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The reduction of wave overtopping by means of rubble mound revetment (at the Dawlish coastline)

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Abstract
To evaluate the performance of an additional rubble mound breakwater to the Dawlish Coastline sea wall defence, a number of 2-dimensional physical model tests were undertaken at the Plymouth University Coast Laboratory. The subsequent results of the tests are presented and analysed in this paper. The research found that the most significant influencing factors on overtopping were the significant wave height and the crest freeboard.

Introduction

This paper will investigate the current state of the Dawlish coastline sea wall and test its overtopping performance against sea level rise and a storm surge. It will then compare these values with that of a modified design containing a rubble mound revetment at the toe of the structure. The performance of the modified structure will then be compared against the existing overtopping performance by percentage reduction. The Dawlish coastline was chosen to be the basis for this paper due to its history of rail service disruption, structural damage and its more recent failures during storm events.

On the 4/5th of February 2014, the Dawlish sea wall failed when it was hit by a large storm. The Network Rail infrastructure behind the sea wall and a number of nearby residential properties were undermined. The same stretch of coastline was then hit again on the 14th of February of the same year, 4 kilometres of coastline was affected and the result was 250 m of sea wall that was significantly damaged, including 100 m of sea wall and 500 m of parapet that completely collapsed. The "damage to track and ballast (was) caused by wave overtopping" (Tony Gee, 2014a).

Fundamental parameters

The parameters described below will identify and describe the basic knowledge required to understand this paper and will provide a sound foundation for understanding coastal engineering.

Wave height

Wave height is the distance from the peak of a wave to the trough. Goda (2010) describes four different wave height definitions that are used throughout coastal engineering calculations: the highest wave (H_{max}), highest one tenth wave ($H_{1/10}$), significant or highest one third wave (H_s, $H_{1/3}$) and mean wave ($\overline{H}$). The significant wave height is used when calculating overtopping rates and run-up for physical models. It is calculated by finding the average value of the highest 3rd ($H_{1/3}$) of waves in a sample. H_{max} (the highest wave) is also used in some calculations to represent extreme conditions and find the maximum tolerable height of the structure.

Wave period and wavelength

The wave period (T) is the time it takes for two peaks (or troughs) to pass the same location. The EurOtop Manual (2016) defines various wave periods used in coastal engineering. The peak period (Tp) is the most common period used as it provides the period relating to the peak of the wave spectrum. The average period (Tm), is calculated from either the spectrum or as the Manual suggests, preferably from the wave record. Finally, the significant period ($T_{1/3}$) is the average period taken from the highest third of the waves. The relationship, as given by the EurOtop Manual, for Tp and Tm is 1.1-1.25 and states that for most cases Tp and $T_{1/3}$ are almost identical. Finding the period of a wave is very useful for coastal engineering practice as it can be used to help find the celerity (C) or speed of the wave. Celerity = wavelength/time($C = \frac{L}{T}$) . Kamphuis (2000) states that the Wavelength (L) is the distance taken for a wave to begin to repeat itself.

Wave steepness
Wave steepness is determined by dividing the wave height by the wavelength. Wave height can increase when the depth of water changes over time (i.e. when a wave travels up a beach or a breakwater). As the water travels up the slope the front of the wave begins to slow (due to friction) and the rear catches up, this forces the wave to increase in height until it becomes too steep. When a wave becomes too steep it begins to break, EurOtop 2016 illustrates the different types of wave breaking that can occur.

The steepness of a wave can be used in part to determine the origin of the wave. A steepness of 0.01 suggests a typical swell sea and a steepness of 0.04-0.06 a typical wind sea. The period becomes the main overtopping factor during a swell sea which is associated with long period waves (EurOtop, 2016).
Once a wave becomes too steep the wave beings to lose shape, deform and ultimately break. The most effective type of wave breaking would be spilling. As spilling occurs over a long distance it is able to dissipate the most energy and reflect very little. As the angle of the slope increases, the amount of reflected wave energy also increases but the wave energy dissipation is reduced. The lowest slope angle would be the most effective type of coastal protection but it is very impractical for engineers to implement due to the size required, the difficulty in constructing such a large structure at sea and the associated cost.

Probability
When designing a coastal breakwater and looking at overtopping of coastal structures it is very important to consider all the parameters detailed in the previous section. It is not only enough to understand the basics of each individual parameter but also how they can combine and interact with each other. It is especially important to look at probability when designing a coastal structure. The proposed design for the Dawlish sea wall was tested against a 100 year sea level rise with a 1:50 year storm surge. This means that it was tested against the highest sea level rise we would expect to occur in 100 years due to global warming and a storm that we would expect to happen once every 50 years.

As the wall has been designed to withstand a 50 year event it would mean that it should be able to survive any combination of parameters put to it within 50 years. These are very important factors when considered together, if you were to look at an event that was statistically going to occur once every 50 years (1:50 year event) you would have to look at a large amount of variables that would combine to make the 50 year event. It does not mean that a catastrophic event will definitely occur once every 50 years, it means that it has the same probability of occurring every day for 50 years. You would then have to start breaking the conditions down to see what combination of events lead to a 1:50 year event.
It is because of this appreciation for individual characteristic combinations that I decided to test my sea wall against both sea level rise and a storm surge to better simulate the real world.

Literature review
In this section the knowledge available to engineers in the coastal engineering field will be established. The aim of this review is to find any gaps in the current findings

and provide the reader with a good foundation of coastal engineer knowledge so that they might further understand the research.

Waves

Overtopping

What is wave overtopping?

The EurOtop Manual (2016) defines wave overtopping as the water that passes over the structures crest. This is caused when the wave run-up levels (the distance the water travels up the structure) are high. It is very common for most, if not all coastal engineered structures to allow some form of overtopping, it would simply be too costly and inefficient to design them otherwise. However, when designing a coastal structure they should not allow for much overtopping during standard daily use, they should only allow for significant overtopping during peak performance, such as throughout a severe storm. Franco *et al.* (1995) agree with the findings of the EurOtop Manual and explains that in certain circumstances of extreme storms it may be sufficient to allow large overtopping if access on top of the breakwater is restricted. He defines this as a structure's functional limit. Hughes (2011) also confirms that there are two design criteria for calculating overtopping which include normal service overtopping (i.e. daily use) and overtopping caused by extreme weather events, such as a storm. Allsop *et al*. (2005) state that costal structures will not stop overtopping, but reduce the frequency of occurrence and effects.

Why is it dangerous?

Wave overtopping is a very real danger, especially in coastal towns. It can cause damage to people, property and possessions. Franco *et al.* (1995) state that multiple factors must be taken into account when assessing the danger posed to people by wave overtopping, as even the psychology, age and clothing of the individual would affect the outcome. This helps show the complexity in calculating tolerable overtopping and relating it to safety. Allsop *et al*. (2005) catagorises the consequences of overtopping as a direct hazard of injury or death, damage to property or infrastructure (e.g. the train line on the Dawlish Coastline) and damage to defence structures. They also find that there are multiple ways of measuring the hazards caused by overtopping: mean overtopping discharge, peak overtopping volumes and velocities. Mean overtopping discharge will be the main focus for this paper as it seems to be most commonly used, converted and easily understandable. Once the water has passed over the crest of the structure it can potentially wash pedestrians away, drag vehicles into the sea or cause damage to property by flooding. Franco et al (1995) tabulate the critical overtopping discharge shows the danger posed by overtopping for the local infrastructure, the community and the structure itself. The diagram shows that damage can occur at very low discharge levels.

The response to severe overtopping events led to one of three solutions: change the land use to remove the stakeholder, accept that overtopping can occur at that location and be prepared, or increase the sea defences to reduce the risk (Allsop *et al*., 2005). Most coastal structures are built in response to a natural disaster, like a severe flood or storm, as it is very hard to convince residents and local councils to spend copious amounts of money on a structure that may only be fully utilised once every 100-200 years. The main reason that communities often vote against coastal structures or reduce them in size (and therefore design life) is purely down to aesthetics. This means that sizable portions of the UK coastline are unprotected and those that are protected must still be wary of overtopping.

Types of overtopping

The EurOtop Manual (2016) lists three main types of wave overtopping. 'Green water' overtopping, (described above in *What is wave overtopping?)* is the most common type of wave overtopping and considered the most dangerous. This is where large quantities of water pass over the top of the structure.
The second type of overtopping listed is when waves break on the structure and droplets are carried over the top of the structure. This is a fairly insignificant amount of overtopping and would not be considered to pose a risk to human life.
The third type of overtopping listed is spray being carried over the structure's crest. This spray is usually caused by high winds and is considered very insignificant. Once a wave becomes steep the wind can sometimes blow the crest of and turn it into spray, which although possibly uncomfortable for nearby pedestrians, holds no real danger.

Wind effect

Ward *et al*. (1996) studied the effects of wind on the runup and overtopping of coastal structres and found that it played a significant role in increasing overtopping, as one might expect. It also stated the difficulties in providing a reliable set of data during testing as the wind effect relies heavily on the scaling effect. However, looking at the effects of wind is not within the scope of this paper. The research undertaken by de Waal and van der Meer 1992, Weggel 1976 and van der Meer and Janssen 1994 all provide overtopping calculations that were conducted in the absence of wind.

JONSWAP spectrum

The Joint North Sea Wave Project (JONSWAP) was a coastal research project undertaken by Hasselmann *et al*. (1973) in the North Sea to help understand the effects that fetch, wave height, wave period and wind speed have on a sea state. Not only did it provide coastal engineers with some very useful formulae, it provided a random sea state spectrum given in terms of peak period and significant wave height. A JONSWAP spectrum was used in the physical testing of the Dawlish sea wall model. JONSWAP was used because an assumption was made that the waves would be perpendicular to the sea wall (as perpendicular waves have the largest energy density) and therefore cause the most overtopping. This meant that our waves would be fetch limited by the English Channel. From the research findings we were able to create a 'random' sea state for each wave period and height.

Nearshore wave actions

There are many types of wave actions that can occur near the shore that affect the amount of overtopping on a structure. These include but are not limited to reflection and shoaling.

Reflection

Wave reflection is, in general, a very important wave action. As the wave breaks on the shore or onto a structure, some of the wave energy is reflected. This reflected energy can positively or negatively interact with the incoming waves. This becomes even more important when designing vertical structures (as they have the highest refection coefficient) and when doing physical modelling in an enclosed wave flume. Goda & Suzuki, (1977) undertook research to find a technique to resolve this problem. In an enclosed wave flume there is nowhere for the reflected energy to dissipate to. In testing this means that the energy from the first wave will be reflected and re-reflected hundreds of time within the tank and will therefore, negatively impact the validity of any tests unless it can be identified and recorded.

Shoaling
Shoaling is the process by which deep water waves have an increased wave height in shallow water. Dave Simmonds (2017) states that the celerity (speed) of a shallow water wave is determined by depth. This means that a fast, deep sea wave will slow as it reaches more shallow water conditions. It does this because the energy becomes concentrated and therefore increases the wave height.

Environmental factors

Tides
The water level at any given location is constantly altering due to tidal movements caused by the moon's orbit of the earth. This is an important factor when considering the design of a coastal structure as the highest astronomical tide (HAT) must be taken into account to ensure the structure has sufficient freeboard (distance above the still water level) and therefore is not drowned. There are two different types of tides that can occur, neap tides (low tide) and spring tides (high tide). Occurring twice every month a spring tide is caused by the alignment of the sun and the moon for the strongest gravitational pull, this high tide is what engineers use to determine the highest astronomical tide (Met Office, 2018).

Erosion
The coastline is one of the world's most dynamic environments, it is in a constant state of flux and as far as we can tell, will never stop changing. The constant movement of water on and off shore can be both detrimental and beneficial, it can remove sand and materials in a process called erosion or add material through cross-shore sediment transport called accretion. Erosion is the more important of the two processes as it is the only detrimental process. Bird (1985) claims that 70% of sandy beaches around the globe are receeding due to erosion. It is because of this that erosion is one of the main reasons for the creation of coastal structures.

Global warming
You cannot mention erosion without also linking and including global warming and sea level rises. "The resulting inundation from rising seas will heavily impact low-lying areas: at least 100 million persons live within one meter of mean sea level and are at increased risk in the coming decades." (Zhang, Douglas, & Leatherman, 2004). Due to the increase in global warming and the subsequent sea level rise it increases the rate of erosion considerably, making it all the more important to properly protect our shores with coastal structures. To further increase the design life of a coastal structure the expected sea level rise should be taken into account for the lifespan of the structure.

Coastal structures

Breakwaters

Goda (2010) defines two different types of breakwaters, vertical and rubble mound breakwaters. A rubble mound breakwater is used to dissipate the wave energy by forcing the waves to break on their rough sloped surfaces. They rely on multiple layers of decreasing rubble size with decreasing permeability, they are most commonly made up of two or three layers. The outer layer being 2 sets of large rock armour used to take most of the impact. The second layer is called the filter layer and should be less permeable than the rock armour and is used to minimise the flow at which the water passes through the structure. The filter layer also acts as wash out protection for the 3rd layer called the core material. The core material is usually made of quarry run (cheap small waste material) which provides the shape of the structure. There are many uses for rubble mound breakwaters, but they are mainly used for beach protection.

A vertical breakwater, instead of dissipating the energy, reflects the energy from the incident wave (Takahashi, 2002). As the name suggests it has vertical sides on both edges and is more commonly used to protect an area that may require boats to dock, like a port. The vertical breakwater is more effective at reducing the energy behind the structure as it doesn't allow any water to pass through. When designing a port and using a vertical breakwater it is important to calculate the size of the port properly. If the vertical breakwater is designed incorrectly it could reflect substantial amounts of water internally and create a sea state inside the port.

Seawalls

Thomas and Hall (1992) outline the function of a seawall as a structure designed to protect against erosion and ease flooding in coastal areas. There are many different types of seawalls and many variations of each. They are similar to a vertical breakwater but are often curved to reflect some of the wave energy back into the sea. The concave shape of the structure can also help prevent and reduce overtopping.

Failure methods

It is not sufficient to design a structure that can withstand the force exerted by the sea on a daily basis. It is also necessary to understand how a structure will fail towards the end of its lifespan. This information helps us to better prepare for structural failure and to enable us to increase the lifespan of the structure. Burcharth (1993) created an illustration of how a typical armour layer would fail, it shows how the force of the water can rotate and displace specific boulders. Once a single boulder has been displaced it then reduces the stability and efficiency of the entire structure as more boulders are likely to fail due to the gap created by the original boulder. This helps engineers understand the importance of using angular boulders that provide natural interlocking abilities or increased weight of the unit so that gravity has a stronger effect. It is also possible to use alternate armour units, each of those shown have different properties, associated costs and uses.

Roughness

The roughness, *r*, of the rock armour can significantly influence the run-up and subsequent wave overtopping. Both de Waal and Van der Meer (1992) and the Shore Protection Manual (1984) investigate the influence that the roughness has on wave overtopping. . The Shore protection manual does it for regular waves whilst the de Waal and Van der Meer, does it for irregular waves. (A regular wave being a perfect sinusoidal wave and an irregular wave being a more accurate representation of a sea state, as tested in this paper). De Waal and Van der Meer's updated a version of the Shore Protection Manual's table 7-2. It shows the influence factor that the roughness has on overtopping.

Physical modelling

Physical models are used in coastal engineering to better understand the effects the sea has on a structure. They allow engineers to examine coastal phenomena beyond the ability of analytics (Hughes S. A., 1993). Unlike mathematical/numerical models which require the engineer to simplify the calculations they can more accurately represent how the sea interacts with the structure.

Overtopping performance

The Overtopping performance of different armour units for rubble mound breakwaters (Bruce *et al*., 2008) is a very useful paper for coastal engineers as it tests 13 types and configurations of rock armour in 179 tests, to determine the impact they have on overtopping. They tested the armour systems using a slope of 1:1.5 and 1:2 and saw a reduction in overtopping of 20% and 10% for rock and cube armour respectively. They also found that the period of the wave is proportional to the wave overtopping on a structure, the larger the period the more overtopping occurs. The most significant finding from the paper and indeed its purpose, was the overtopping performance of different armour types, Bruce *et al*, (2008) found that the most effective armour type for reducing wave overtopping was a dual layer Icelandic Berm breakwater, followed closely by a dual layer of Tetrapod or a dual layer of rough rock (used in this paper).

Scale effect

When working with physical models, one must first choose a scale to reduce the prototype dimensions down to model scale. This can be done by conforming to Froudes or Reynolds scaling parameters. Scale affects stem from the reduction in size of the prototype to model structure. As the size is decreased the interactions between the structure and the water begin to lose accuracy. EurOtop (2016) states that for a scale model to be accurate it must fulfil Froude's and Reynolds' law simultaneously. It also states that you cannot fulfil both laws on a scale model, so there will always be some error. Froude's law is more commonly used to scale a model as it contains gravity, pressure and inertial forces which are all paramount for wave motion. Reynolds' law is much less common in coastal engineering because it deals solely with viscosity.

Laboratory effect

Laboratory effects arise from the failure to provide geometric similitude between the prototype and the model. Geometric similitude is the correct scaling of all aspects from the prototype to the model. It can also arise from incorrect wave and current scaling, which, if not closely monitored can ruin the validity of the study (EurOtop, 2016). It is very important to ensure that both the model and the conditions are as similar to the prototype as possible to ensure accuracy and validity.

Design overtopping discharge

Overtopping discharge is the average amount of overtopping volume for a 1 m section, it can be displayed in either l/s per m or m^3/s per m. In this paper the results will primarily be display in m^3/s/m but for ease of comparison some discharges have been converted to l/s/m. Although the measurements are taken per second, due to the nature and irregularity of waves and overtopping there is no constant discharge, it is merely converted to these units for ease of understanding (EurOtop, 2016).
Pre-1992 coastal engineers designed seawalls to allow for 2% of waves to overtop the structure during a 'design storm'. This 'design storm' had set wave conditions and a water level which meant that many designs would be over designed or under designed as the sea level and conditions are very site specific. The Hydraulics Research Station decided that this method was no longer viable due to the associated cost with overdesigning a structure and the potential catastrophes that could occur from under designing. The new method would be to calculate the expected overtopping discharge for the specific site as the most important test of a seawall is the quantity and frequency of wave overtopping (Owen 1982).

Materials and methods

This section will look at how the experiment was undertaken and what equipment was required to run the experiment.

Materials and equipment

- 35 m wave flume
- Wave generator
- 1:25 scale model of a section of the Dawlish sea wall
- Rubble stockpile
- 8 wave gauges
- 5 pressure sensors
- HD video camera
- Collection tank

Rubble size calculation

Bradbury *et al*. (1988) give three different methods for the design of rock armoured structures. The Hudson formulae referenced from the Shore Protection Manual Vol II (1984), the method detailed in the CIRCA report and van der Meer's equation.
Hudson's method was chosen to be the best estimate for preliminary design and therefore used in this paper. It was chosen over van der Meer's equation as this method relies on assumptions made by the engineer, which I am unable to confidently make, due to my inexperience. The CIRCA method is only applicable to structures with nearly impermeable cores.
Although Hudson's formulae was chosen it still has several limitations; these are stated in both the Shore Protection Manual and briefly by Bradbury *et al*. (1988). The limitations are:
the original equation was derived from small scale model test with regular waves;
the period of the wave is not taken into account; and
only non-overtopping structures were tested in the research.
The calculations shown below are for the initial estimate of rubble mound size and mass. An estimate was all that was deemed necessary at this point as ultimately the materials available would determine the final dimensions.

Table 1: Hudson formula input conditions

H (wave height)	0.16m
ρ_s (density of stone/rock)	2300Kg/m^3
ρ_w (density of water)	1025kg/m^3
$cot\alpha$ (slope angle)	2
K_d (stability coefficient)	3.5 (Rough, angular, random)

$$M_{50} = \frac{\rho_s H^3}{K_d \left(\frac{\rho_s}{\rho_w} - 1\right)^3 cot\alpha} \quad (1)$$

$$M_{50} = \frac{2300 \times 0.16^3}{3.5(\frac{2300}{1025} - 1)^3 \times 2} \quad (2)$$

$$M_{50} = 0.699\ kg \quad (3)$$

$$D_{50} = (\frac{M_{50}}{\rho_s})^{\frac{1}{3}} \quad (4)$$

$$D_{50} = (\frac{0.699}{2300})^{\frac{1}{3}} \quad (5)$$

$$D_{50} = 0.067\ m = 67\ mm \quad (6)$$

Laboratory test method

The Plymouth Coast Lab 35 m wave flume was the venue for testing the physical model. The wave flume is a glass sided tank supported by a steel frame, it is 35 m in length, and 0.6 m wide. The tank uses a wave generator that can work with a minimum water depth of 0.3 m and can produce waves of up to 0.7m in height. The tank has a wave absorber at the end of the tank, however, due to the position of the breakwater this was not used.
A JONSWAP spectrum was automatically generated by the wave generation software in the laboratory and used to model the irregular waves once the tank parameters were established. The values for the peak period and significant wave height were taken as a range of values representative of the Dawlish sea conditions taken from the Channel Coastal Observatory (2017), see Table 3 for wave conditions. It was decided that 200 waves would be sent at the wall to ensure enough data was collected.

Table 2: Wave conditions

Wave Heights	1.5	2	2.5	3
Periods	7	11	15	

The dimensions for a 1:25 scale model of the Dawlish sea wall was calculated from Tony Gee's design sketches (2014b). A model of this section of the Dawlish sea wall

was then constructed out of plywood and then coated in paint and varnished. The model was hollow and therefore needed to be weighted at the base with 50 kg to ensure that no movement would take place from the wave impact. Once the model was ready it was installed in the wave tank and sealed in place to ensure that no water could pass the structure other than over the crest.
Once the model was in place, a large collection tank (measuring 0.6 m x 0.6 m x 0.4 m) was installed directly behind the wall and sealed in place to ensure water tightness. The tank was also weighted with 50 kg to ensure no movement during wave reflection within the collection tank. The best method for measuring the volume of water was discussed and due to the equipment available it was decided that a wave gauge would be modified to provide voltage readings in the tank. The wave gauge had to be calibrated by placing a known volume of water into the collection tank and measuring the voltage output. This was repeated until the tank was full. This was then used to convert the voltage readings from the experiment into overtopping volumes.

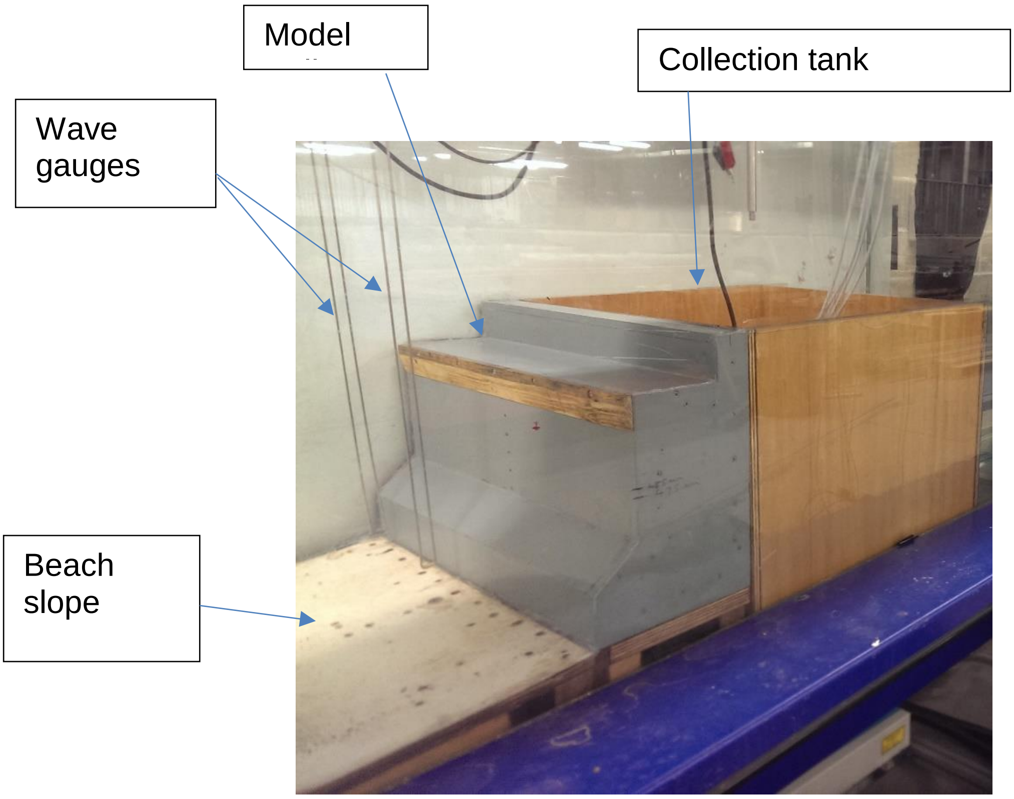

Figure 1: Model setup

The remaining 7 wave gauges were then placed methodically throughout the tank. The first wave gauge was set 1 m from the wave generator paddle, this wave gauge was used to measure the initial conditions of the waves as they were generated. 3 more wave gauges were set up at the toe of the beach slope. These wave gauges were used to measure the conditions closer to the wall and could be used to record the reflected energy and correct the data. Two more wave gauges were placed in parallel in front of the model to record the wave conditions just prior to interacting with the wall. Two were used to increase the validity and reliability of the recorded wave conditions. A High Definition (HD) video camera was set up to record the

waves breaking on the structure and to provide a visual of the rubble mounds reaction. The tank was manually filled with a water tap to the required height.
The initial test was undertaken on the current sea wall design, (no rubble mound revetment) to provide a set of baseline results (Figure 10). This would show how the current wall would act under the increased wave loading of the 1:50 year storm and the 100 year sea level rise obtained from UK Climate Projections (Met Office, 2017). A rubble mound revetment was then installed, as seen in Figure 11 and the same tests and conditions were run. A chicken wire cover was added to the rubble mound to ensure that the rubble didn't move, ensuring each test was under the exact same condition.

Figure 2: Modified sea wall model

Assumptions and limitations
Due to the complex interaction of water and structure multiple assumptions had to be made to make the calculations and subsequent testing possible.
These assumptions were necessary for the experiment to be undertaken but they did come with limitations. The first assumption was that there was no reflection within the tank, from the data given by the 3 wave gauges at the toe of the beach we know that that was not true. However, to include reflection in the calculations would significantly increase the time taken to process the data making it unfeasible for this study.

The second assumption made was a constant water depth. However, every wave that overtopped the structure would have reduced the water depth by a small amount. Due to the tank set up it was impossible to exactly replace this volume of water after each test and it was therefore ignored. The fluid was assumed to be of constant temperature, incompressible and of a fixed density of 1025 kg/m^3. In reality the temperature of the fluid would have changed, and the density of the fluid would not be exactly 1025 kg/m^3, fluids also become compressible under high pressure like the pressure exerted on impact. However, these calculations are very complex and advanced and therefore could not be undertaken as part of this study.
Hann, M (2018) stated that rubble mound structures need turbulent flow through the armour layer to ensure that there is no scale effect. As it was impossible to conserve

the Reynolds number for the armour layer and the limitations of the wave flume meant a large scale factor was used, the viscous forces may have been greater in the model than they should be, resulting in a possible scale effect.
Due to the location of the laboratory, tests were undertaken in a short time-span. If the test were to be undertaken again a minimum of 3 weeks would be required. This would allow for a sample of tests to be repeated 4-5 times to get a more accurate average. It would also make it possible to test the model at a greater variety of depths and periods, providing a larger data set to analyse. Intermediate periods of 9s and 13s would be an interesting addition to the data as it would allow us to see if the trends altered in these intermediate periods.

Results and discussion

Baseline

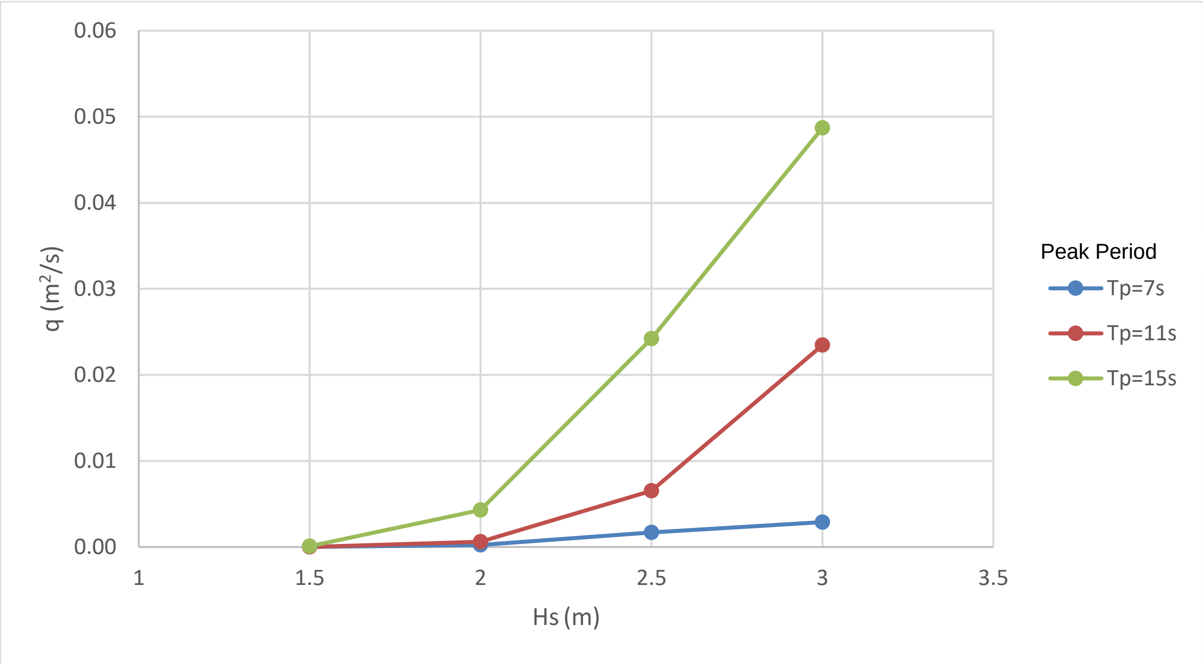

Figure 3: Overtopping Discharge Vs Significant Wave Height (d=12.1 m)

N.B. Graph lines have been joined for visual clarity only. Not representative of data extrapolation (Comment applies for all graphs).

Figure 12 shows the results from the baseline tests undertaken to understand the current overtopping discharge against significant wave height for the Dawlish sea wall without any modifications made to the wall. From this graph it can be clearly seen that as the wave height increases, so does the overtopping discharge. This is a very common finding for overtopping research as the increased wave height reduces the crest free board. Juhl & Sloth (1994) stated that the two most important factors with respect to wave overtopping was the significant wave height and the crest freeboard.

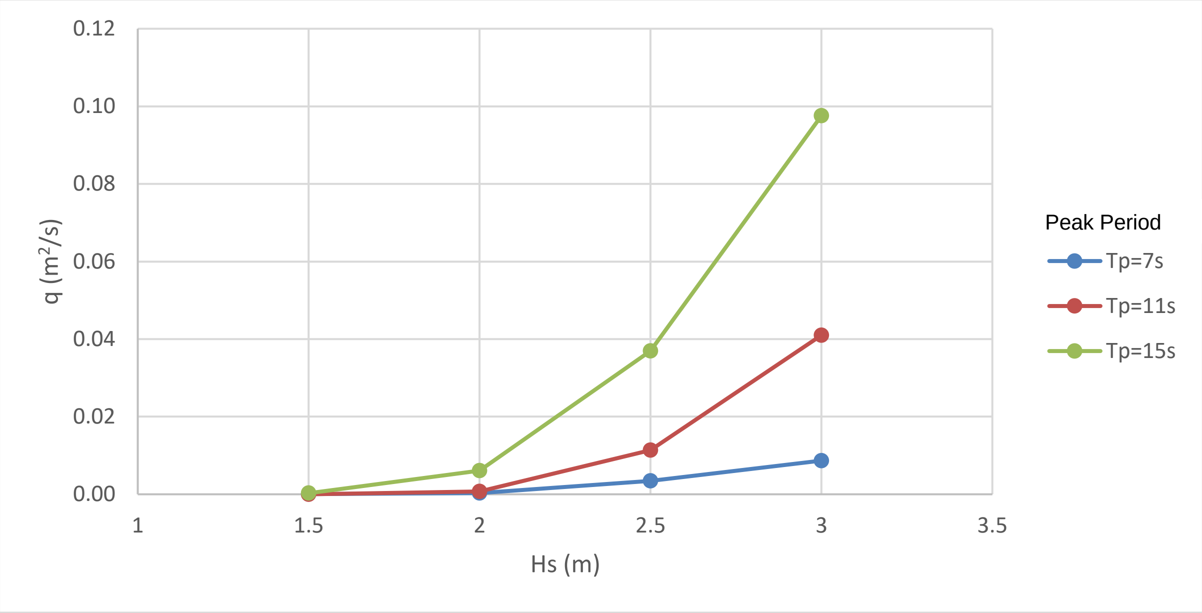

Figure 4: Overtopping Discharge Vs Significant Wave Height (d=12.6 m)

Figure 13 shows the baseline conditions for the Dawlish sea wall at an increased depth of 0.5 m (prototype) compared to Figure 12. It also shows the overtopping discharge against the significant wave height without any modifications to the wall. From this graph we can also see that as the wave height and period increases so does overtopping. Additionally, we can see that at a wave height of 3 m the increase in overtopping between the 11 s to 15 s period gives a far greater increase in overtopping when compared with the 12.1 m water depth. At 12.1 m the increase in overtopping between the 11 s to 15 s was 0.0252 m^2/s and the overtopping increase at 12.6 m depth was roughly 0.0566 m^2/s. This suggests the importance of freeboard height at reducing overtopping as confirmed by Franco & Aminti (1988) in 'Wave Overtopping On Rubble Mound Breakwaters', wherein they stated that "*Increasing the freeboard of the vertical wall (F) has the greatest effect in reducing the overtopping discharge*".

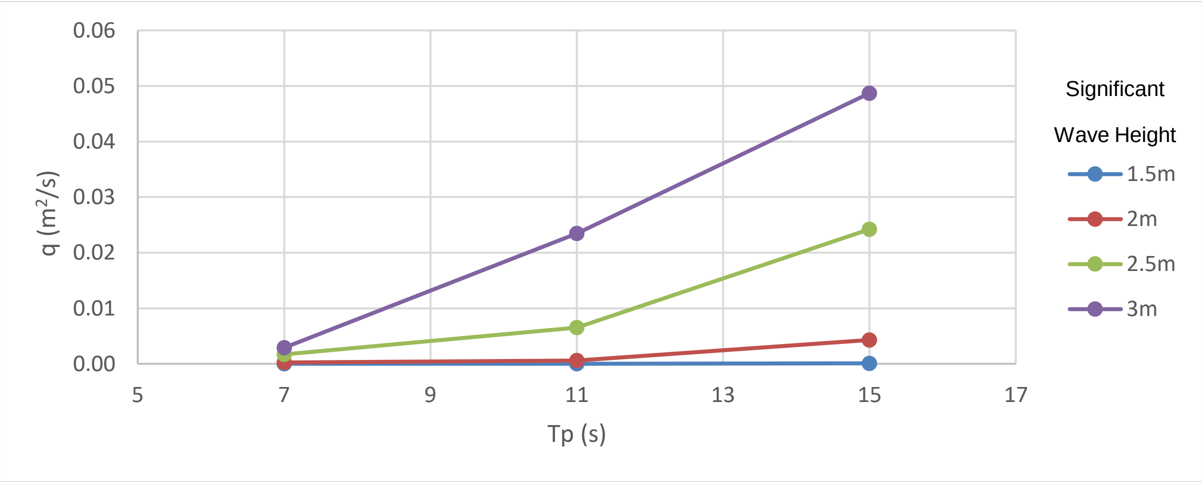

Figure 5: Overtopping Discharge Vs Peak Period (d=12.1 m)

Figure 14 shows the results taken from the baseline tests of the original Dawlish sea wall to provide a datum for our modified results. It shows the overtopping discharge against the period of the wave. From this graph we can see that as the period of the wave increases so does the overtopping discharge. These findings align with that stated by Bruce *et al* (2008) that the period of the wave is proportional to the wave overtopping on a structure, i.e. the larger the period the more overtopping occurs. The EurOtop Manual (2016) also agrees that the period becomes the main overtopping factor during a swell sea which is associated with long period waves (EurOtop, 2016).

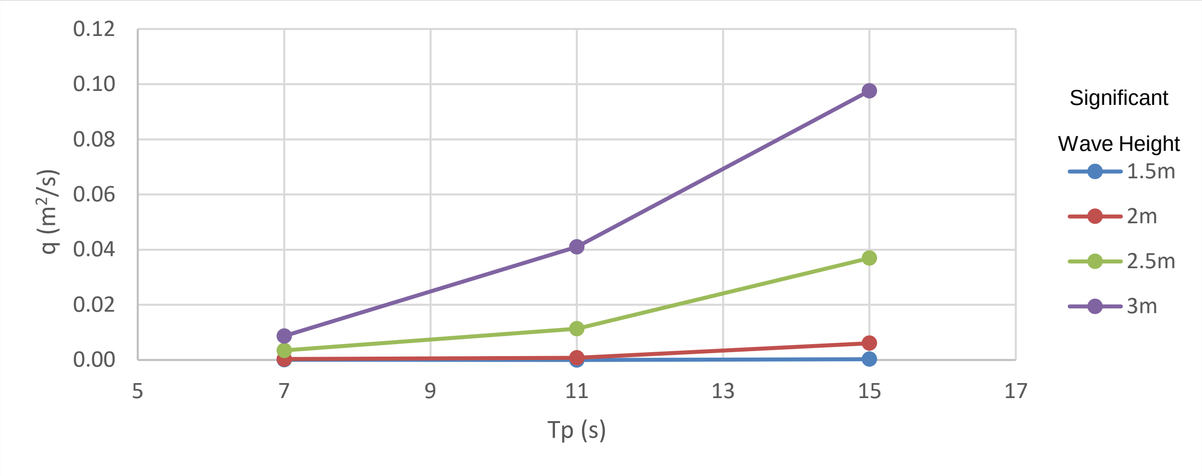

Figure 6: Overtopping Discharge Vs Peak Period (d=12.6 m)

Figure 15 shows the overtopping discharge against the period of the wave for the Dawlish sea wall without any modifications made to the wall. This was undertaken to provide a baseline to compare the results obtained from the modified Dawlish sea wall. This graph shows that as the period of the waves increase the overtopping and also wave height increase. We can see that due to the 0.5 m increase in water depth the overtopping has increased by 66.8% for the 3 m wave with a period of 15 s. However, the increase in water depth does not correspond to a linear increase of all the values, the same wave height with a period of 11 s has an increase in overtopping of 54.5%. This suggests that the greater the period the greater the increase in discharge due to the increased water depth.

Rubble mound

Figure 16 shows the results obtained from testing the Dawlish sea wall with an additional rubble mound revetment placed at the toe of the structure. We can immediately see that we have no overtopping for all periods until the wave height is increased to 2.5 m. The results obtained from this graph show a significant reduction in wave overtopping compared to the baseline results obtained from the experiment. For the 3 m wave height the overtopping was reduced by 66.5% for a 15 s period, 90.5% for a 11 s period and 100% for a 7 s period. This suggests that the rubble mound has less effect on larger period waves. There is little research as to why the period affects the overtopping, the JONSWAP spectrum relates the frequency (1/period) to the spectral density, which can be converted into energy. This would be interesting to explore further to find out if the energy related to each period was the factor that increased overtopping.

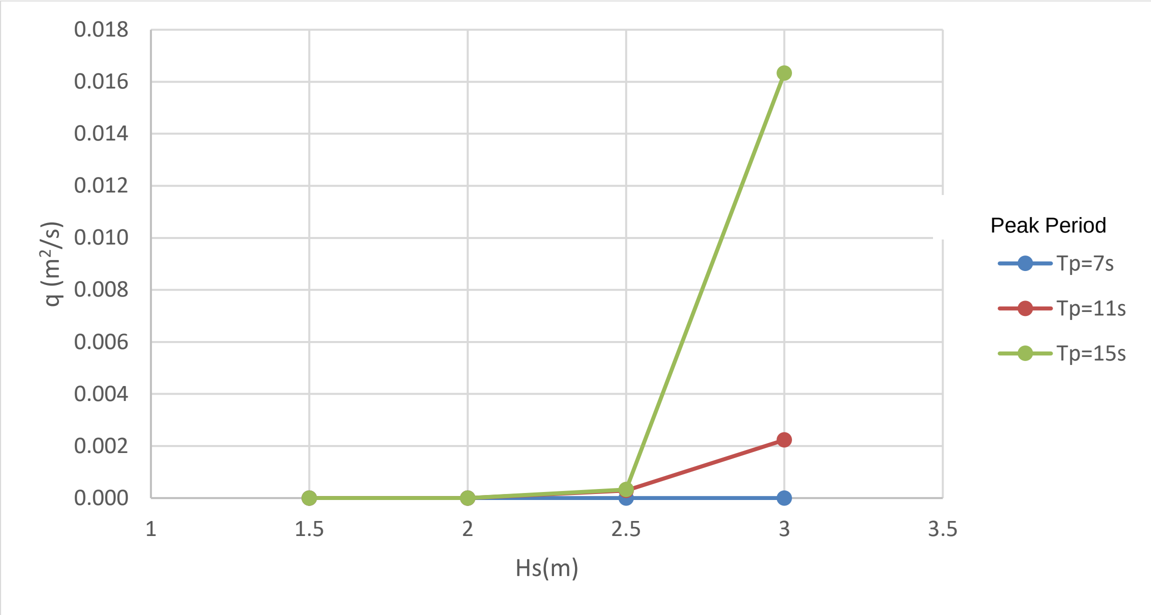

Figure 7: Overtopping Discharge Vs Significant Wave Height (d=12.1 m)

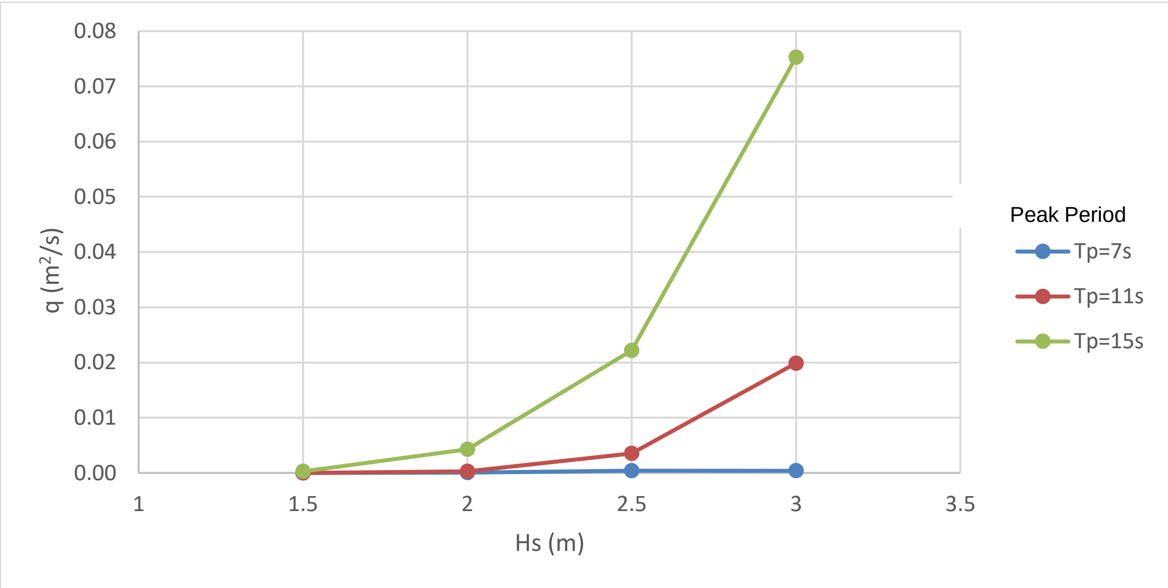

Figure 8: Overtopping Discharge Vs Significant Wave Height (d=12.6 m)

Figure 17 shows the overtopping discharge against the significant wave height for the modified wall. When compared against the baseline conditions we can clearly see that they both follow the same trend. However, it is important to note that the addition of the rubble mound breakwater has reduced the overtopping at a wave height of 2.5 m and period of 15 s by 39.9%. At a wave height of 2.5 m and a period of 11 s it was reduced by 69.0% and at the same wave height with a period of 7 s the overtopping was reduced by 89.2%. This graph also shows the same trend of the greater the period the smaller the reduction due to the rubble mound.

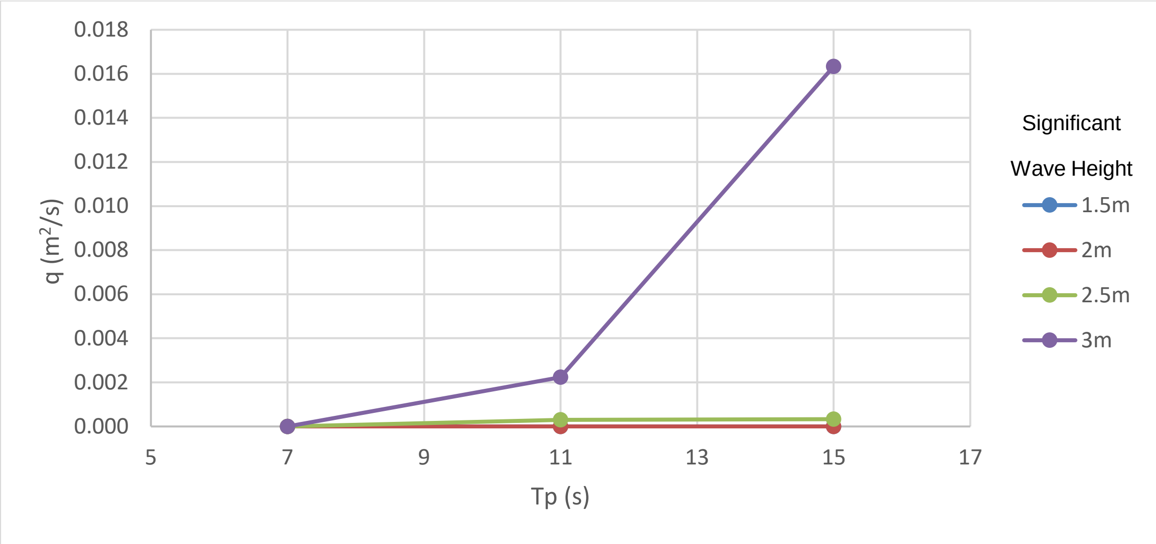

Figure 9: Overtopping Discharge Vs Peak Period (d=12.1 m)

Figure 18 shows the overtopping discharge against the period of the wave. For the 1 m and 2 m wave height the graph shows that there is no overtopping at any period. As displayed in Figure 17, at a depth of 12.1 m the rubble mound revetment had a 100% reduction in wave overtopping for all wave height and periods up to 2 m and past that reduces any wave height (up to 3 m) with a period of 7 s to 0 m^2/s/m.

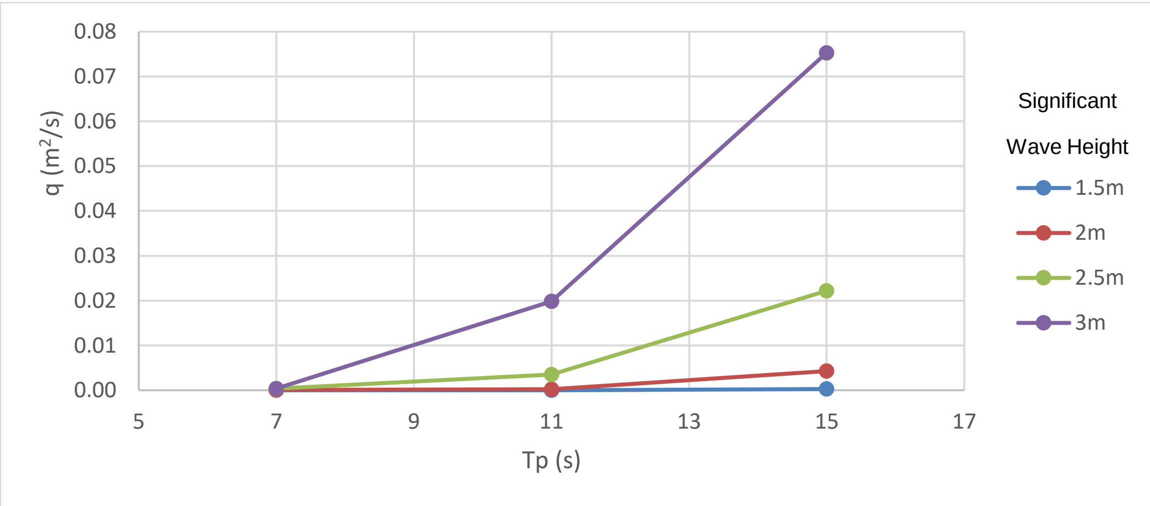

Figure 10: Overtopping Discharge Vs Peak Period (d=12.6 m)

Figure 19 shows the overtopping rates for 4 waves of varying height, each with 3 different wave periods. The heights and periods can be seen in Table 3 (page 110. When compared with the baseline results obtained earlier in this section we can see a reduction in wave overtopping of 22.8% at the highest wavelength and longest period. The reduction percentage increases as the period decreases similarly to the graphs produced for a 12.1 m water depth. Table 4 & 5 show the prototype overtopping for both the original Dawlish sea wall and the modified structure with a rubble mound revetment. The last column shows the % difference for each test.

Table 4 shows the results obtained for a depth of 12.1 m and has a range of values from a 100% reduction to 66.5%. Table 5 shows a range of reductions from 100% - 22.8%. It is interesting to note the anomalous data point that occurs for the 1.5 m wave height and 7 s period, it shows a value of overtopping as 0.001 $m^3/s/m$ which corresponds to 1 l/s/m. It is interesting to note this point, as the next data point returns to the norm and shows a value of 0 $m^3/s/m$ which is consistent with the findings for the 12.1 m water depth. A possible explanation for this error is due to some ingress of water into our collection tank from water droplets on the side of the tank.

Table 3: Percentage reduction in overtopping volumes (d=12.1 m)

Tp	Hs	Baseline Qp(m2/s)	Modified Qp(m2/s)	Difference %
7	1.5	6.57E-06	0	100
11	1.5	2.75E-06	0	100
15	1.5	8.76E-05	0	100
7	2	2.38E-04	0	100
11	2	6.00E-04	0	100
15	2	4.29E-03	0	100
7	2.5	1.69E-03	0	100
11	2.5	6.54E-03	0.000294763	95.49120323
15	2.5	2.42E-02	0.000330781	98.63384219
7	3	2.89E-03	0	100
11	3	2.35E-02	0.002232883	90.47811157
15	3	4.87E-02	0.016332717	66.46259388

Table 4: Percentage reduction in overtopping volumes (d=12.6 m)

Tp	Hs	Baseline Qp(m2/s)	Modified Qp(m2/s)	Difference %
7	1.5	0.0000	0.00E+00	100
11	1.5	0.0000	0.00E+00	0
15	1.5	0.0003	2.73E-04	0.657988914
7	2	0.0004	8.13E-05	76.7754103
11	2	0.0008	2.67E-04	64.38948644
15	2	0.0061	4.28E-03	29.36556172
7	2.5	0.0035	3.77E-04	89.15114979
11	2.5	0.0114	3.52E-03	68.95704783
15	2.5	0.0369	2.22E-02	39.92839484
7	3	0.0087	3.82E-04	95.59151972
11	3	0.0410	1.99E-02	51.50311423
15	3	0.0976	7.53E-02	22.83468217

Table 5: Tolerable overtopping

Depth (m)		12.1	12.6	12.1	12.6
Tp (s)	Hs (m)	Baseline Q (l/s)		Modified wall Q (l/s)	
7	1.5	0.00657	0.040042	0	0
11	1.5	0.002749	0	0	0
15	1.5	0.087582	0.275	0	0.273191
7	2	0.2375	0.35	0	0.081286
11	2	0.6	0.75	0	0.267079
15	2	4.2875	6.0625	0	4.282213
7	2.5	1.6875	3.475	0	0.376998
11	2.5	6.5375	11.35	0.294763	3.523375
15	2.5	24.2125	36.925	0.330781	22.18144
7	3	2.8875	8.675	0	0.382436
11	3	23.45	41	2.232883	19.88372
15	3	48.7	97.5625	16.33272	75.28441

The EurOtop Manual (2016) investigated the limits of overtopping of coastal structures for vehicles and people. Table 7 shows the results from the tests undertaken on the modified Dawlish sea wall and the baseline. Due to the close nature of the train line behind the wall and the footpaths that follow it, there is a very low tolerable discharge for overtopping.

Table 7 displays in green, the wave heights and periods and the overtopping values that fall within the limits set by the EurOtop Manual (2016) and highlights in red the overtopping that exceeds these limits. This shows the success and future proofing that the addition of the rubble mound would provide to the Dawlish Coastline. We can clearly see that the addition of the rubble mound would allow the train line to continue operating at the previously inoperable conditions of 12.1 m at a 2.5 m wave with a period of 15 s and a 3 m wave with a period of 11 s. At a depth of 12.6 m it allows the train line to operate at a wave height of 3 m and a period of 7 s. Although improvements have been made this continues to show the devastating impact that sea level rise has on coastal structures and significant overtopping. As the table shows, with the addition of a rubble mound at a depth of 12.1 m only a significant wave height of 3 m with a period of 15 s or more is unsafe to keep the train line running. However, due to the sea level rise of 0.5 m and the increase in overtopping caused by this, the train line can only operate on a significantly reduced schedule. The increase in water depth would require the train line to start checking if it should close when a wave height of 2.5 m is expected.

Reflection

Figure 20 shows a wave being reflected off the model after wave impact. This shows a source of error from the testing itself and is very important to note. Due to this reflection the incoming wave will either be increased or decreased depending on how the two waves interact, meaning that after the first few waves the overtopping values may be negatively skewed.

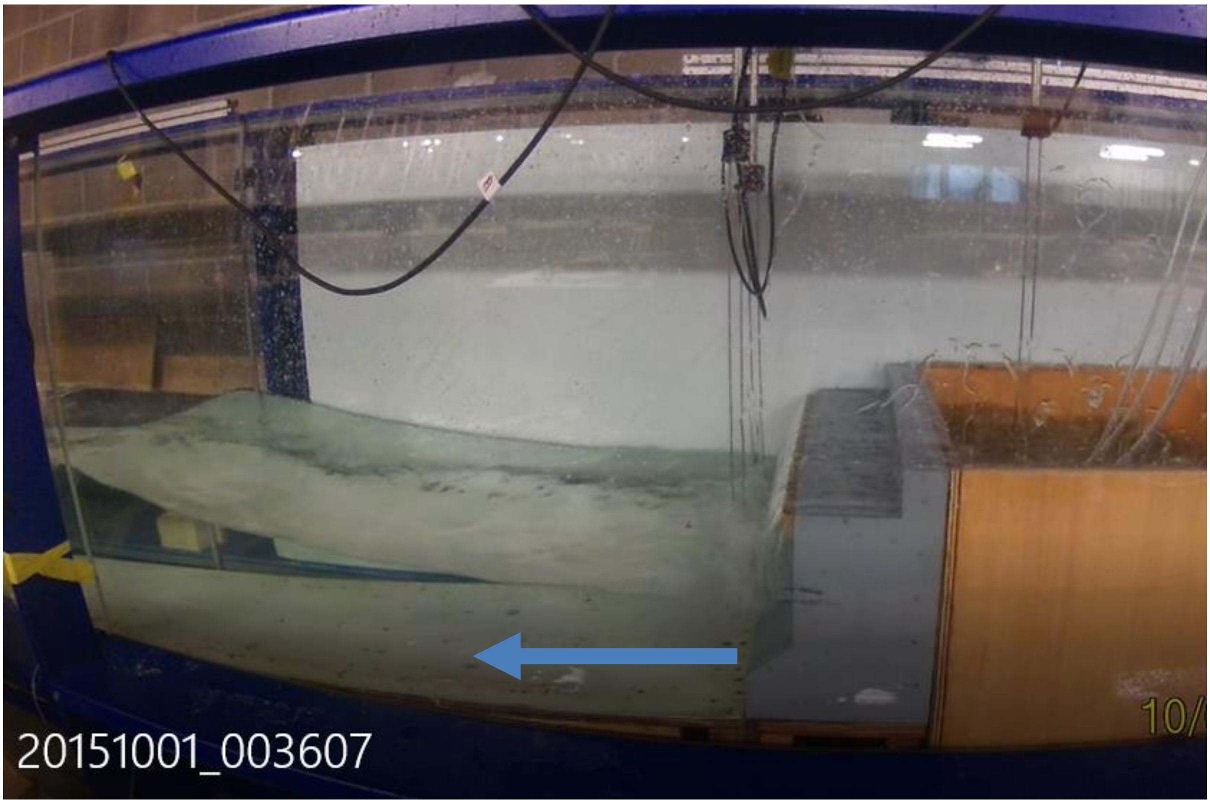

Figure 11: Reflection within the tank

This reflection was not removed during the process due to the complexity and advanced coding skills required to undertake the process. The results from this baseline test should be taken as a guide only.

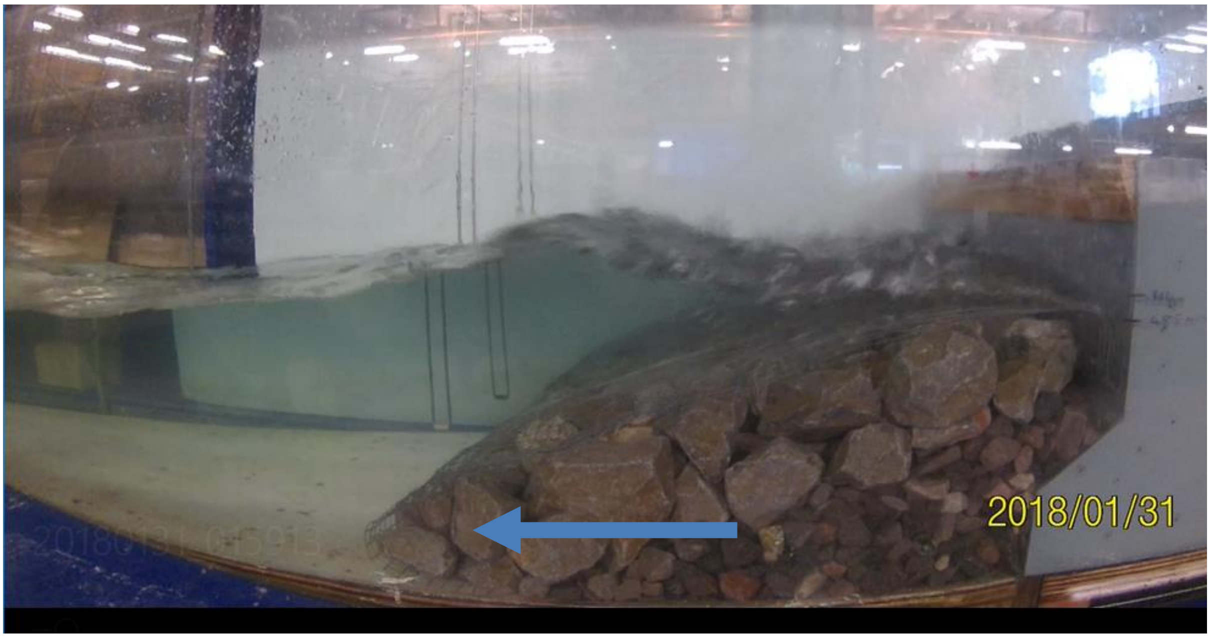

Figure 12: Rubble Mound Reflection

Figure 21 also shows wave reflection within the tank after a wave has impacted upon the structure. However, although the reflection is causing an error within the test it is interesting, because the reflected wave is much smaller than in Figure 20, it shows how much wave energy is absorbed by the rubble mound revetment.

Wave breaking

The green overtopping of both the current Dawlish defence with the addition of the rubble mound showed the same wave shape and similar overtopping rates (Figure 22)

Figure 13: Green Overtopping

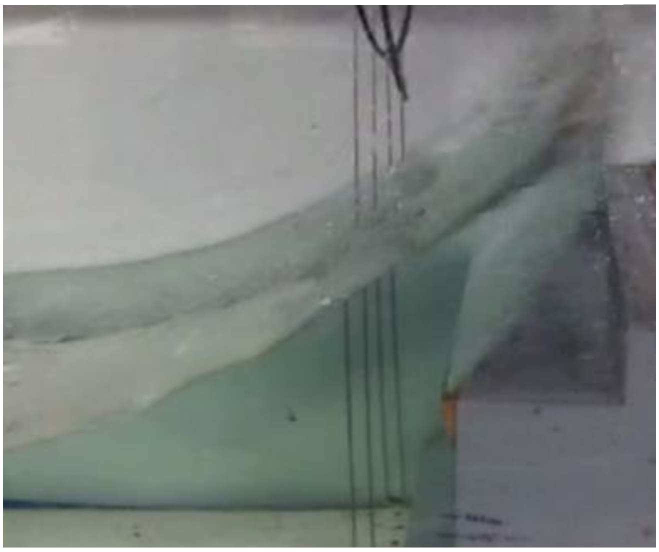

Figure 14: Green Overtopping

However, the overtopping caused by waves that entrap air showed a significant difference in overtopping when comparing the modified sea defence to the current sea defence. This suggests that the rubble mound absorbs a lot of the energy that would usually result in the explosive overtopping seen when air is trapped against vertical structures (Figure 23). Figures 24-25 shows the comparison between the two different types of wave shapes and overtopping rates.

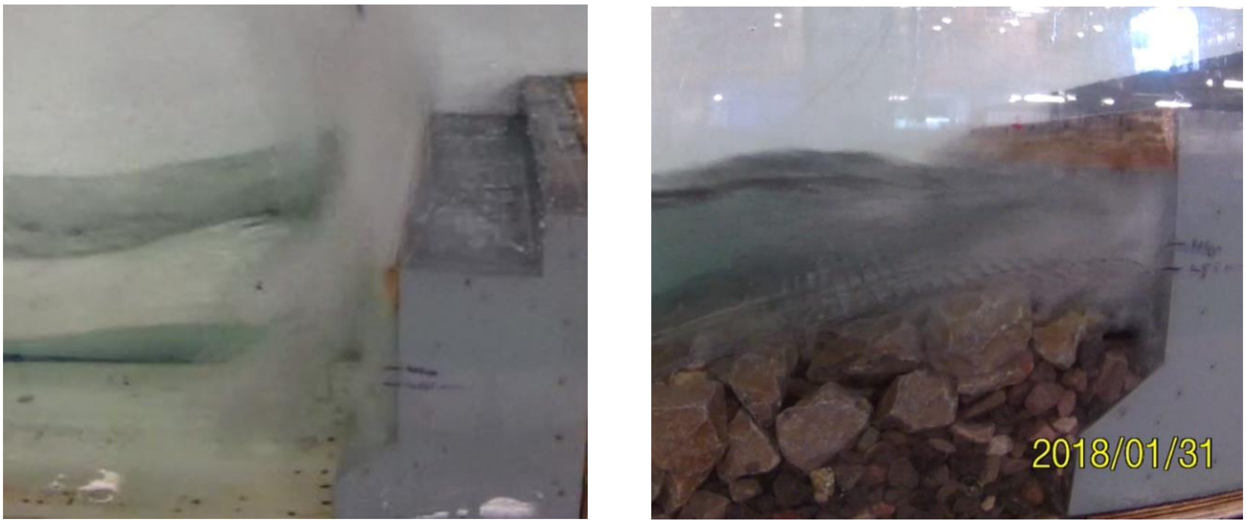

Figure 15: Comparison of wave shape

Figure 16: Comparison of wave overtopping

Conclusion

In conclusion, the most crucial factors when considering overtopping are the significant wave height and crest freeboard. However, none of the research considered, indicated the importance of period on overtopping, as the research has shown. As suggested in the results and discussion section more research on the relationship between period, energy and overtopping would be very interesting and could possibly be the source of another paper. The addition of the rubble mound revetment was a success in terms of overtopping reduction. The data show a significant reduction in overtopping volume for both the 12.1 m and 12.6 m water depths. For the water depth of 12.1 m we see an average reduction in overtopping of 95.9%, this suggests that the rubble mound would be a very effective method of reduction in the short term (i.e. before the sea level rises significantly due to global warming). At a depth of 12.6 m the average reduction in overtopping is 53.2%, this is significantly less than that of depth 12.1 m. This suggests that the rubble mound revetment would still be effective in 100 years' time but to ensure cost effectiveness the design should be enhanced to reduce overtopping further.

For the baseline results, at a depth of 12.1 m the train-line was unable to run at 3 out of the 12 cases tested. With the addition of the rubble mound the train-line would be able to run in all but the most extreme condition of a 3 m wave height with a 15 s period. For a depth of 12.6 m the train line would have problems running for the following conditions: 2.5 m wave height with a 15 s period and all three periods for a 3 m wave height. When the rubble mound was introduced the train-line was still unable to run at a wave height of 2.5 m and a period of 15 s, but was able to run at the previously inoperable condition of a 3 m wave height and a period of 7 s. For the 3 m wave height the overtopping was still too dangerous to run the train-line at a period of 11 s and 15 s. Although the rubble mound was a success in terms of reduction quantities it would still need to be re-designed to increase the lifespan and improve its performance against sea level rise. The redesign of the structure could include a bull nose, which have been known to significantly reduce overtopping as they reflect the wave back into the sea.

If more time was available, it would also be interesting to run numerical models on the Dawlish sea wall to see if they confirm the results found. If not, it would be interesting to then alter the numerical model to match the results found and improve the calculation for further research. To further reduce the overtopping a wave return wall could be added. This was researched by James Applegate (2018) and it was found to reduce overtopping at a depth of 12.1 m by 82% and at a depth of 12.6 m by 64%. Further research could be done on the design and cost effectiveness of a combination of rubble mound and wave return wall.

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