensuring that the line of the wave travelling from the roundhead is intercepted, the breakwater length can be reduced considerably (figure 15).

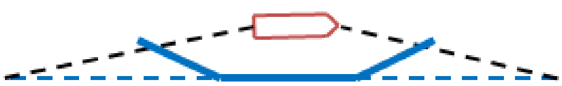

Figure 15 – Optimised breakwater layout

h. Calculate Breakwater Volume

Now that the cross-section and length have been found, the volume of each layer is easily found from which a cost estimate can be developed. In this model, costing is automated based on unit rates inputted by the engineer which provides a metric for comparison.

5. Case Study

A case study has been used to demonstrate the effectiveness of this algorithm in estimating the conceptual layout of a breakwater. Figure 16 shows a breakwater protecting two berthed LNG vessels. This layout is the product of a several month FEED study.

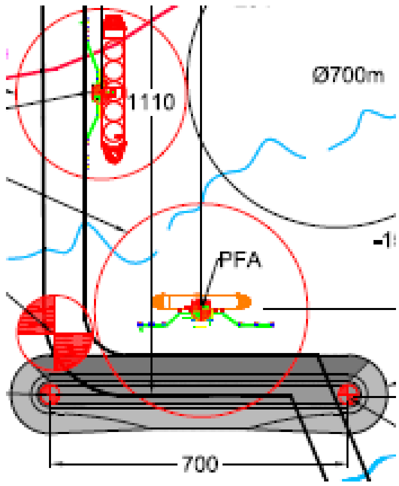

Figure 16 - FEED Design of a Breakwater Protecting Two Berthed LNG Vessels

The significant wave direction is 178°, the breakwater faces 180°, is 700m long crest centre-point to crest centre-point and the ends of the base are 755m apart. Using the same bathymetric and wave datasets, the root-finding algorithm is used to develop a concept design. The wind/wave time series has 20 years of data.

To account for the two vessels, the algorithm is run independently for a 75,000m^3 vessel close to the breakwater and a 210,000m^3 vessel in the more distant position. The design breakwater length is then worst case for each side as shown in figure 17.

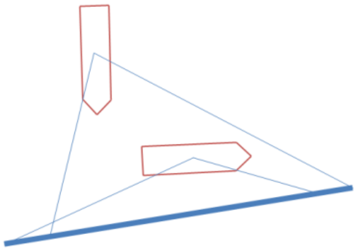

Figure 17 - Superimposition of Two Breakwater Optimisation

Overlaying the FEED study with the newly created design (figure 18) shows that a similar breakwater layout has been found. The exact percentage of difference in lengths is difficult to determine as the points from which diffraction occurs are somewhere beneath the water level making the effective length of the FEED breakwater somewhere between 700m and 750m.

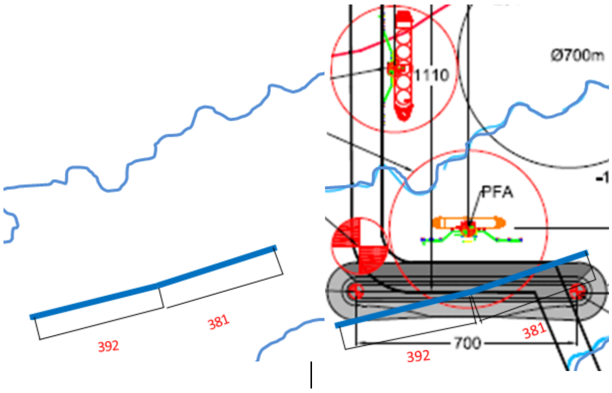

Figure 18 - Comparison of Breakwater Layouts

6. Results and Discussion

The algorithm performed well on this study. It was able to produce an answer similar to that which took several months and significant project costs to create using the conventional approach. This study took an hour to prepare the data and set up the model and less than 10 seconds to run. The length of breakwater produced was 773m in total which is within the error range of 2~10% which is acceptable considering that the FEED design has been through a full dynamic mooring assessment as well as numerical and physical modelling during a several month design period and other studies such as current and sediment modelling have also been undertaken. The breakwater did have a slightly different alignment to the one from the FEED study, though other aspects which are currently outside of the scope of this tool had an effect on the actual design. The algorithm developed a concept that was a straight breakwater as the total length was less than the threshold at which step **4.g** reduces the layout by creating a convex shape.

Brent's algorithm was capable of finding the value of x which gave $f(x) = operability$ in around 6 iterations for each side for each time. The simplicity of the root-finding concept means that it can be followed and understood by any engineer without having to delve into advanced computer methods which offers a level of comfort through transparency. Though the algorithm was able to create a similar sized layout, it does use simplified processes and relationships to ensure a fast running time. An example of this is the dynamic mooring assessment which is only able to consider the wave height, period and direction. In a full dynamic mooring simulation, currents, winds, line strengths and the full range of vessel movements would also be considered. The wave transformation model is accurate though not as accurate as a 2D or 3D SWAN model, however it is fast and computationally inexpensive to run which is important in this project. Improvements in the accuracy of the algorithm may be possible through developing a more accurate dynamic mooring assessment model though as a proof of concept and foundation for further development in this tool, this study has proved useful.

Considering that in the worst case, the algorithm was within 10% accuracy, the potential for using this at a series of locations to find the most optimal is realistic. This algorithm is capable of being used in concept studies to develop reasonably accurate layouts from which a better understanding of the likely cost of a breakwater can be obtained before committing significant sums of money to the project.

The model can be set up to test a range of directions in different depths to find which is most optimal, offering a useful search tool to help develop economic layout concepts during the initial stages of a project.

This methodology has been applied to a detached rubble mound breakwater, though could easily be implemented for caisson, concrete armour or berm breakwaters concepts or adapted for breakwaters that meet the shore.

7. Conclusion

The algorithm performed well in this study and was able to develop a similar layout to that which was developed through a several month FEED study. Its benefits are quite obvious, though such a tool should only be used to augment the judgement of an experienced designer and should only be used for producing potential concept designs. As a tool to gain an understanding of the likely configuration, size and ultimately, cost of a breakwater, this tool has proven to be successful.

References

Besley (1991). Overtopping of Seawalls, Design and Assessment Manual. Environment Agency, Bristol. Wallingford, HR Wallingford Ltd.

Brent, R. P. (1973). Chapter 4. Algorithms for Minimization without Derivatives. Englewood Cliffs, NJ, Prentice Hall.

BSI (1999). British Standard Code of Practice for Maritime Structures. Part 1: General Criteria. BS 6349: Part 1: 2000 9 (and Amendments 5488 and 5942), Series, Editor London, British Standards Institution.

CIRIA and CUR (2007). The Rock Manual. The Use of Rock in Hydraulic Engineering. Series, Editor London, CIRIA.

Elchahal, G., R. Younes, *et al.* (2013). "Optimization of Coastal Structures: Application on Detached Breakwaters in Ports." Ocean Engineering **63**(0): 35-43.

Hudson, R. Y. (1958). "Design of Quarry Stone Cover Layer for Previous Term Rubble-Mound Breakwaters " Research Report No. 2–2, Waterways Experiment Station, Coastal Engineering Research Centre, Vicksburg, MS.

Goda, Y. (1975). "Deformation of Irregular Waves Due to Depth Controlled Wave Breaking." Report for the Port and Harbour Research Institute **14**(3): 59-106.

Goda, Y., T. Takayama, *et al.* (1978). "Diffraction Diagrams for Directional Random Waves". ASCE Proceedings of the 16th Coastal Engineering Conference **1**: 628-50.

Goda, Y. (2010). Random Seas and Design of Maritime Structures. Advanced Series on Ocean Engineering - Volume 33, Series, Editor London, World Scientific Publishing Co. Pte. Ltd.

Goda, Y. and Y. Suzuki (1975). "Computation of Refraction and Diffraction of Sea Waves with Mitsuyasu's Directional Spectrum." Technical note of Port and Harbour Research Institute **230**: 45.

British Standards Institution (1999). Maritime Structures. Part 7: Guide to the design and construction of breakwaters, British Standard Institute. **BS 6349-7:1991**.

Kicinger, R., Arciszewski, T. & Jong, K. D. (2005) Evolutionary computation and structural design: A survey of the state-of-the-art. Computers & Structures, 83, 1943-1978.

Kraus, N. (1984). "Estimate of Breaking Wave Height Behind Structures." Journal of Waterway, Port, Coastal and Ocean Engineering **110**(2): 276-82.

Owen, M. W. (1980). "Design of Seawalls Allowing for Wave Overtopping." HR Wallingford, Report EX 924.

Penney, W. G. and A. T. Price (1952). "The Diffraction Theory of Sea Waves and the Shelter Afforded by Breakwaters." Philos. Trans. Roy. Soc. A **244**(882): 236-53.

Pullen, T., W. Allsop, *et al.* (2007). Eurotop — Wave Overtopping of Sea Defences and Related Structures: Assessment Manual., Series, Environmental Agency, UK Expertise Netwerk Waterkeren (NL) Kuratorium fr Forschung im Ksteningenieurwesen (DE),.

Shuto, N. (1974). "Nonlinear Long Waves in a Channel of Variable Section." Coastal Engineering in Japan **17**: 1-12.

TAW (2002). "Technical Report Wave Run-up and Wave Overtopping at Dikes." TAW, Technical Advisory Committee on Flood Defences. Author: J.W. van der Meer.

van der Meer, J. W. (1988b). Rock Slopes and Gravel Beaches under Random Wave Attack. PhD-thesis, , Delft University of Technology.

Weigel, R. L. (1962). "Diffraction of Waves around a Semi-Infinite Breakwater." Journal of Hydraulics Division, ASCE **88**(HY1): 27-44.

Notation

A	Slope coefficient for Owen's overtopping method
A_c	Armour freeboard (m)
A_c*	Dimensionless armour freeboard
B	Slope coefficient for Owen's overtopping method
B	Crest width of breakwater (m)
C_t	Transmitted wave coefficient
g	Acceleration due to gravity
h	Water depth (m)
$H_{1,2,3...}$	Wave height component (m)
H_{eff}	Effective transformed wave height (m)
$H_{0'}$	Effective offshore wave height (m)
H_s	Significant wave height (m)
K_d	Diffraction coefficient
K_r	Refraction coefficient
K_s	Shoaling coefficient (monochromatic)
K_{si}	Non-linear shoaling coefficient
L_0	Offshore wave length (m)
Q	Mean overtopping discharge (m^3/m/s)
$Q*$	Non-dimensional overtopping
R_c	Crest freeboard (m)
T_m	Mean period of wave (s)
T_p	Peak period of wave (s)
W_L	Water level (m)
ξ_m	Iribarren number

About the Author

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