Study wave overtopping on the wave flume at WRU

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Abstract: At present design of sea dike is done based on wave run up creteria, but in reality sea dike is slided and/or breached caused wave overtopping discharge during typhoon or monsoon. The paper presents testing results implemented on the wave flume at WRU in order to determine wave overtopping standard for the sea dike without crownwall. Tests show that formula and graphs developed by TAW (2002) can be used for Vietnam sea dike design according to wave overtopping standard.

1. Introduction

At present, the sea dike design in Vietnam is carried out with application of guideline code 14TCN130 – 2002, in which dike crest level is computed according to wave run up criteria. The formula is as following:

$$H_{\text{đđp}} = H_{\text{trp}} + h_{\text{nd}} + h_{\text{sl}} + a \qquad (1)$$

In which:

$H_{\text{đđp}}$: dike crest level (m)

H_{trp}: design water level with frequency p

h_{nd}: design storm surge with frequency p

h_{sl}: Wave run up on the dike slope

a: Additional height to increase safety level

The formula (1) has still shortcomings as bellows:

- Design frequency does not the same for each components in the formula, for example water level and wave run up is computed with 1/20 years (p =5%) while storm surge of 1/5 years.

- Design water level: water level series consists of yearly maximum events are chosen at tidal station. A probability distribution function is selected to formulate the change of water level (usually used Gumbel or Pearson distribution) and frequency curve is drawn to determine design water level. If tidal station is not available at constructed location, then it is computed by correlating with nearby tidal station. With above computed procedure, storm surge is included in water level and consequently storm surge is double mention in the formula (1)

- In formula (1) dike crest level is computed using wave run up standard ($R_{u2\%}$). It means that overtopping is only allowed 2 waves above 100 waves averagely. However at the exposed sea dike such as Cat Hai, sea dike No.1 in Hai Phong province, Hai Hau and Giao Thuy in Nam Dinh province is always overtopped when typhoons hitting, even low level combination between tides and typhoons or monsoon lasting more than 7 days. Dikes in above mention locations were breached during the typhoon in 2005. The main reason of dike breaching is due to low design frequency.

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Wave overtopping caused inner slope erosion in typhoon no. 7/2005 at Cat Hai (Hai Phong)

Sea dike no.3 (Tiên lãng, Hải Phòng) was broken due to typhoon no. 7/2005

Co Vay (Nam Định) dike after typhoon 7/2005

Thịnh Long (Nam Định) revetment after typhoon 7/2005

Hau Loc dike (Thanh Hoa) after typhoon 7/2005

Hoang Hoa dike (Thanh Hoa) after typhoon 7/2005

Figure 1: Dike breaching after typhoon No. 7/2005 in Hai Phong, Nam Dinh and Thanh Hoa.

- In this conditions, the sea dike is suffered from overtopping should be considered. If allowing overtopping, the dike crest is lower than that in case of wave run up, but methods to strengthen the dikes (it must be stronger in term of loads attached), cover protection, inner toe scour, water released after the typhoon, double dike lines etc. have to be considered carefully in order to avoid dike breaching caused heavily damages of socio-economic activities behind the sea dike.

2. Average overtopping discharge of dike without crownwall

According to TAW (2002) and/or EurOtop (2007), average wave overtopping discharge is determined as following:

For breaking wave: $\gamma_b \xi_{0m} \leq \xi cr \approx 2.0$:

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0.067}{\sqrt{\tan\alpha}}.\gamma_b.\xi_{0m}.\exp\left(-4.75.\frac{R_c}{H_{m0}}.\frac{1}{\xi_{0m}}.\frac{1}{\gamma_b\gamma_f\gamma_\beta\gamma_v}\right) \tag{2}$$

For non-breaking wave: $\gamma_b \xi_{0m} > \xi cr \approx 2.0$:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.20.\exp\left(-2.6\frac{R_c}{H_{m0}}\frac{1}{\gamma_f\gamma_\beta}\right) \tag{3}$$

When wave breaking many times on very shallow foreshore: $\xi_{0m} \geq 7.0$:

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{1}{\gamma_f\gamma_\beta}.\frac{R_c}{H_{m0}(0.33+0.022\xi_{0m})} \tag{4}$$

In which q is average wave overtopping discharge (l/s/m)

H_{m0} can be taken approximately as significant wave H_s and be computed from the wave spectrum:

$$H_s \approx H_{m0} = 4\sqrt{m_0} = 4\sqrt{\int_0^\infty S(f)df} \tag{5}$$

where $S(f)$ is wave spectral energy distributed function at certain frequency f and m_0

ξ_{0m} is Irribaren number:

$$\xi_{0m} = \frac{\tan\alpha}{\sqrt{s_m}} \tag{6}$$

S_m is characterized for wave steepness and computed as formulae:

$$s_m = \frac{H_{m0,d}}{L_0} = \frac{2\pi H_{m0,d}}{gT_m^2} \tag{7}$$

Parameters such as γ_b, γ_f, γ_β, γ_v indicate level of affecting of berm, wave incident direction, dike slope, types of revetment and materials and sea crownwall (not apply in this study case).

tanα is dike slope (slope 1:3 up to 1:5 usually is applied for Vietnam sea dike)

R_c freeboard height computed from still water level to dike crest

In above formulae, wave parameters are considered at dike toe. Based on TAW (2002), The reliability of equation (2) is described by taking the coefficients 4.75 as normally distributed stochastic parameters with means of 4.75 and standard deviations $\sigma = 0.5$ (these values are determined based on testing data on wave overtopping) and 5% upper and lower exceedance curves will be taken coefficients 5.57 and 3.93. Similarly the reliability of equation (3) is described by taking the coefficients 2.6 as normally distributed stochastic parameters with means of 2.6 and standard deviations $\sigma = 0.35$ and 5% upper and lower exceedance curves will be taken coefficients 3.17 and 2.03. With the purpose to increase level of safety, TAW (2002) suggested to use coefficients 4.3 instead of 4.75 in equation (2) and 2.3 to replace 2.6 in equation (3).

3. Testing results on the wave flume at WRU

3.1. Wave flume

Wave flume used for the testing was developed and manufactured DELFT (W|L DELFT HYDRAULICS) and transfer to WRU under the project "Upgrading training Capacity in Coastal Engineering at WRU". Its dimensions are 54m long, 1.2m height and 1.0m width. Very modern and accurate observed equipment is set up. Random wave with JONSWAP spectrum of maximum wave height of 0.3m and period of 3.0s can be generated. Data can be collected and analyzed automatically.

3.2. Foreshore and sea dike cross section

The crest level including crownwall is 4.0 to 5.5 m above MSL. Freeboard (Vertical distance between still water level and dike crest) is mostly 1.5m to 2.5m. Dike slope is 1:3 to 1:4.

Ground elevation of foreshore is changed significantly in space. At serious eroded sections, it is very short foreshore and water depth is greater than 5m to 6m during typhoons, at other locations foreshore slope is rather gentle with maximum depth up to 4m in typhoon condition.

Based on existing cross section and its composition, model on the flume has 75cm high and outer slope 1:3. In order to minimize other negative affects caused by small scale model (for example it could be not simulated roughness correctly in the small model), smooth and waterproof slope is applied. In fact, the affect of roughness is tested accurately only on the large scale model or field testing and the role of roughness is characterized by reduction factor γ_f.

The length of foreshore on the model is FLL = 6.5m for the case of a real short foreshore (less than and equal a wavelength in deep water L_0) and FLL = 12.0m for larger foreshore ($2L_0$). In each testing case of foreshore length, vertical distance is tested is FLH = 0.25m, 0.30m and 0.35m respectively. Slope of transition section between dike slope and foreshore is 1:20.

3.3. Model layout and testing parameters

Model layout for wave overtopping is shown in Figure 1 in which interfaces between dike and flume and dike and foreshore are specially treated by using waterproof materials to minimize seepage through dike body to the reservoir collecting overtopping water. 5 waves observed gauges were set along the flume such as dike toe, middle foreshore and other end of foreshore etc. to collect incident wave parameters. Signals from wave gauges are transmitted and stored in the computer. Overtopping water is kept in the tank behind the dike and re-pumping to the flume and to determine total volume of overtopping water in each testing.

In the testing, random wave with normal JONSWAP spectrum is generated. This type of wave spectrum is fitted with wind wave in East sea. The incident wave height generated by wave flume is 0.10m, 0.15m and 0.20m respectively. Wave spectral period for above wave height is T_p = 1.5s, 1.7s, and 2.0s.

In order to generate more wave pattern, water depths of 0.55m and 0.60m is also applied in the testing. With combination of all above conditions, 168 testing are implemented. All conditions including foreshores, wave conditions, water level are listed in table 1. Duration of each testing is at least $1000T_p$ (1000 waves) in order to ensure fully band of waves occurring in the East sea.

3.4. Testing procedure and observed parameters

Basic procedures applied in the testing are as following:

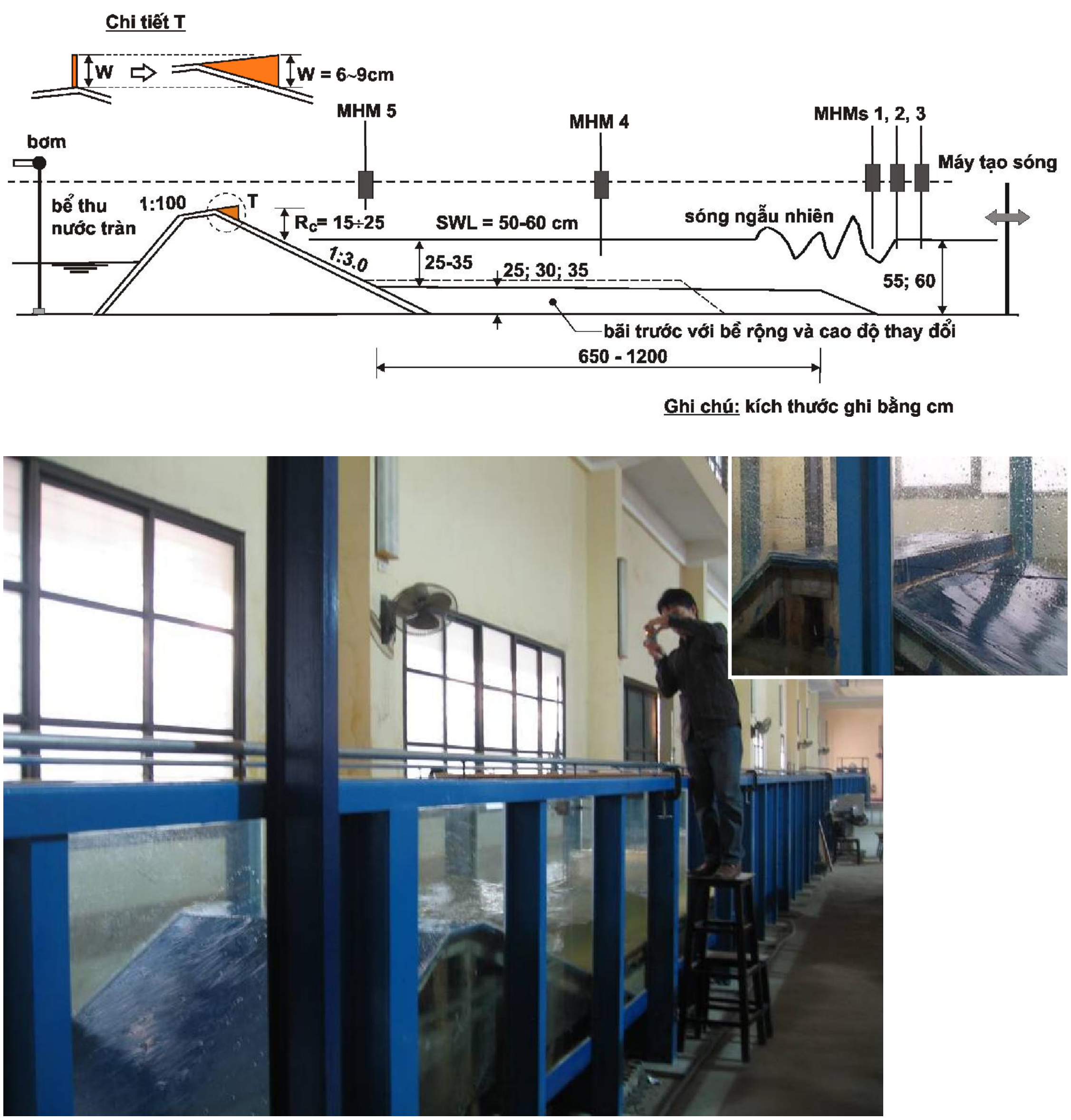


Figure 2. General layout of wave overtopping testing

- Testing preparation: wave flume cleaning periodically, put water up to testing levels (0.55m and 0.60m), positioning of wave observed gauges and space parts and connect them to the computer
- Calibration of wave gauges according to water temperature and other constitutes in the water.
- To boot wave generator according to controlled programme.
- To start storing data when waves generated in the flume to reach stable conditions (it is about 5 minutes)
- To measure water volume in overtopping reservoir and pump back to the flume
- To finish testing when testing time is long enough
- Measure water depth in the flume after each test.

Table 1: All conditions applied in the wave overtopping testing (non crownwall)

Foreshore (Length FLL, Height FLH)	**Wave conditions (Hs, Tp)**	**Water depth (D) in the flume**
02FLL × 04FLH = (6.5m,12m) × (0.25m; 0.30m; 0.35m)	03Hs × 03 Tp = (0.10m; 0.15m; 0.20m) × (1.5s; 1.7s; 2.0s)	02D = (0.50m; 0.60m)

Total tests: 168 with combination of conditions in table 1

- Determine total overtopping water and testing duration
- Check logically data collected during the test stored in the computer
- Writing down testing diary

3.5. Testing results on wave overtopping with non crownwall dike in wave flume

72 tests in case of non crownwall dike are analyzed for non breaking and breaking wave. Observed data are put into dimensionless graphs of TAW in order to compare with that implemented by TAW (2002).

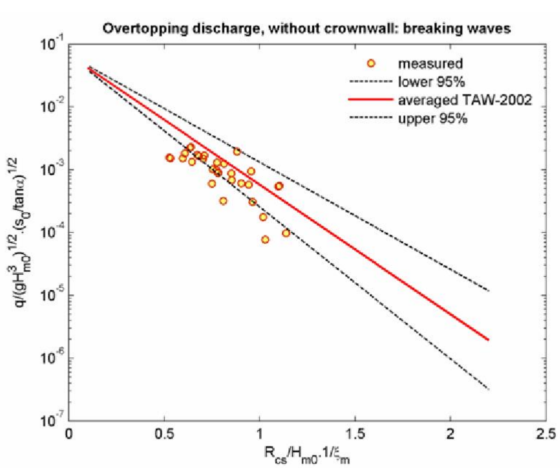


Figure 3: Overtopping discharge without crownwall, breaking waves using $T_{m-1,0}$

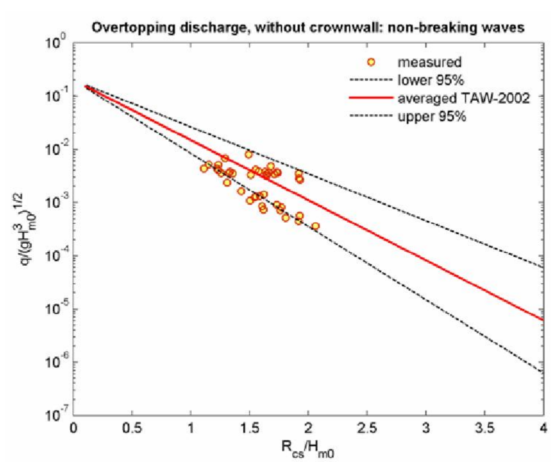


Figure 4: Overtopping discharge without crownwall, Non- breaking waves using $T_{m-1,0}$

In case of breaking waves:

Vertical axis (loga scale): dimensionless overtopping discharge $\frac{q}{\sqrt{gH_{m0}^3}}\sqrt{\frac{s_0}{\tan\alpha}}$

Horizontal axis: dimensionless freeboard $\frac{R_c}{H_{m0}}\frac{1}{\xi_{0m}}$

In case of non-breaking waves:

Vertical axis (loga scale): dimensionless overtopping discharge $\frac{q}{\sqrt{gH_{m0}^3}}$

Horizontal axis: dimensionless freeboard $\frac{R_c}{H_{m0}}$

Figures 3 and 4 compare observed data on the wave flume at WRU and TAW (2002) graphs in which wave period $T_{m-1,0}$ is taken at dike toe.

As discussed before, $T_{m-1,0}$ is taken at dike toe to replace T_p at deep water in wave overtopping computation, but $T_{m-1,0}$ is unstable depend on morphological and wave conditions at certain field sites. That's why TAW (2002) suggest to use the relationship $T_p = 1.10T_{m-1,0}$ for using deep water wave parameters in computation.

Figure 4 shows testing result on relationship between T_p and $T_{m-1,0}$ at dike toe, middle and other end foreshore. It can be concluded that $T_{m-1,0}$ tends to increase when incident wave coming to dike (from deep to shallow water) and at the dike toe the equation $T_p = 1.15\ T_{m-1,0}$ can be applied for computation.

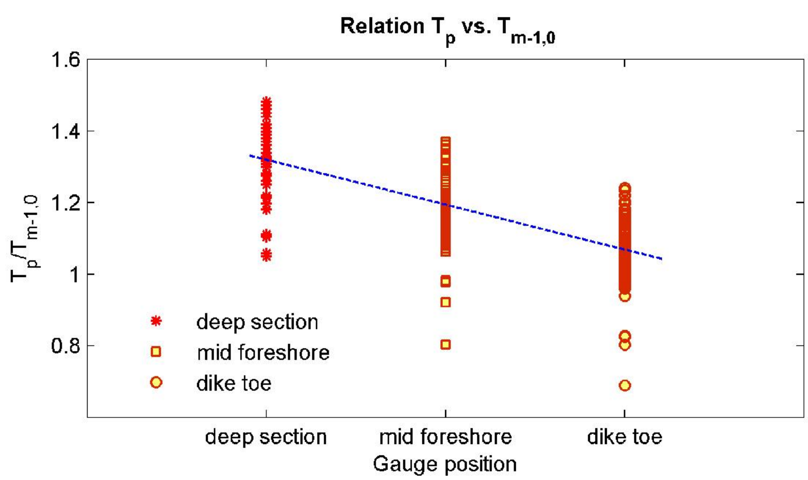
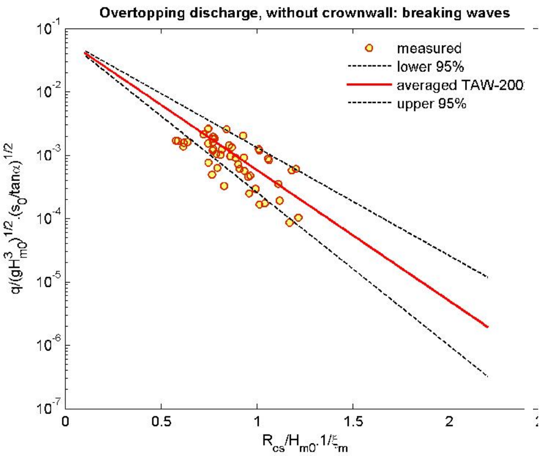


Figure 5: Relationship between T_p and $T_{m-1,0}$ at dike toe, middle and other end foreshore

Testing results using the equation of $T_p = 1.15\ T_{m-1,0}$ replaced $T_{m-1,0}$ are re-drawn in the graphs of TAW in Figures 6 and 7. It shows that results that using $T_{m-1,0}$ and $T_p = 1.15\ T_{m-1,0}$ are similar. Finally it can say that methods to compute wave overtopping discharge done by TAW (2002) can be applied for Vietnam conditions.



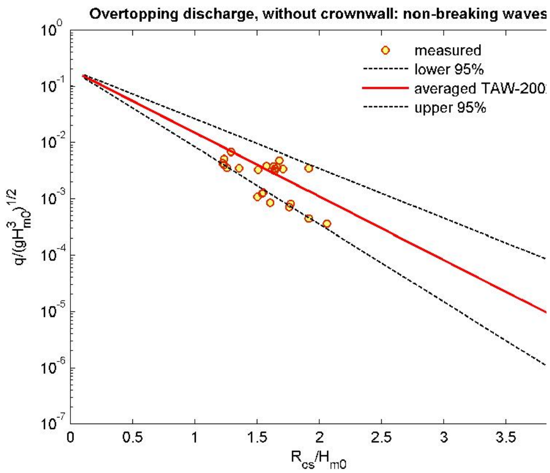


Figure 6: Overtopping discharge without crownwall, breaking waves using $T_{m-1,0} = T_p/1.15$

Figure 7: Overtopping discharge without crownwall, Non- breaking waves, $T_{m-1,0} = T_p/1.15$

4. Conclusion

- 72 tests in dike without crownwall show that they are fitted with results done by TAW (2002). It means that we can use equations and graphs developed by TAW in order to calculate freeboard with different values of wave overtopping discharge.

- Tests implemented in case of wave overtopping on smooth slopes which can generate maximum volume of wave overtopping. However, in reality dike can be constructed with different types of material, protected by concrete, grass etc. Incident waves change significantly in time and space. All must be taken into account for economically computation of dike cross section.

References

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