

EFFECT OF WAVE DISSIPATORS ON OVERTOPPING OVER A BERMED SEAWALL IN DEEPWATER

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Wave overtopping is an important factor for designing harbor breakwater and shore-protection structure, and the safety of facilities, working areas and people in their lee. This paper reports the results of a series of laboratory experiments on the effect of six types of wave energy dissipation mechanisms on wave overtopping over a bermed seawall that protects a reclaim land for a specialized yacht industry zone in Taiwan. Numerical method using NN-Overtopping is also applied to determine suitable building setback distance for storm in different recurrent periods.

Keywords: bermed seawall; wave overtopping; laboratory experiments; NN-Overtopping; building setback

INTRODUCTION

Wave run-up and overtopping over a maritime structure during the action of a storm is important for assessing the crown elevation, construction cost and safety of the structure, and their impact on the drainage, building and operations in the lee. Conventionally, quantification of the overtopping over a structure with a vertical or inclined seaward face has been studied analytically and experimentally since the 1950s (e.g., Saville 1955, Hunt 1959, Ishihara et al. 1960, Iwagaki et al. 1965, Shiraishi et al. 1968, Shi-igai and Kono 1970, Goda 1975, Owen 1980, Aminti and Franco 1988, Mizuguchi 1993, Franco et al. 1994, Allsop et al. 1995, van der Meer and Janssen 1995, Besley, 1999, Goda 2000, Bruce et al. 2006). Both regular and irregular waves have been used in the laboratory on a model structure with or without armor units to reduce the incoming wave energy and overtopping. From these, various empirical formulae or graphs for estimating mean overtopping rates have been derived from laboratory data for scientific applications (Shore Protection Manual 1984).

Among the investigators who have worked on wave overtopping, Goda (1975) reported the results of a series of experiments for seawall with vertical and simple front slope, from which non-dimensional overtopping rate $Q = q/(2gH_o)^{1/2}$ was plotted using deepwater wave steepness (H_o/L_o), water depth, and the slope of the seabed; where q is the dimensional overtopping rate per unit run of the structure. In addition, Owen (1980) conducted experiments for overtopping on seawalls in simple and composite forms (with berm) subject to irregular waves, and proposed an exponential form $Q = A \exp(-BR_q)$ for non-dimensional overtopping rate Q and the non-dimensional freeboard R_q ; where coefficients A and B were obtained by regression analysis, $Q = q/T_m g H_s$, $R_q = R_c/T_m (g H_s)^{1/2} \times 1/\gamma$; in which T_m is the mean wave period, H_s is the significant wave height, R_c is the freeboard of the structure, γ represents roughness coefficient. For bermed seawall, he included additional factors, such as the front slope, berm width, water depth above the berm, and surface roughness. Later, Franco et al. (1994) adopted the same exponential form of $Q = A \exp(-BR_q)$, but using simplified expression of $Q = q/(g H_s^3)^{1/2}$ and $R_q = R_c/H_s \times 1/\gamma_s$; where γ_s donates the effect of structure on overtopping. Van der Meer and Janssen (1995) considered the effect of other factors on overtopping (wave breaking, wave obliquity, front slope, berm property, surface roughness and shallow water effect), and defined $Q = q/(g H_{mo}^3)^{1/2}$ and $R_q = R_c/H_{mo} \gamma_b \gamma_f$ using wave height H_{mo} in spectral waves. Allsop et al. (1995) discussed the effect of water depth on overtopping for breaking and non-breaking waves. Furthermore, while also casting the non-dimensional overtopping rate $Q = A \exp(-BR_q/\gamma)$ in exponential form, Besley (1999) investigated the effect of freeboard R_c ; where $R_q = R_c/T_m (g H_s)^{1/2}$, and A and B are empirical coefficients dependent on the front slope of the structure. On the other hand, Aminti and Franco (1988) defined an different form for overtopping, $Q = A R_q^{-B}$, where $Q = q/T_m g H_s$ and $R_q = R_c/T_m (g H_s)^{1/2} \times R_c/H_s$ to examine the effect of freeboard on a bermed seawall. Bruce et al. (2006) compared the overtopping performance using different armor units on rubble mount breakwaters.

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Overall, the work of Owen (1980) was well recognized for his effort in establishing the formulation framework that continues to be used today, as well as that of Besley (1999) who had comprehensively addressed overtopping for different structural forms (Reeve et al. 2012). However, modern research on wave overtopping in Europe has emphasized the approach using integrated numerical model, especially in neural network (CLASH 2001, Pozueta et al. 2004, NN_Overtopping Manual 2005), based on a prodigious amount of data from laboratory tests and field measurements in many countries. These efforts have notably culminated in the production of the EurOtop Manual (2007), which presents a comprehensive set of design methods and equations and working tool, dealing with wave run-up and overtopping for a wide range of seawalls, dykes and armored rubble mound structures (see also Pullen et al. 2008, Allsop et al. 2008).

While Taiwan's luxury sail boat manufactures have been well known internationally in recent years, all yachts have been built on land away from the sea and on-site ship loading facilities for testing are scarce. In 2010, the City Government of Kaohsiung, a modern port city in Taiwan, announced a project to develop a 110-hectare specialized zone for the yacht industry in southern Taiwan. The site, with its ocean-fringing land reclaimed from land-fill over several years, aims to accommodate 30 yacht builders and peripheral manufacturers to build and launch ships for testing. Assessment of wave overtopping risk across its 4-km long frontage was part of the EIA to ensure the safety for the building behind the seawall, upon considering global warming with sealevel rise and the increase in storm intensity against the reclaim land for the Nanxing project (Fig. 1). Consequently, a study was called for wave overtopping over the seawall during storm attack, in order to determine the amount of wave overtopping, subsequent design of drainage system for the site, and the setback distance required for the safety of building and human movement behind the seawall.

This paper aims to present the results of the laboratory experiments and numerical methods, to check the suitability of the existing overtopping equations, in addition to determine the building setback distance. During the experiments, a total of six wave energy dissipation mechanisms were tested and their performance of overtopping on a scaled model was compared. Numerical method, such as NN_Overtopping, was also performed to estimate the mean overtopping rate and other statistical percentages in probability, as well as to assess the risk to the manufactory building setback at 30 to 50 m behind the seawall under storm waves in 10-, 25-, 50- and 100-year recurrent period. Verification of overtopping phenomenon using other numerical tools (e.g., CADMAS-Surf and Flow3D) was also performed successfully.



Figure 1. Nanxing yacht industry zone on a reclaim site in Kaohsiung, showing western seawall.

LABORATORY EXPERIMENTS

Condition of Seawall

The western seawall protecting the reclaim land has a seaward face with slope 1:2 (vertical: horizontal) above the berm that covers the bulk of inner core stones (see cross-section view in Fig. 2). The seaward slope and berm is covered by 20-tonne precast "Link" blocks in double layers of 4 m high in total, from the large foot-protection mounts near the toe at -7.5 m to +7.93 m, almost reaching the crown at +8.50 m. However, these artificial blocks, initially in neat and tidy patterns on the front slope, have been disturbed and scattered along the seawall (Fig. 3) under the attack of severe typhoons

(tropical cyclones) since it was first built in 2000. This implies different roughness effect for wave run-up and overtopping along the seawall. In addition, the reclaim land, though seemingly flat, inclines slightly toward its south and does not have sufficient capacity to drain the storm water that may overtop the seawall, for which suitable drainage system is required.

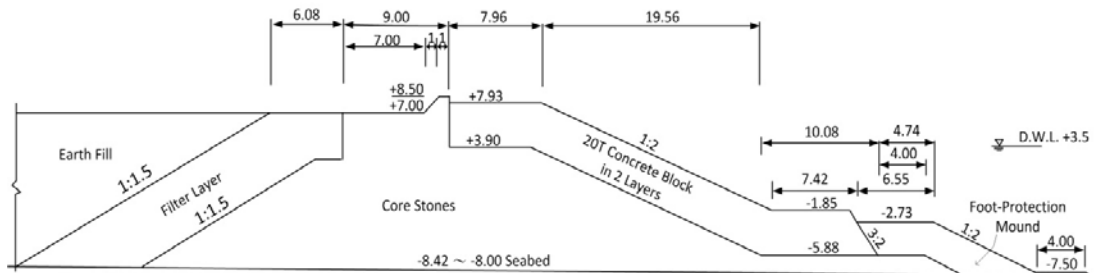


Figure 2. A typical cross-section of the seawall for Nanxing yacht industry zone. (Units: meters)



Figure 3. Armoring blocks scattering on the seaward face of the seawall (29th April 2011).

Wave Conditions and Tidal Levels

The regional coastline in Kaohsiung aligns with the SE to NW direction. It receives predominate waves from SW during summer to autumn, relatively weaker waves from NW in the other seasons, and high waves from SSW to WNW quarters when typhoon attacks the southern part of Taiwan in summer to autumn. According to the typhoon database of the Central Weather Bureau (CWB) in Taiwan, the island state was directly attacked by 3~4 typhoons yearly in average during 1958-2013. Among these, about 42% of the typhoons (or at least 1.5 typhoons in each year) had produced severe waves and heavy rainfall at the project site. In general, maritime structures in Taiwan have been designed using storm waves with 50-year recurrent period, for which deepwater wave condition (at 192.4 m depth) with significant wave height and period of $H_s = 7.7$ m and $T_s = 11.7$ s from SW direction has been adopted, and $H_s = 9.2$ m with $T_s = 12.7$ s from SSW; while storm waves with 10-year recurrent period are $H_s = 5.2$ m with $T_s = 9.6$ s from SW, and $H_s = 6.1$ m with $T_s = 10.4$ s from SSW. For storms in 100-year recurrent period, wave condition in deepwater is $H_s = 8.7$ m with $T_s = 12.4$ s from SW, and $H_s = 10.4$ m with $T_s = 13.5$ s from SSW. Should the one-tenth wave height be required, Rayleigh distribution applies, such that $H_{1/10} = 1.273 H_s$. In this study, the original deepwater wave height at 192.4 m depth was transformed to the depth of 31.65 m and scaled down as the depth in the wave flume to generate the incident waves by a piston-type wave maker.

Information for the local tidal levels is required for studying wave overtopping, since a design storm wave may affect the seawall during different stages of a tidal cycle, with additional water level induced by storm surge and the prevailing waves. The hourly reconstructed tidal levels at Number 10 Pier in Kaohsiung Harbor during 1966~2001 are used, after filtering the ascending trend of water level and abnormal data. From this, the data from January 1984 to June 1999 are selected, in which relevant tidal levels for the low tide system in Kaohsiung Harbor are: the highest high water level (HHWL) at +2.60 m and spring high water level (WHL) at +1.30 m, while the mean sea water level (MWL) is +0.74 m. The elevations of HHWL and WHL are scaled down to 0.646 m and 0.619 m, respectively, for the laboratory tests. The magnitude of storm surge is calculated using the pressure difference

between the storm center and the ambient atmospheric condition, from which the maximum value of 1.0 m may be added to the HHWL or HWL for overtopping experiments.

Experimental Set-up

The experiments for overtopping were conducted using a two-dimensional wave flume of 1.2 x 1.0 x 35 m (high, inner width, length) available in National Sun Yat-sen University in Kaohsiung, Taiwan (Lee 2013a). The length scale of 1:49 (model:prototype; hence time scale of 1:7) was chosen to fit the model seawall with a wooden approaching seabed slope of 1:10 into the partitioned wave flume. The flume was equipped with a piston-type wave maker to generate the model waves (0.134~0.223 m at toe, with period 1.37~1.67 s) for the experiments (Fig. 4). Five capacitance wave gages were deployed to measure the incident wave height (P1), wave heights in transformation (P2 and P3) leading to the toe of the seawall (P4; for numerical calculation using NN_Overtopping to be described later), as well as to monitoring the instantaneous water levels in the collection tank (P5) that represent the overtopping quantity (Fig. 5). Video recorder using a SONY digital camera stationed in front of the flume was also in use to record the image of wave overtopping. To reduce the total number of model blocks for the experiments, the inner width of the flume was partitioned, such that only 0.25 m out of the total width of 1.0 m was used for installing the model seawall with collection tank and different arrangements of wave dissipators (model blocks) during experiments, while keeping the berm width constant as 0.16 m.

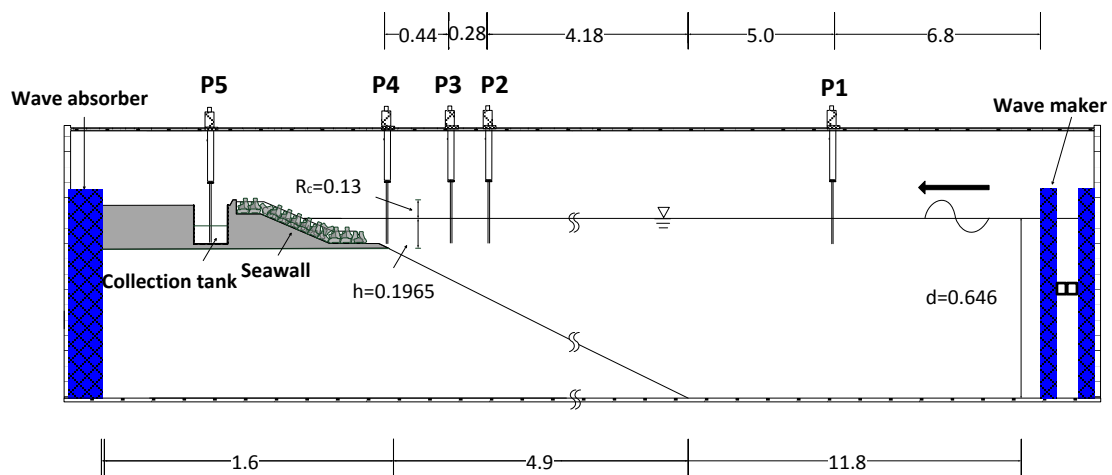


Figure 4. Schematic view of wave flume, wave maker, wave gages, model seawall and water collection tank. (Units: meters).

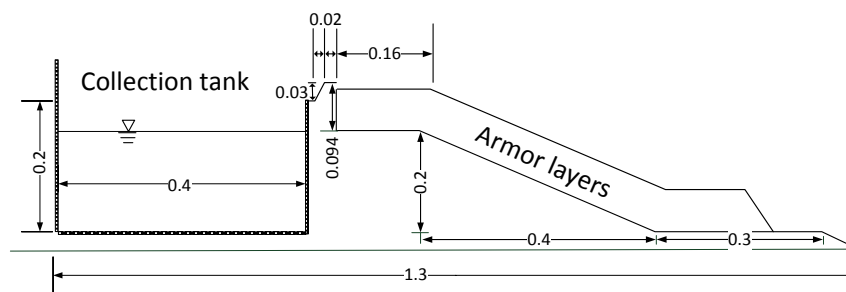


Figure 5. Details of model seawall cross-section and water collection tank. (Units: meters)

Arrangements of Wave Energy Dissipators

A total of 6 types of wave energy dissipation mechanism were tested in the laboratory. These included a basic type with smooth front slope and five others with different arrangements of wave energy dissipation. Each type was designated by a single-word abbreviation as follows:

- (1) smooth: smooth front slope,
- (2) rough: rough front slope,
- (3) single: model blocks in single layer on front slope,
- (4) double: model blocks in double layer on front slope,
- (5) submerged: model blocks for submerged breakwater, and
- (6) detached: model blocks for emergent detached breakwater.

The first type (Fig. 6a), as the basic type with a smooth front slope made of clear acrylic in 5 mm thickness, received the largest quantity of overtopping in the whole series of experiments. In order to reduce the overtopping rate, the second type (Fig. 6b) was introduced by placing a PVC anti-slip mattress on the front slope to increase the surface friction. To further reduce the overtopping, model blocks in single and double layers on the front slope were arranged, as the third and fourth types, respectively. In type 3 (Fig. 6c), aluminum model “Link” blocks (66 mm wide and 75 mm high) were arranged in 4 columns cross the partitioned space of 0.25 m in the flume and by 13 rows in the direction of wave propagation. A total of 55 blocks were deployed, which included 52 Link blocks on the slope above and three large concrete cubes beneath them to preventing the sliding of the blocks above. With blocks in double layers (type 4, Fig. 6d), the base layer was the same as that in type 3, while the upper layer had blocks in 3 columns by 13 rows, resulting in total of 94 model blocks. Type 5 in the form of submerged breakwater (Fig. 6e) had 19 blocks on the bottom layer and 15 blocks above it, and all 34 blocks were placed on the berm of the model seawall. Finally, in type 6 associated with emergent detached breakwater (Fig. 6f), the third and fourth layers using 8 and 4 blocks, respectively, were added to the layout in type 5.

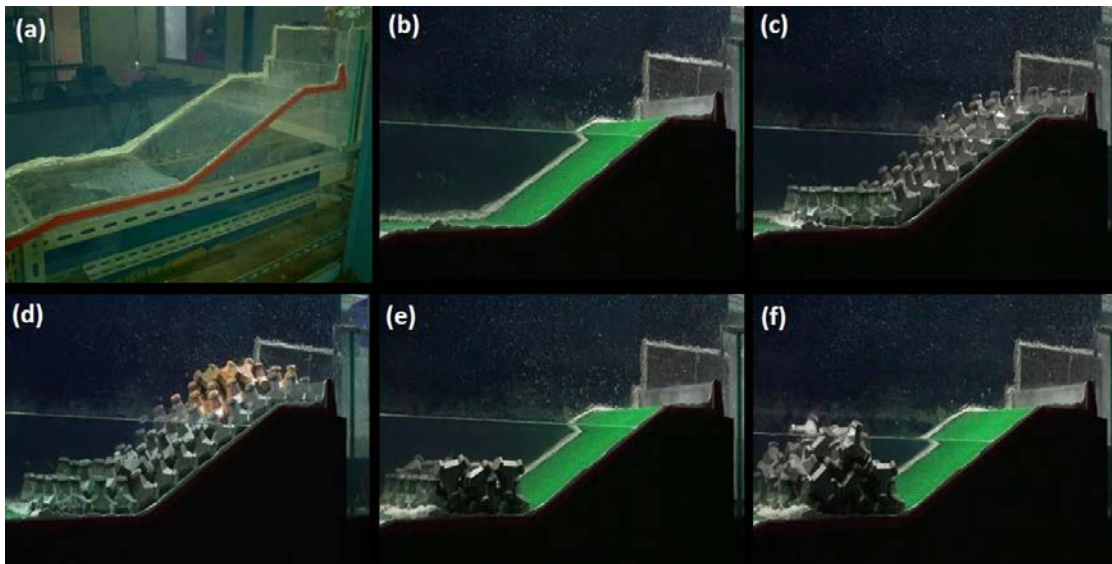


Figure 6. Model seawall with layout of different wave energy dissipation mechanisms tested in the laboratory: (a) smooth front slope, (b) rough front slope, (c) blocks in single layer, (d) blocks in double layers, (e) submerged breakwater, (f) emergent detached breakwater.

EXPERIMENTAL RESULTS AND ANALYSIS

Processing Experimental Data

In every test for the 6 types of wave dissipation mechanism mentioned above, a total of 25 waves were generated in the flume and the incident wave profiles were recorded by gage P1 (Fig. 4), while the temporal variations in overtopping rate monitored by a vertical scale attached to P5. The latter provided the volume of individual wave overtopping rate (m^3/s) after calibration using linear regression equation. Each run was repeated at least once, rendering a total of 149 runs in this study. In addition, the video image obtained by the SONY movie camera was used to assist in identifying the individual overtopping event. The zero-up crossing method was then applied to find the individual height and period of each wave component corresponding to the overtopping volume recorded in the collection tank with known dimensions. However, only the last 10 waves recorded at P1 and P4 were selected for analysis in this study, in order to exclude the influence of wave reflection from the seawall.

New Dimensional Analysis

Most empirical equations for predicting wave overtopping are cast in non-dimensional form based on the results of laboratory experiments involving different types of seawall with different wave dissipation arrangements. Among them, the exponential form of $Q = A \exp(-BR_q)$ (Owen 1980) and $Q = AR_q^{-B}$ (Aminti and Franco 1988) are two of the most well used formulae, in which parameter R_q includes wave height and freeboard.

In order to consider the effect of the berm width (C_b), a new non-dimensional form is derived in this study. Applying the π -theorem to the physical quantities with relevant dimensions in SI units, such as overtopping rate per unit width q [L^2/T], freeboard R_c [L], mean wave height H_m [L], mean wave period T_m [T], berm width C_b [L], and acceleration due to gravity g [L/T^2]; a new non-dimensional relationship $Q = AR_q^B$ is derived using $Q = q/(2gH_m^3)^{1/2}$ with $R_q = T_m(g/C_b)^{1/2}H_m/R_c$ or $R_q = T_m(gC_b)^{1/2}/R_c$ to present the laboratory results obtained in this study.

Comparison on Empirical Overtopping Formulae

First, the experimental results (Lee 2013a) are fitted to the non-dimensional wave overtopping relationship $Q = A \exp(-BR_q)$ proposed by Owen (1980), where $Q = q/T_m g H_s$ and $R_q = R_c/T_m (g H_s)^{1/2}$; in which T_m and H_s are evaluated at the toe (gage P4 in Fig. 4) and the dimension for q is [$m^3/s/m$]. It is worth noting that R_q does not include the effect of berm on a seawall. Judging from the results depicted in Fig. 7, and the coefficient A and B obtained from regression analysis with the goodness-of-fit R^2 shown in Table 1, Owen's relationship seems fitting better with the data of types 3 and 4 (with blocks in single and double layers on the front slope, respectively) in this study. However, data scattering is obvious; despite the R^2 values are acceptable. On the other hand, fitting the data to the first expression of $R_q = T_m(g/C_b)^{1/2}H_m/R_c$ derived in this study renders Fig. 8, and that to the second expression of $R_q = T_m(gC_b)^{1/2}/R_c$ renders Fig. 9 and Table 2. Upon comparing the graphical results in Figs. 8 and 9, it is apparent that the second expression for R_q is better than the first one, and the values of goodness-of-fit R^2 of the former is also better than that using Owen's relationship. Therefore, this new relationship is selected for further data analysis and discussion.

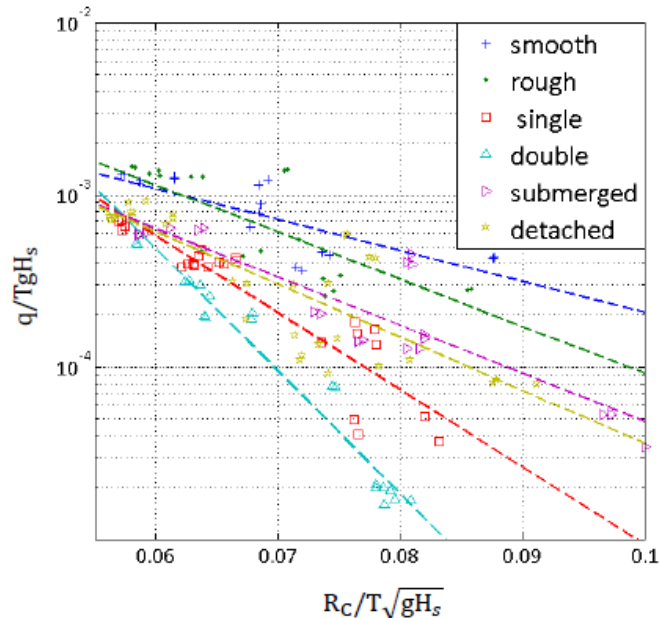


Figure 7. Fitting experimental results using Owen's (1980) non-dimensional Q and R_q .

Table 1. Coefficients A and B and R^2 using Owen's relationship (1980).

Type	A	B	R^2
a. smooth	0.013	41.46	0.67
b. rough	0.050	62.92	0.68
c. single	0.278	103.01	0.84
d. double	8.973	163.80	0.96
e. submerged	0.042	70.63	0.70
f. detached	0.031	64.69	0.86

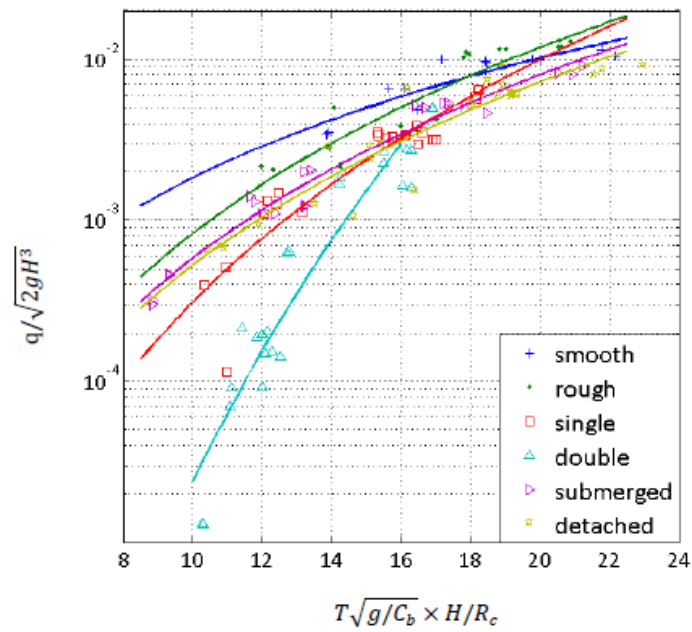


Figure 8. Fitting experimental results using new non-dimensional Q and R_q , in which $R_q = T(g/C_b)^{1/2} H/R_c$.

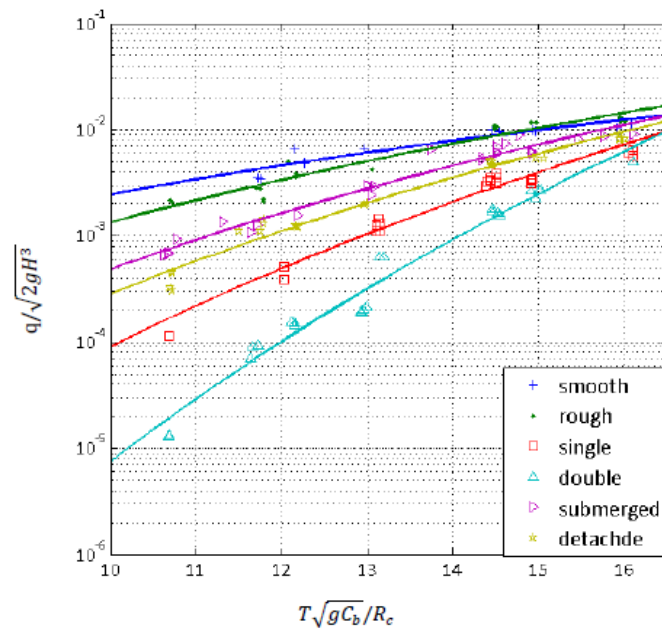


Figure 9. Fitting experimental results using new non-dimensional Q and R_q , in which $R_q = T(gC_b)^{1/2}/R_c$.

Table 2. Coefficients A and B and R^2 using new relationship derived in this study.

Type	A	B	R^2
a. smooth	9×10^{-7}	3.46	0.88
b. rough	1×10^{-8}	5.06	0.93
c. single	4×10^{-14}	9.35	0.98
d. double	3×10^{-20}	14.34	0.97
e. submerged	1×10^{-10}	6.66	0.96
f. detached	9×10^{-12}	7.50	0.97

DISCUSSIONS

Effect of Wave Dissipation Arrangements on Overtopping Rate

Overall, the dimensional individual overtopping rate q is proportional to $H^{3/2}$ and T^B (Fig. 9) where coefficient B is positive number (Table 2), but inversely proportional to $(R_c)^B$, for a given seawall with known berm width C_b . Hence, with the same incident wave condition to a seawall, the greater the freeboard R_c , the less the non-dimensional mean overtopping rate Q or dimensional q . Figure 9 also indicates the effect of wave energy dissipation on the overtopping rate received during laboratory experiments, in which the magnitude in Q in descending order is: type 1 (smooth surface), type 2 (rough surface), type 5 (submerged breakwater), type 6 (emergent detached breakwater), type 3 (blocks in single layer), and type 4 (blocks in double layer), respectively. Moreover, the relative magnitude in Q in each type relative to the basic type can be estimated from the same figure. This implies that blocks in double layer on the front slope (type 4) was the best in reducing overtopping than other types, while type 3 (blocks in single layer) being the second best, with types 6 (emergent detached breakwater) and 5 (submerged breakwater) in the third and fourth place, respectively, at least from the laboratory results shown in the present study. Similar results are depicted in Fig. 10 using the ratio of overtopping rate against that of the basic type (type 1). Hence, the most effective mechanism to reduce overtopping is to place concrete blocks directly on the sloping front of a seawall, with extra benefit in reducing wave energy impact on the structure for its safety. From the result shown, the percentage of reduction in Q ranges from 99% to 50% (from Q/Q_{smooth} of 2%~50% on the ordinate of Fig. 10) on the abscissa of $R_q = T_m(gC_b)^{1/2}/R_c$ from 11 to 16 for type 4 (double layers on front slope) comparing with that of type 1 (smooth front slope); while Q/Q_{smooth} varies from 17% to 77% for type 6 (detached breakwater) within the same range of R_q , or in 73% to 23% reduction in Q relative to that in type 1.

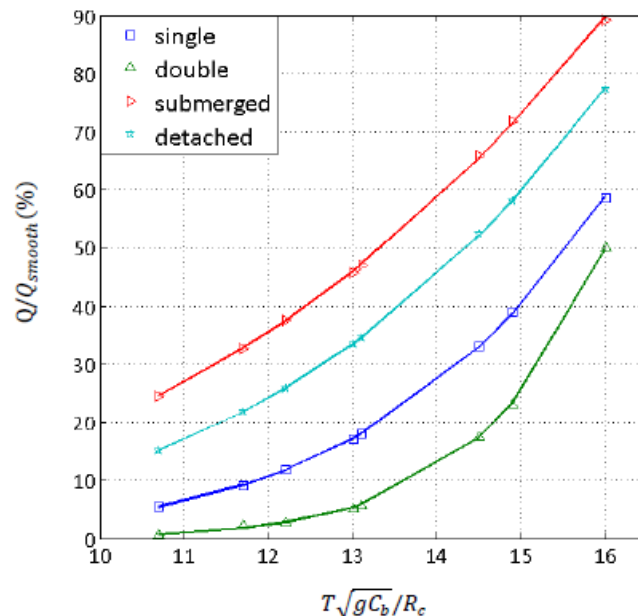


Figure 10. Ratio of non-dimensional Q in types 3–6 versus that in type 1 (smooth surface).

Effect of Water Level on Overtopping Rate

In the laboratory study, model incident wave heights representing the magnitude of the storm waves associated with four kinds of recurrent period (in year) on the HWL and HHWL were generated, and the overtopping rates also recorded. Upon fitting the results using the newly derived non-dimensional parameters Q and R_q (Fig. 11), a consistent trend in data distribution can be observed. This figure implies that the average non-dimensional value of $q/(2gH^3)^{1/2}$ during the HHWL was 2.79 times of that during the HWL, calculated from the values of 2.75, 2.91, 2.67 and 2.81 for storm recurrent period of 10, 20, 25 and 50 years, respectively. These four values of ratios are calculated from the non-dimensional Q in type 3 (c: single layer) being 8.68, 5.56, 4.51 and 3.29 times that in type 4 (d: double layers) at HWL for storm recurrent period of 10, 20, 25 and 50 years, respectively; together with 3.16, 1.91, 1.69 and 1.17 times, respectively at HHWL. Thus, the two sets of numbers that lead to the Q ratios in HWL and HHWL as 2.75 (e.g., being 8.68/3.16), 2.91 (from 5.56/1.91), 2.67 (from 4.51/1.69) and 2.81 (from 3.29/1.17), respectively (Lee 2013a). Further examination

reveals that the Q value for storm wave in 50-year current period was almost equal to that for storm in 10-year recurrent period at HWL.

Furthermore, the results for in Q for types 5 and 6 (submerged and emergent breakwater, respectively) at HWL and HHWL are depicted in Fig. 12. The values in this figure suggest that the average non-dimensional value of $q/(2gH^3)^{1/2}$ during the HHWL was 1.37 times of that during HWL, calculated from the values of 1.49, 1.38, 1.35 and 1.26 for storm recurrent period of 10, 20, 25 and 50 years, respectively. Similarly, the non-dimensional Q in type 5 (submerged) was 1.62, 1.50, 1.45 and 1.37 times that in type 6 (detached) at HWL for storm recurrent period of 10, 20, 25 and 50 years, respectively; while these values were 1.36, 1.25, 1.24 and 1.15 times, respectively at HHWL (Lee 2013a)

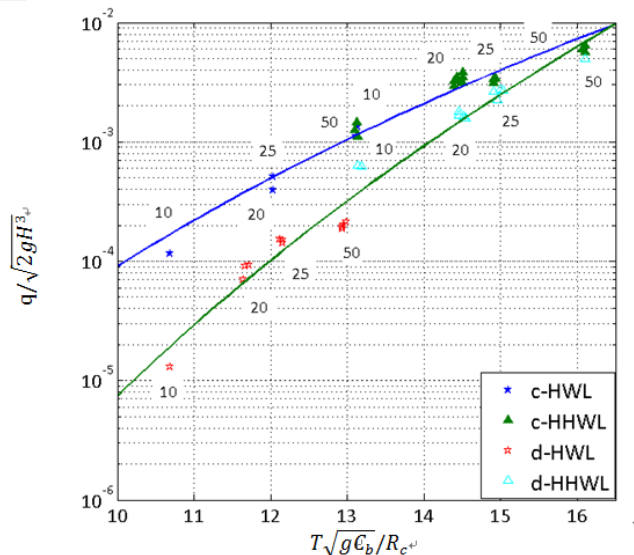


Figure 11. Non-dimensional Q and R_q for type 3 (c: single layer) and type 4 (d: double layer), with digits showing storm recurrent period (Year) within the graph.

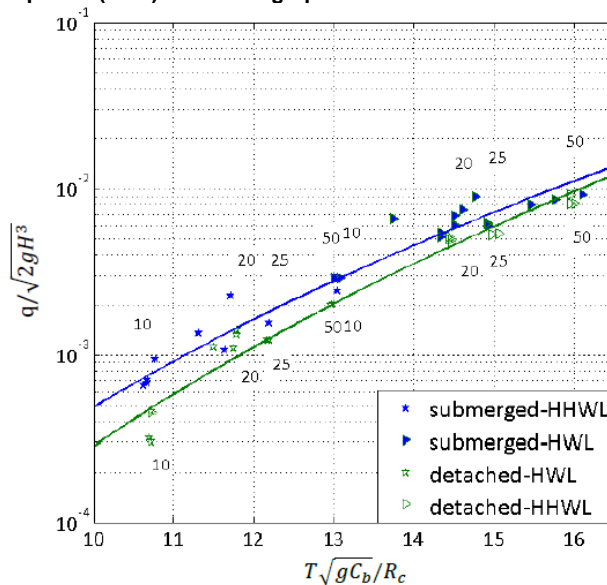


Figure 12. Non-dimensional Q and R_q for type 5 (submerged) and type 6 (detached), with digits showing storm recurrent period (Year).

NUMERICAL METHOD USING NEURAL NETWORK

NN_Overtopping method

Numerical calculations used in this study included Flow3D, CADMAS-Surf (Isobe et al. 2001) and neural network (e.g., NN_Overtopping Manual 2005). Both Flow3D (Lee 2013b) and CADMAS-Surf

provide video images suitable for graphical display, while NN_Overtopping returned several statistical quantiles of overtopping rates through interrogation of the CLASH database (CLASH 2001; EurOtop Manual 2007). The term “CLASH” represents the complex acronym for “Crest Level Assessment for coastal Structures by full scale monitoring, neural network prediction and Hazard analysis on permissible wave overtopping”. Only the results produced by NN_Overtopping (Coeveld et al. 2005) are reported in this paper for the discussion on determining building setback distance behind the seawall.

Prior to applying the NN_Overtopping, the Spanish Coastal Modelling System (SMC; González et al. 2007) was employed to calculate the wave conditions from deepwater to the toe of seawall at -8 m depth, using design storm waves associated with four recurrent periods (10, 25, 50, 100 years). An input file called “NN_Overtopping.inp” was first established, which contained 15 hydraulic and structural parameters of the designed seawall cross-section (see Fig. 2), water depth and wave conditions at toe (Fig. 13); or more specifically wave incidence $\beta(^{\circ})$, $h(m)$, $H_{m0,0,toe}(m)$, $T_{m-1,0,toe}(s)$, $h_t(m)$, $h_b(m)$, $B_t(m)$, γ_f , $\cot\alpha_d$, $\cot\alpha_u$, $R_c(m)$, $B(m)$, $h_b(m)$, $\tan\alpha_B$, $A_c(m)$, and $G_c(m)$. In this study, the value for the surface roughness factor γ_f was taken as 0.4 and 0.5 for the 20-tonne Link blocks in tidy arrangement and scattering showing small quarry stones, respectively (see patterns of arrangement in Fig. 3; $\gamma_f = 1$ for smooth surface based in Bruce et al. 2006). While preparing the values of these parameters, the structural parameters were decided from the cross-section of the existing seawall, and the total design water level (DWL) should include the HHWL or HWL with storm surge and anticipated sea level rise.

An example applying the NN_Overtopping presented in this paper is for the conditions: (1) total design water level (DWL) at +4.0 m comprising HHWL with storm surge (0.9 m) and sea level rise (0.5 m); (2) seawall section with blocks scattering on the front face ($\gamma_f = 0.5$); (3) subject to storm waves in $H_{1/10}$ and $T_{1/10}$ at toe in recurrent period of 10, 25, 50 and 100 years. An output file called “NN_Overtopping.lis” was generated, upon running the NN_Overtopping.exe file. This output file had 10 column, in which column 1 stood for the mean overtopping rate q_{mean} ($m^3/s/m$), columns 2~8 for statistical quantiles of the estimated overtopping rate $q_{2.5\%}$, $q_{5\%}$, $q_{25\%}$, $q_{50\%}$, $q_{75\%}$, $q_{95\%}$ and $q_{97.5\%}$, respectively, column 9 represented the sequence number in the input cases (for instance, cases 1~4 for the wave conditions in each storm recurrent periods when $\gamma_f = 0.4$ and cases 5~8 for corresponding values when $\gamma_f = 0.5$), and the last column gave the remark for the calculation in each case. These overtopping rates can be used to assist civil engineers on the design of a suitable drainage system for the new industrial zone on the reclaim land.

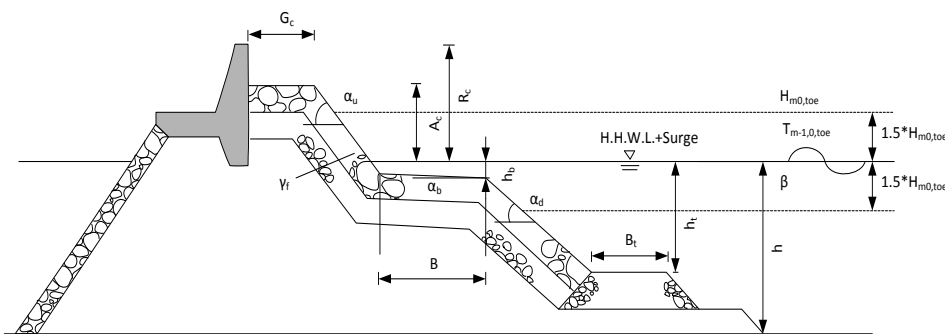


Figure 13. Schematic view of the 15 technical parameters used in NN_Overtopping.inp file (NN_Overtopping Manual 2005).

Building Setback Distance versus Storm Recurrent Period

To assist architect and planner in determining the width of the buffer zone (for landscaping with drainage, trees and roadway etc.) behind the seawall against the onslaught of a specific design storm, two criteria are considered. The first criterion is the limiting mean overtopping rate Q_{mean} ($m^3/s/m$ or liter/s/m) for different types of object (human, vehicles, property and structures) and the target in each type (human- pedestrians; vehicles- low or moderate speed; property- small or large yachts, building and facilities; structures- seawall/dikes, promenade/road), as well as the hazard or damage associated with each target (EurOtop Manual 2007). For example, $Q_{mean} = 1$ liter/s/m is recommended for damage to building (type: property; target: building), and $Q_{mean} = 50\text{--}200$ liter/s/m for no damage to protected seawall crest/rear slope (type: structures; target: seawall/dike). The second criterion is to assume a

linear decay in overtopping rate from the seawall crest to the building at distance x (m), such that $Q_{\text{building}} = Q_{\text{crest}} / x$. Taking the limiting mean $Q_{\text{building}} = 1$ (liter/s/m) for a building at $x = 50$ m from the seawall, then the allowable $Q_{\text{crest}} = Q_{\text{building}} \cdot x = 50$ liter/s/m = $0.05 \text{ m}^3/\text{s/m}$. This specific value of overtopping rate can be marked using a horizontal line (the dash-dot line in Fig. 14) on a graph showing Q ($\text{m}^3/\text{s/m}$) versus storm recurrent period (year). Similarly, the maximum allowable overtopping rate at seawall crest of $Q_{\text{mean}} = 200$ (liter/s/m) = $0.2 \text{ m}^3/\text{s/m}$ is also indicated in the Fig. 14 (dashed line).

By joining the output of Q_{mean} , $Q_{75\%}$, $Q_{95\%}$ and $Q_{97.5\%}$, respectively, for each of the storm recurrent period of 10, 25, 50 and 100 years (Fig. 14), and taking the allowable Q_{crest} of 0.05 and $0.2 \text{ m}^3/\text{s/m}$ marked on Fig. 14, the following observations can be stated that, with $H_{1/10}$ (m) waves and building setback at $x = 50$ m:

- (1) Building will be safe with storm in 10-year recurrent period for all $Q < Q_{97.5\%}$;
- (2) Building will be safe with storm in 25-year recurrent period for Q_{mean} to $Q_{95\%}$, unsafe for $Q > Q_{97.5\%}$;
- (3) Building will be safe with storm in 50-year recurrent period for Q_{mean} to $Q_{85\%}$, unsafe for $Q > Q_{85\%}$;
- (4) Building will be safe with storm in 100-year recurrent period for Q_{mean} to $Q_{75\%}$, unsafe for $Q > Q_{75\%}$;
- (5) Seawall crest may become unsafe when storm recurrent period greater than 75 years, because $Q > 0.2 \text{ m}^3/\text{s/m}$.

Once overtopping rate is estimated, provision of suitable drainage system is recommended. Furthermore, different results for different scenarios (such as different input wave conditions, roughness factor, building setback distance proposed and different target to be protected) should also be investigated for comparison.

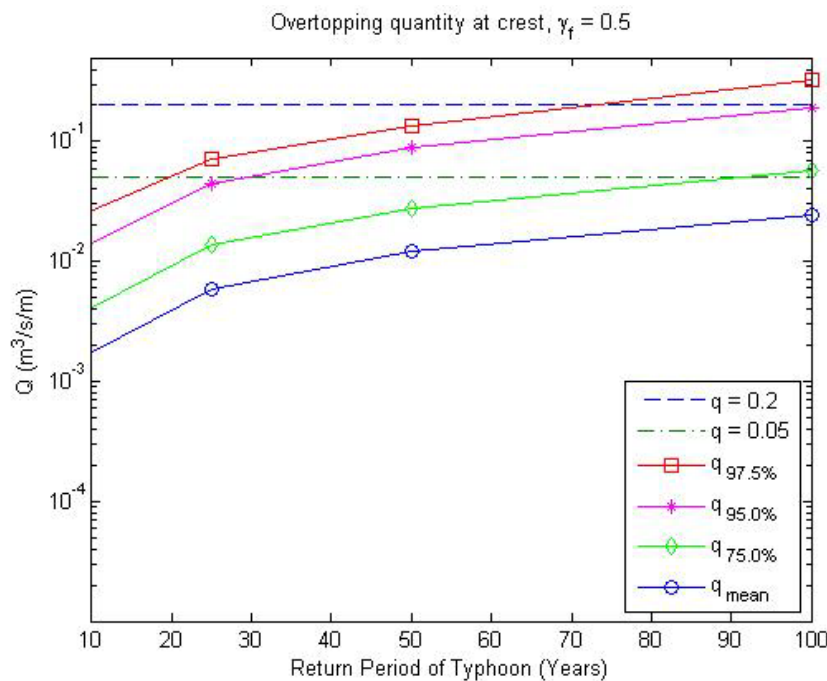


Figure 14. Dimensional overtopping rate versus storm recurrent period, showing also specific quantiles of overtopping rate from NN_Overtopping and allowable overtopping rate for target to be protected.

CONCLUDING REMARKS

Remarks arising from the results of laboratory experiments and numerical methods carried out for this study are as follows:

- (1) Laboratory experiments, empirical formulae (EurOtop Manual 2007) and neural network method (e.g., NN_Overtopping 2005) have been widely used in the research on wave run-up and overtopping. However, laboratory experiments are often costly and time consuming, as well as being subject to scale effect; while the latter two approaches have become popular in recent years for its convenience in set-up and improved accuracy if applied appropriately and correctly.

- (2) The results arising from the present laboratory experiments show that blocks in double layer on the front slope (type 4) was the best in reducing overtopping than other 5 types, while type 3 (blocks in single layer) being the second best, with types 6 (emergent detached breakwater) and 5 (submerged breakwater) in the third and fourth place, respectively; while rough surface (type 2) and smooth surface (type 1) were not effective for reducing wave run-up and overtopping.
- (3) There are several non-dimensional or dimensional formulae available for estimating overtopping quantities, derived mostly from laboratory data on seawalls in different forms, regular or irregular waves of breaking or non-breaking, as well as with or without armoring blocks in various kinds to dissipate wave energy. Care has to be taken to verify the desired accuracy before applications.
- (4) A new non-dimensional expression that involves the berm width (C_b) for a berm seawall is derived for the non-dimensional freeboard parameter R_q , such as $R_q = T_m(g/C_b)^{1/2}H_m/R_c$ or $R_q = T_m(gC_b)^{1/2}/R_c$. Together with non-dimensional overtopping rate $Q = q/(2gH_m^3)^{1/2}$, they fit well the new expression of overtopping $Q = AR_q^b$ with the results obtained in the present laboratory experiments.
- (5) Example for applying the results of NN_Overtopping is given for determining the building setback distance using limiting overtopping rates acceptable (EurOtop Manual 2007), for storm waves associated with different storm recurrent periods (of 10, 25, 50 and 100 years) and different statistical quantiles (such as $q_{2.5\%}$, $q_{5\%}$, $q_{25\%}$, $q_{50\%}$, $q_{75\%}$, $q_{95\%}$ and $q_{97.5\%}$), in order to assist in the design of drainage system in the lee of a seawall.

ACKNOWLEDGMENTS

The authors acknowledge the research funds to JRCH under grant NSC 101-2221-E-110-02-, National Science Council, Republic of China.

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