STABILITY OF BREAKWATER ARMOUR UNITS AGAINST TSUNAMI ATTACK

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Abstract

The design of breakwater armour units against tsunami attacks has received little attention in the past because of the comparative low frequency of these events and the rarity of structures designed specifically to withstand them. However, field surveys of recent events, such as the *2011 Great Eastern Japan Earthquake Tsunami* and the *2004 Indian Ocean Tsunami,* have shown flaws in the design of protection structures. During these extreme events, many breakwaters suffered partial or catastrophic damage. Although it is to be expected that most normal structures fail due to such high order events, practicing engineers need to possess tools to design certain important breakwaters that should not fail even during level 2 events. Research into the design of critical structures that only partially fail (i.e., "resilient" or "tenacious" structures) during a very extreme level 2 tsunami event should be prioritized in the future, and in this sense the present paper proposes a formula that allows the estimation of armour unit damage depending on the tsunami wave height.

Keywords: *rubble-mound breakwater; solitary waves; tsunami; Tohoku; stability; Hudson formula; Van der Meer formula*

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INTRODUCTION

On March 11, 2011, a large earthquake of magnitude 9.0 on the Richter scale occurred offshore the northeast coast of Japan, generating a major tsunami that devastated large parts of Japan's north-eastern coastline. This *2011 Great Eastern Japan Earthquake Tsunami* has been described as a one in several thousand years event, and was one of the worst tsunamis to affect Japan since records began. In its aftermath, the reliability of the different available tsunami counter-measures is being re-assessed, with important questions being asked about the ability of hard measures to protect against them. A variety of failure mechanisms have been reported for different types of structures (Mikami et al., 2012). Generally speaking, composite breakwaters (those protected by armour units such as tetrapods) were more resilient than simple caisson breakwaters. Armour units of different sizes and types were sometimes used in the same breakwater, with lighter units suffering more damage and showcasing how damage is dependent on the weight of the units (as can be expected from formulas such as that of Van der Meer, 1987).

To date, research has been carried out on the design of dykes and vertical structures against wind waves (Goda, 1985, Tanimoto et al., 1996), including assessments of the reliability of these structures (Esteban et al., 2007). For the case of solitary waves, Tanimoto et al. (1984) performed large-scale experiments on a vertical breakwater using a sine wave and developed a formula for calculating wave pressure. Ikeno et al (2001, 2003) conducted model experiments on bore type tsunamis and modified Tanimoto's formula by introducing an extra coefficient for wave breaking. Mizutani and Imamura (2002) also conducted model experiments on a bore overflowing a dike on a level bed and proposed a set of formulae to calculate the maximum wave pressure behind the dike. Esteban et al. (2008) calculated the deformation of the rubble mound foundation of a caisson breakwater against different types of solitary waves. However, all the methods outlined above deal with simple type caisson structures or dykes, though many composite breakwaters exist (where the caisson is protected by armour units placed on its seaside part). To this effect, Esteban et al. (2009) calculated the effect that a partially failed armour layer would have on the forces exerted by a solitary wave on a caisson, allowing for the determination of the caisson tilt. Subsequently, Esteban et al. (2012a) proposed an initial formula for the design of armour units against tsunami attack, though this formula was based on the analysis of only two ports in the Tohoku area, and thus its accuracy is questionable. Formulae that can be used to design armour stones against anticipated current velocities are already given in the Shore Protection Manual (1977), based on a variety of previous research. More recent researchers (see Sakakiyama, 2012, Hanzawa et al., 2012, Kato et al., 2012) have also proposed methods to design armour against tsunami attack, focusing on the current velocity and overtopping effect, though it can be difficult for a practicing engineer to reliably estimate these parameters in the case of an actual tsunami.

In the present work, the authors have set out to verify the accuracy of the formula of Esteban et al. (2012a) by expanding the analysis to a number of other ports that were affected by the *2011 Great Eastern Japan Earthquake and Tsunami* and the *2004 Indian Ocean Tsunami*. The goal is to obtain a formula that can be easily applied by a practicing engineer to check whether a certain armour layer (in either a composite or rubble mound breakwater) is likely to catastrophically fail during a given tsunami event.

Following the *2011 Great Eastern Japan Earthquake Tsunami* the Japanese Coastal Engineering Community has started to classify tsunami events into two different levels (Shibayama et al., 2012), according to their level of severity and intensity. Level 1 events have a return period of several decades to 100+ years and would be relatively low in height, typically with inundation heights of less than 7-10 m. Level 2 events are less frequent events, typically occurring every few hundred to a few thousand years. The tsunami inundation heights would be expected to be much bigger, typically over 10 m, but would include events of up to 20-30 m in height.

The way to defend against each tsunami level would thus follow a different philosophy. “Hard measures”, such as breakwaters or dykes, should be strong enough to protect against loss of life and property for a level 1 event. However, the construction of such measures against level 2 events is often seen as unrealistic from a cost-benefit point of view. Thus, during these events it would be accepted that hard measures would be overcome and the protection of the lives of residents would rely on “soft measures”, such as evacuation plans and buildings. Nevertheless, hard measures would also have a secondary role to play in delaying the incoming wave and giving residents more time to escape. Although many structures in tsunami-prone areas are designed primarily against storm waves, it is desirable that they can survive level 1 tsunami events with little damage to continue to provide some degree of protection to the communities and infrastructures behind them.

BREAKWATER FAILURES DURING PAST TSUNAMI EVENTS

To derive a formula for the design of breakwater armour units against tsunami attack, the authors used real-life failures of armour unit layers at several locations along the south-west of the Sri Lankan (for the *2004 Indian Ocean Tsunami*) and northern Japanese (for the *2011 Great Eastern Japan Earthquake and Tsunami*) coastlines. The authors themselves carried out the surveys, relatively independently from other researchers during the 2004 event (Okayasu et al., 2005, Wijetunge, 2006), and as members of the larger Tohoku Earthquake Tsunami Joint Survey Group in 2011 (Mori et al., 2012, Mikami et al., 2012). Also, the authors continued to return to the Tohoku area at regular intervals during the 18 months that followed the event, compiling further reports of the failure of various breakwaters along the affected coastline. A summary of each port surveyed is given in the sections below.

For each breakwater section an armour damage parameter, *S*, similar to that used in Van der Meer (1987) was obtained, which was defined as follows:

$$S = \frac{A_e}{D_{n50}^2} \qquad (1)$$

where A_e is the erosion area of the breakwater profile between the still water plus or minus one wave height and D_{n50} is the mean diameter of the armour units. For the case of the Sri Lankan ports this *S* value was based on surveys of the average required volumes of material required to restore each breakwater to its initial condition, while for the case of Japan it was based on the number of armour units missing from the most severe damaged parts of each breakwater section. *S*=15 defines catastrophic damage (Kamphuis, 2000), and thus any damage with *S* higher than this value (e.g. for the case of rubble mound breakwaters) was assigned *S*=15.

Damaged ports in Sri Lanka

Sri Lanka was hit by a massive tsunami, triggered by a 9.0 magnitude earthquake, off the coast of Sumatra, on 26 December 2004. It was the worst natural disaster ever recorded in the history of the country, causing significant damage to life and coastal infrastructure. A total of 1,100 km of coastline was affected (particularly along the east, south and west of the country), leaving approximately 39,000 dead and destroying 100,000 homes. Fisheries were badly damaged, including the ports of Hikkaduwa, Mirissa and Puranawella. A considerable variation in tsunami inundation heights was recorded, ranging from less than 3.0 m to as high as over 11.0 m, with the height generally showing a decreasing trend from the south to west coast (Okayasu et al., 2005; Wijetunge, 2006).

Hikkaduwa Fishery Port

Hikkaduwa port is located on the southwest coast of Sri Lanka, approximately 100 km south of Colombo. It is situated at the northern end of Hikkaduwa town, between Coral Garden Bay and Hikkaduwa River and by the side of the Colombo-Galle (A002) highway. The region is a major tourist destination, possessing a submerged coral reef in the near shore area which highlights its ecological importance as a conservation area. The Hikkaduwa fishery anchorage evolved as a result of structures that were constructed to prevent sand bar formation across the Hikkaduwa river outlet. The harbour basin is enclosed by the southern and northern breakwaters, with the outer breakwater taking off from the southern breakwater to provide the necessary shelter during the SW monsoon. The length of the southern (main) and outer breakwater is approximately 378 m while the length of the northern (secondary) breakwater is 291 m.

The seaside and leeside of the main breakwater was covered with 1.0 to 3.0 ton rock armour while the outer breakwater used 6.0 to 8.0 ton armour. The head of the outer breakwater consisted of 8.0-10.0 ton armour. The tsunami waves which approached the port were relatively small since they had undergone diffraction due to the geographical features of the southern coast of Sri Lanka. Figure 1 illustrates the damage to the primary armour of the outer breakwaters. Water depths in front of the breakwaters at these damaged sections were found to be approximately 0.5 to 4.0 m below MSL at the time of survey. The measured tsunami wave height at this location was 4.7 m, and as the freeboard was 3.5 m this would imply that the tsunami would have overtopped the breakwater with an overflow height of 1.2 m. The average *S* factor for the main section of the outer breakwater was 4.5.

INSERT FIGURE 1

Mirissa Fishery Port

Mirissa fishery port is located in the eastern side of Weligama Bay, which is approximately 27 km east of Galle. This location is ideal for a fishery port as the eastern headland of the bay provides protection from the SW monsoon waves. The port consists of a 403 m main breakwater and a 105 m secondary breakwater. The seaside of the main breakwater was covered with 4 to 6 ton primary rock armour while the leeside used 3 to 4 ton armour. Figure 2 illustrates the damage observed at the seaward side of the main breakwater. The water depths at the main breakwater varied from 3.0 to 5.0 m below MSL at the time of the field survey. The measured tsunami wave height at this location was 5.0 m and thus would have resulted in an overflow height of 1.5 m (as the freeboard of the breakwater was 3.5m). The average *S* factor was 5.3.

INSERT FIGURE 2

Puranawella Fishery Port

Puranawella fishery harbour is located at the southern end of Sri Lanka and consists of two rubble mound breakwaters: the main breakwater (405 m long) at the southern side and the secondary breakwater (200 m long) at the northern side of the harbour. The tsunami caused extensive damage to both breakwaters and other fishing facilities. The primary armour was displaced at several locations along the main breakwater, as shown in Figure 3. The root of the seaside of the main breakwater was covered by 2.0 to 4.0 ton primary armour while the seaside and leeward of the trunk section used 4.0 to 6.0 ton armour. The breakwater head was covered with 5.0 to 8.0 ton rock armour. Water depths at the main breakwater varied from 3.0 to 7.0 m MSL at the time of field survey. The measured tsunami wave height at this location was 6.0 m and the corresponding *S* factors were 3.71 and 7.38 for the root and trunk sections, respectively. The freeboard in all sections was 3.5 m, and thus the tsunami would have overtopped all sections with an overflow height of 2.5 m.

INSERT FIGURE 3

Japanese Ports

Kuji Port

Kuji port, located in the northern part of Iwate Prefecture, has a composite breakwater that uses 6.3 ton tetrapod armour units, as shown in Fig 4. The breakwater was directly facing the incoming wave, and thus would have been directly hit by the tsunami. Interestingly, the armour units were placed in a very steep layer, though there did not appear to be any major damage due to the tsunami event (S=0). Probably the reason why no damage occurred is because of the relatively low tsunami inundation height in this area, with values of 6.34 m , 6.62 m and 7.52 m measured behind the breakwater by the Tohoku Earthquake Tsunami Joint Survey Group in 2011 (6.62 m was selected for the subsequent analysis of the armour unit stability).The freeboard was 6.2 m, and thus the tsunami would have hardly overtopped the breakwater, with an overflow height of between 0.14 to 1.32m.

INSERT FIGURE 4

Noda Port

Most of the composite caisson breakwater at this fishing port withstood well the tsunami attack, except for one section, where both the caissons and the 3.2 ton tetrapod armour units protecting it were completely removed and scattered by the force of the wave (*S*=15). Figure 5 shows how the damaged section was temporarily repaired using much bigger 25 ton tetrapod units. The inundation heights measured by the Joint Survey Group behind the breakwater were 16.58 m, 17.64 m and 18.3 m. Thus, for this location a wave height of 17.64 m was selected as representative for the analysis. According to this, the breakwater would have suffered an overflow water height of 12.24 m, as the freeboard was only 5.4 m. The breakwater was directly facing the incoming wave, though the failure mechanism is not clear, as the section that failed was not located near the head of the breakwater, but in an area closer to land. Local bathymetry effects might have played a role in intensifying the height of the wave at this section in the breakwater though a more detailed analysis would be needed before any definite conclusions can be reached. The remaining section of the breakwater held up relatively well, even though it was composed of the same type of units.

INSERT FIGURE 5

Taro Port

The various breakwaters that protected Taro port suffered extensive damage, as shown in Fig. 6. The breakwater at the entrance of the bay (sections A-C in Fig. 6) was composed of 2 distinct sections: approximately two-thirds had 800 ton caissons protected by either 70 or100 ton hollow pyramid amour units (two types of weights were used in its construction), with the remaining being protected by similar armour but without any caisson behind them (as this section of the structure was located in an area of complex bathymetry next to small islands; see Fig. 6). The “rubble mound type section” (section C) was completely destroyed, with the armour scattered by the force of the tsunami (*S*=15).

Behind this breakwater there were two composite breakwaters consisting of 25 ton tetrapods that were completely destroyed by the tsunami, with the caissons and tetrapods scattered around the port (*S*=15). Figure 6 shows the final location of some of these caissons from aerial photographs obtained by the authors through a private communication.

To obtain an estimation of the height of the wave as it struck each element of this port would be difficult, and there is considerable disparity in the measurements by the Joint Survey Group. Measurements of 13.86 m, 15.18 m, 19.55 m, 19.56 m, 21.03 m and 21.95 m were taken at various locations behind the breakwaters. All these points were located away from the main breakwater that was protecting the entrance of the bay, thus adding to the uncertainty of the actual wave size that hit the structure. Part of the difference in these measurements could be related to the complex sheltering process provided by the various breakwaters, as shown in Figure 6. Also, some small islands were present in the offshore area, and while these are unlikely to have provided much protection, they could explain some of the scatter in the recorded inundation heights. It is thus likely that at least the outer breakwater could have faced a wave of 21.03 m and that the inside breakwater possibly faced a smaller wave (15.18 m). The freeboard of the breakwaters was approximately 4.1 m, resulting in overflow heights of 15.93 m at the outer breakwater and 11.08 m in the inside.

By September 2012 many of the scattered armour units had been collected and placed back to their approximate original locations. Section C (the outside breakwater, made of hollow pyramids) had been restored to its initial condition, and the 25 ton tetrapods had been used to create a new rubble mound breakwater around section D (which no longer had caissons behind it). Also, at this time, new tetrapod armour units were being manufactured to re-build the remaining sections of the breakwater.

INSERT FIGURE 6

Okirai Port

This fishing port was protected by a composite armour breakwater that used 3.3 ton x-blocks, which were completely removed and scattered around the port by the force of the tsunami (S=15). In this case not only the armour but also the some of the caissons failed (see Fig. 7). The breakwater was not directly facing the open sea, but rather situated at the inside of Okirai Bay, slightly to the north of the opening. Thus, reflection and diffraction processes could have played a part in altering the shape of the wave. The Joint Survey Group recorded inundation heights of 15.54 m, 15.57 m and 16.17 m behind the breakwater, and thus a value of 15.57 m was selected as representative for this location, resulting in an estimated overflow height of 13.57 m (2.0m freeboard)

INSERT FIGURE 7

Ishihama Port

This fishing port is located along a relatively straight stretch of the coastline to the east of Kesenuma. Two composite breakwaters of roughly the same size had been constructed at this location, both of which used tetrapods. However, the size of the armour units varied throughout both breakwaters. The north side breakwater had 2 ton armour at the edge with the land, which failed and were just visible above the water line (*S*=15). The central part of the breakwater had 8 ton tetrapods, which partially failed (*S*=5). Finally, the head of the breakwater was protected by massive tetrapods which did not appear to have been significantly displaced (one unit had been clearly displaced, and it could have been possible that more were slightly moved, though it is difficult to ascertain this without knowing the original position of the units). None of the caisson units in the northern breakwater appeared to have experienced any displacement.

The southern breakwater was also protected by relatively small 2 ton armour near to its land side, which failed similarly to those at the northern part (*S*=15). The central section was protected by what appeared to be a mixture of armour unit weights, 2 ton, 3.2 ton and 6.3 tons in size. The reason for this mixture is unclear, and it is possible that some of the lighter units were originally from an adjacent section and were carried by the wave. Nevertheless, gaps in the armour could be observed in this section, equivalent to an *S*=4. The final section of the breakwater was made of much heavier 6.3 ton units that appeared not to have been displaced. However, the head of the breakwater had not been protected by armour, resulting in the last caisson tilting into the sea, though still remaining accessible from the adjacent caisson.

Inundation heights of 14.88 m, 15.39 m and 15.54 m were measured by the Joint Survey Group behind the breakwater and thus a wave height of 15.39 m was used in the analysis of this structure. The freeboard varied along different sections of the breakwater (between 5.2 m and 5.6m), resulting in overflow heights of approximately 10 m.

INSERT FIGURE 8

Hikado and Ooya Ports

These two composite breakwaters are situated fairly close to each other and face the open sea, such that the tsunami would have struck them directly. Three different measurements of wave heights were taken in this area, 15.7 m (by the authors themselves) and, 15.0 m and 16.55 m (by other members of the Tohoku Earthquake Tsunami Joint Survey Group). In the present analysis, the authors chose to use their own value of 15.7 m for the tsunami height at the breakwater. The freeboard at Ooya was 1.8 m and that at Hikado was 3.4 m, resulting in overflow heights of 13.9 m and 12.3 m, respectively.

Esteban et al. (2012a) reported that three different types of armour units were present at the breakwaters. Ooya port had 3.2 ton Sea-Locks (See Fig. 9), and Hikado port had both 5.76 ton X-block sand 28.8 ton Hollow Pyramid units along the breakwater (X-blocks in the body of the breakwater and heavier Hollow Pyramids at the head, as shown in Fig. 10). The X-block and Sea-Lock armour completely failed; the units were scattered over a wide area in front of the breakwater, with only the top of some of them still showing above the water surface. However, none of the caissons at either of these ports suffered any noticeable damage.

INSERT FIGURES 9 and 10

LABORATORY EXPERIMENTS

Esteban et al. (2012a) performed laboratory experiments using solitary waves generated by a wave paddle in a wave flume at Waseda University, Japan (dimensions 14 m × 0.41 m × 0.6 m). The experimental layout they used is shown in Fig. 11. A rubble mound breakwater protected by two layers of randomly placed stone was constructed on one side of the tank (a total of 3 different stone sizes were used, with median weights *W* of 27.5 g, 32.5 g and 37.5 g). Esteban et al. (2012a) tested two different breakwater configurations, with a seaward angle, α, of 30^0 and 45°. Each of the breakwater configurations was also tested for three different water depths, *h*=17.5 cm, 20 cm and 22.5 cm, none of which resulted in the overtopping of the breakwater.

The wave profile was measured using two wave gauges, one located in the middle of the tank and the other one just before the breakwater (to measure the incident wave height). Solitary waves that with a half-period *T*/2=3.8 s were used to simulated the wave. Since the experiments were carried out in a 1/100 scale, this represents a *T*=76 s wave in field conditions (using Froude scaling). The waves generated were 8.4 cm in height, corresponding to 8.4 m in field scale. The height of the wave, *H,* was identical in all experiments, as the input to the wave paddle remained unchanged.

The average number of extracted armour units for each experimental condition was counted with the aid of a high-speed photographic camera and each of the experimental conditions was repeated 10 or 15 times to ensure accurate results. Generally, damage to the 45° structure was far greater than to the 30° structure, as expected. The wave profile did not significantly change according to the water depth in front of the breakwater, and thus the pattern of damage did not appear to be significantly sensitive to this parameter. This is different from the results of Esteban et al. (2009), who found that different types of waves could be generated for different depths (bore-type, breaking and solitary type waves). However, in the experiments of Esteban et al. (2012a) the water depth did not vary sufficiently between each experimental condition to result in significant differences in the wave profile.

ANALYSIS

The authors used the Hudson formula (CERC, 1984, Kamphuis, 2000) as the starting point for the analysis. According to this formula, the weight of required armour, W, is proportional to the incident design wave height, H, as follows:

$$W = \frac{\gamma H^3}{K_D (S_r - 1)^3 cos\alpha} \qquad (2)$$

where γ is the density of armour (tonnes/m³), S_r is the relative underwater density of armour and K_D is an empirically determined damage coefficient. A summary of the values of K_D used for the various types of armour units analysed in the present research can be found in Table 1 (Kamphuis, 2000). The use of Hudson K_D values is for rubble mound structures exposed to wind waves which are not overtopped. Hence, the way in which they are being included in the present study is not that for which they were intended (i.e., for very long period waves overtopping rubble mound structures and composite breakwaters). Nevertheless, when resisting the tsunami current forces the armour units will benefit from an interlocking effect, and in the absence of any better measure it is proposed that these K_D values are used.

INSERT TABLE 1

Unlike formulae such as that of Van der Meer's, the Hudson formula does not provide an indication of the degree of damage that can be expected for a certain event (although it should be noted that typically Hudson K_D values are considered to indicate 0%-5% damage levels, the Hudson formula cannot predict higher levels of damage). However, the objective of the present work is to attempt to quantify structure resilience. Thus the damage to each section of the armour of each breakwater was interpreted using a damage factor S similar to that used by Van der Meer (1987), as shown in Eq. (1). A ratio R was defined as the weight of armour, $W_{required}$, that would be required according to the Hudson formula using the height of the tsunami ($H_{tsunami}$) as H_s over the actual weight, W_{actual}, of the armour at the breakwaters in the field, given by:

$$R = \frac{W_{actual}}{W_{required}} \qquad (3)$$

Where:

$$W_{required} = \frac{\gamma H_{tsunami}^3}{K_D (S_r - 1)^3 \cos\alpha} \qquad (4)$$

Table 2 shows a summary of the parameters used in each of the breakwater sections that were analysed. Figures 12 and 13 illustrate the ratio *R* versus *S* values for composite and rubble mound breakwaters, showing how armour units that had lower values of *R* failed completely (represented by higher *S* values) whereas units with higher *R* only showed partial or no failure. In Figure 13 it the field results represent breakwaters that were overtopped, whereas those in the laboratory were not, and thus these two sets of data cannot be interpreted together. The reasons for including the data is only to show that the laboratory experiments provide some evidence for the shape of the trend line drawn, i.e., to expect a low *S*, a large *R* is required for the case of rubble mound breakwaters.

INSERT TABLE 2

INSERT FIGURES 12 and 13

MODIFICATION TO THE HUDSON FORMULA FOR TSUNAMI EVENTS

According to the results outlined in the previous sections, the authors developed a modification to the Hudson formula that could be employed for the design of armour units in tsunami prone areas. Thus, armour units would first be designed using the Van der Meer or Hudson formulae against wind waves in the area, as usual in the design of any breakwater. However, at the end of the design procedure a check should be made that the breakwater meets the requirement of the formula below:

$$W = A_t \frac{\gamma H_{tsunami}^3}{K_D (S_r - 1)^3 \cos\alpha}$$

(8)

where $H_{tsunami}$ is the tsunami level specific wave height at that location and A_t is a dimensionless coefficient obtained from Table 3. This A_t depends on the type of breakwater and tsunami level, includes the effects of overtopping, and is derived from Figs. 12 and 13.

For level 1 events, the armour in all breakwaters should experience little to no damage (i.e., an *S* value less than 2) since the breakwater would have to resist not only the first wave of the tsunami but also subsequent waves, and thus it is imperative that the structure does not deform significantly, or that partial failure in the armour does not result in an amplification of wave forces (Esteban et al. 2009 showed how a partly failed armour layer can amplify the forces exerted by a solitary wave on the caisson of a composite breakwater). However, for level 2 it is expected that normal breakwaters would fail, and designing them against these high-order events is probably uneconomical. Nevertheless, and although uneconomical, a practicing engineer might need to design a certain breakwater against these high order events (for example a port that might be used for relief operations after such a disaster). In this case, these “important breakwaters” should be designed with a partial failure in mind (maybe with an S=4) so that they can continue to provide protection yet not prove too expensive. In such breakwaters the possibility of overtopping should be allowed, as the crucial point would be for them to be used after the event, and designing them against the $H_{tsunami}$ of a level 2 event would require unnecessary high freeboards. One important exception to this would be breakwaters protecting critical infrastructure, whose failure could have disastrous consequences (one example could include the protection of a nuclear power station). It should be noted that by this statement the authors are not saying that the construction of such breakwaters would make nuclear installations 100% safe. The construction of nuclear power stations in tsunami and earthquake prone areas generally pose important risks to coastal communities, as exemplified by the Fukushima disaster following the *2011 Great Eastern Japan Earthquake Tsunami*. These should be designed using the most conservative parameters possible ($H_{tsunami}$ of a level 2 event and an A_t=1), with the crest of the breakwater higher than the $H_{tsunami}$ for a level 2 event.

INSERT TABLE 3

In this type of design, it would be very important to analyse $H_{tsunami}$ correctly, and to do this a certain wave height should be chosen, corresponding to historical records of tsunamis in the area and to the perceptions of accepted risk. For the case of Japan, these are framed around the dual tsunami level classification, where the highest tsunami inundation level that is believed can occur at a given place (for a return period of several thousand years) should be used for the level 2 $H_{tsunami}$. Thus, depending on the area where a breakwater is to be designed and the tsunami risk in the region, the required W of the armour would be ultimately determined by the wind wave conditions, or by the tsunami risk.

To illustrate this philosophy, Table 4 shows an example of the armour requirements for two of the ports surveyed by the authors, for different port classifications. In both of the ports shown, it is assumed that $H_{tsunami}$=7 m for a level 1 event and $H_{tsunami}$ is equal to that experienced during the *2011 Great Eastern Japan Earthquake Tsunami* for a level 2 event. This shows how, assuming that the armour and breakwater type stayed the same, both Taro and Ooya currently have armour units of approximately the size required to withstand a level 1 event (the Sea-Locks at Ooya are slightly smaller than required, 3.2 tons vs. the 3.8 tons required, though this probably would not warrant the reinforcement of the units). However, if disaster risk managers (for whatever reason) required the outside breakwater of Taro to be operational after a tsunami event, then 190 ton units would be needed, almost twice the size of the largest units (100 tons). If a nuclear power station was to be built behind it, this would require units weighting 290 tons, the crest of the breakwater to be over 21 m high, and a change in the nature of the breakwater (as a caisson would be required to ensure that the area behind it would not be flooded).

INSERT TABLE 4

DISCUSSION

The field trips in Tohoku attempted to establish the extent of damage in the armour by visual inspection, though this was difficult because the position of the original units were not known. The *S* values given in the present study are an estimate of the missing number of armour units in a section, though it was difficult in many cases to know whether units had moved during the tsunami. In some breakwater sections, for similar armour weights, some parts showed more damage than others, and the *S* was reported for the most damaged sections, not an average. Limitations of using this *S* parameter were evident during the field surveys, e.g. the case of breakwaters that had massive armour but were situated in relatively low water. Thus, an *S* value of 2 or 3 would probably represent complete failure of the armour (because of the limited number of units). Although this did not influence the present results (as these massive units did not fail), this parameter is thus not well suited for small breakwaters protected by massive armour. Also, the way that the *S* values were calculated for these composite breakwaters differed from that used to calculate the rubble mound values (both for the laboratory experiments and the Sri Lankan ports), which were averages of the breakwater sections evaluated.

Judging from video footage of the *2011 Great Eastern Japan Earthquake Tsunami*, these events comprise complex phenomena, and one of the defining failure modes might be the overtopping effect of the wave. A prolonged overflowing effect would generate a very intense current, and many structures along the Tohoku coastline appeared to have failed due to erosion of the landside toe of the structure. This has led some researchers (Kato et al., 2012, Sakakiyama, 2012, Hanzawa, 2012) to state that the failure mode is directly related to this overflowing current. Nevertheless, the initial impact of the wave also has an effect on the breakwater armour, and it would appear logical that once this initial wave shock has been absorbed, the overflowing current would have no effect on the armour units. Also, although ultimately the current might be the determining factor in the failure of the armour units, there is probably relationship between the height of the wave and the magnitude of the current. Establishing the exact current magnitude for a given tsunami event is far more difficult than establishing the tsunami wave height (which can easily be measured through field surveys). Thus, the formulae proposed can be used as a proxy for the effect of the current, and thus be easily used by a practicing engineer in determining the required armour size.

The design of a composite or rubble mound breakwater in a tsunami zone is thus a complex process. Not only does the stability of the armour have to be checked against wind waves in the area, but also against tsunamis. The exact failure mechanism for each of the breakwater types is still unclear, and whether armour units were displaced by the incoming or the outgoing wave could not be easily established for any of the field failures recorded. In any case, all the breakwaters were overtopped, and the entire area was completely underwater at one point during the tsunami attack (which would have also generated large underwater currents around the structures). Importantly, the landside part of the structure should also be checked for potential scour from the wave as it starts to overtop. It is likely that most of the landside toe failure occurs during the initial overtopping, since once a large inundation height is established behind the breakwater the current would probably flow at a higher level, and thus scour would be less significant. Finally, the effect of the returning wave should also be checked, as this can result in the inverse process and lead to the destruction of many structures that survived the initial wave attack, as evidenced in the Tohoku area.

Previously tsunami counter-measures in Japan had been designed to be higher than the expected tsunami wave height, though they were clearly under-designed for the *2011 Great Eastern Japan Earthquake Tsunami*. Following this event there is a general perception that it is too difficult and expensive to design tsunami counter-measures against level 2 events. However, it is also clear that some important structures might have to be designed so that they fail in a non-catastrophic way. These were described by Kato (2012) as “tenacious structures”, representing a structure that would slowly fail over the course of the event while retaining some functionality (this idea is similar to what has been described by other authors as “resilient” structures, which would indicate a structure that would suffer limited damage even if its design load was greatly exceeded). The difference between “tenacious” and normal structures is shown by the failure of the breakwaters at Kamaishi (which could be regarded as a “tenacious structure”, as it suffered great damage but somehow survived the event) and that at Ofunato (which was completely destroyed).

The erection of vertical barriers and dykes can clearly give extra time for residents to evacuate even if they suffer major damage due to a level 2 event. Much is still not understood about the failure of protective measures in the event of a tsunami, and their ability to delay the arrival of the flooding water must be carefully balanced against the extra cost of the armour units. In this respect, significant research is still needed to ascertain the failure mechanism of armour units, and whether their placement will increase the forces acting on the caissons behind them, especially if the armour units fail (Esteban at al., 2012b). Also, the inclusion of crest levels and overtopping depths in an equation to predict failure should be prioritized in future research.

Unfortunately, ascertaining adequate level 2 tsunami heights is difficult. It requires adequate historical records, spanning millennia, though most countries' histories are far shorter, and, even when tsunamis are recorded in historical documents these do not usually show very detailed information (particularly for the case of the earlier documents). The field of paleotsunami can thus be very useful, though it often appears to be difficult to get reliable results as the top levels of the soil in urban areas can be disturbed by human activities, and these are the areas which are of greatest concern as they concentrate most of the coastal population (Shibayama et al., 2012).

CONCLUSIONS

Following the *2011 Great Eastern Japan Earthquake Tsunami* there is a general perception that much is still unclear about the failure mechanism of coastal defences. The present research describes the field surveys of real life breakwater failures in the Tohoku region and South Western of Sri Lanka and attempts to obtain a design methodology for armour units based on this evidence. This methodology was inspired by the Hudson formula, but uses the failure definitions given in the Van der Meer formula. It is recommended that breakwaters in tsunami-prone areas should be designed to withstand level 1 events, but that only important infrastructure should be designed to remain functional (allowing partial failure equivalent to an S value of 4) even after being overtopped by the more extreme level 2 tsunami events. Critical infrastructure (such as that protecting nuclear installations) should be designed to avoid any damage or overtopping to take place even during level 2 events.

Establishing the required tsunami inundation heights for level 1 and 2 events is notoriously difficult, and requires the study of ancient records and tsunami deposits. As most countries do not have records that span several millennia and these records are often not detailed, the study of tsunami deposits and seismic faults should be intensified to determine the worst events that can be expected in each region.

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Table 1. Summary of armour units surveyed.

Unit	Approximate Weight	K_D
Sea-Lock	3.2 tons	10
X-Block	5.76 tons	8
Hollow Pyramid	28.8 tons	10
Tetrapods	Varies	8
Rock	N/A	4

Table 2. Summary of all the parameters used in the analysis of each breakwater section

Breakwater Section	Type	$H_{tsunami}$ (m)	freeboard (m)	W_{actual} (tons)	S	K_D	α	$W_{required}$ (tons)
Ooya Port	Comp.	15.7	1.8	3.2	15	10	30	122.2
Hikado Port X-Block	Comp.	15.7	3.4	5.8	15	8	30	152.7
Hikado Hollow Pyramids	Comp.	15.7	3.4	28.8	4	10	30	122.2
Kuji Port	Comp.	6.62	6.2	6.3	0	8	45	19.8
Taro Hollow Pyramids (A1)	Comp.	21.03	4.1	70	0	10	30	293.7
Taro Hollow Pyramids (A2)	Comp.	21.03	4.1	100	0	10	30	293.7
Ishihama tetrapod (A1)	Comp.	15.39	5.2	2	15	8	30	143.9
Ishihama tetrapod north (A2)	Comp.	15.39	5.4	8	5	8	30	143.9
Ishihama tetrapod north (A3)	Comp.	15.39	5.6	16	1	8	30	143.9
Ishihama tetrapod south (A1)	Comp.	15.39	5.2	2	15	8	30	143.9
Ishihama tetrapod south (A2)	Comp.	15.39	5.2	3.2	4	8	30	143.9
Ishihama tetrapod south (A3)	Comp.	15.39	5.2	6.3	0	8	30	143.9
Taro Tetrapods	Comp.	15.18	4.1	25	15	4	30	276.1
Noda port	Comp.	17.64	5.4	3.2	15	4	30	433.3
Okirai (X-Block)	Comp.	15.57	2	3.3	15	4	30	298
Hikkadua Section 2-7	R. M.	4.7	3.5	6	5	4	30	8.2
Mirissa Section 1	R. M.	5	3.5	2	6	4	30	9.9
Mirissa Section 2-10	R. M.	5	3.5	4	5	4	30	9.9
Puranawella Section Observed 2, 1A, 1, 2A, 2	R. M.	6	3.5	4	4	4	30	17.1
Puranawella Section 5, 6A, 6	R. M.	6	3.5	5	7	4	30	17.1
Taro Hollow Pyramids (B1)	R. M.	21.03	4.1	70	15	10	30	293.7
Taro Hollow Pyramids (B2)	R. M.	21.03	4.1	100	15	10	30	293.7
Lab Experiments (rock, A1)	R. M.	8.4	Non overtopped	28	0	4	30	40.4
Lab Experiments (rock, A2)	R. M.	8.4	Non overtopped	28	0	4	45	70
Lab Experiments (rock, B1)	R. M.	8.4	Non overtopped	33	0	4	30	40.4
Lab Experiments (rock, B2)	R. M.	8.4	Non overtopped	33	0	4	45	70
Lab Experiments (rock, C1)	R. M.	8.4	Non overtopped	38	0	4	30	40.4
Lab Experiments (rock, C2)	R. M.	8.4	Non overtopped	38	0	4	45	70

Table 3. Values of A_t for different breakwater types and tsunami Levels.

Type of Breakwater	Structure Type and Tsunami Level used for $H_{tsunami}$		
	Normal breakwater (Level 1 tsunami)	Important breakwater (Level 2 tsunami)	Critical breakwater (level 2 tsunami)
Rubble Mound	1.0	0.65	1.0
Composite	0.35	0.15	1.0

Table 4. Example of required armour size for different type of breakwater types

Breakwater and armour unit	Breakwater type	Type	$H_{tsunami}$	A_t	$W_{required}$	Notes
Taro Hollow Pyramids	Normal	R. M.	7	1	10.8	Pre-tsunami armour was 70-100 tons
	Important	R. M.	21.03	0.65	190.9	
	Critical	Comp.	21.03	1	293.7	
Ooya Port Sea-Lock	Normal	Comp.	7	0.35	3.8	Pre-tsunami armour was 3.2 tons
	Important	Comp.	15.7	0.15	18.3	
	Critical	Comp.	15.7	1	122.2	