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# Correction of structure geometry design of IRAN-LNG Breakwater based on physical modeling results

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**Key words:** Toe berm stability, Physical modeling, IRAN-LNG Breakwater


## Abstract:

In this paper the physical modeling investigations on IRAN-LNG main breakwater sections which resulted to correction of the design of toe berm geometry and weight range are presented.
Two types of breakwater section with only difference in toe berm part were tested in order to finding out and choosing more stable toe berm.
The geometrical scale factor of 1:35 was selected. Three wave conditions as yearly, 50 years and 100 years return periods with three water levels have been considered. Totally 34 tests were performed in order to carry out the stability of Armour layer Xbloc® units and toe berm stability and also wave run-up heights and Overtopping rates in SCWMRI laboratory. Results have been presented by profiles, photographs and related values.


## Introduction:

The LNG Plant is located on the south-western coast of Persian Gulf in Iran, at Tombak, approximately 40 kilometers from Assaluyeh, adjacent to the Boushehr port. The LNG Plant is located at the west end of the area where three other LNG plants are planned to be constructed. For IRAN-LNG Port Project, Iranian Ministry of Oil, as main client of the project has asked SCWMRI to realize a two dimensional physical modeling study of the breakwater sections.
In this study two important points of: Hydraulic stability and Hydraulic responses of breakwater section were investigated.
The breakwater section, hereafter known as the critical section of main breakwater of IRAN-LNG port, is composed of two type of materials, one at the Armour layer as Xbloc® (artificial Armour unit) and the other at the filter layer, toe berm (and toe berm under layer), core and mattress of the structure as Armour stones with specific grading and weight range.
Two types of breakwater section with only difference in toe berm part have been presented by designer in order to finding out and choosing more stable toe berm (one of them will construct in the place of section B8-B8). The first one has higher elevation and placed on extended filter layer which has been named B8-B8-A (Figure 1) and the second one has lower elevation that placed on a heavy under layer named B8-B8-B in this paper (Figure 2). The differences between the breakwater section B8-B8-B and the previous B8-B8-A is the additional new designed toe Berm under layer and both level and thickness reduced related to the main toe Berm. This toe

Berm under layer and main toe Berm are composed of two types of materials, one at the toe Berm under layer (1 to 3 ton stones) and Toe Berm (3 to 5 ton stones) respectively. The comparison of two sections is plotted in Figure 3.

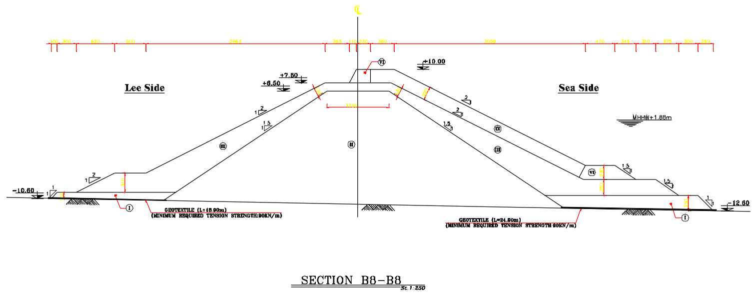


**Fig. 1)IRAN-LNG Port breakwater section (B8-B8-A).**

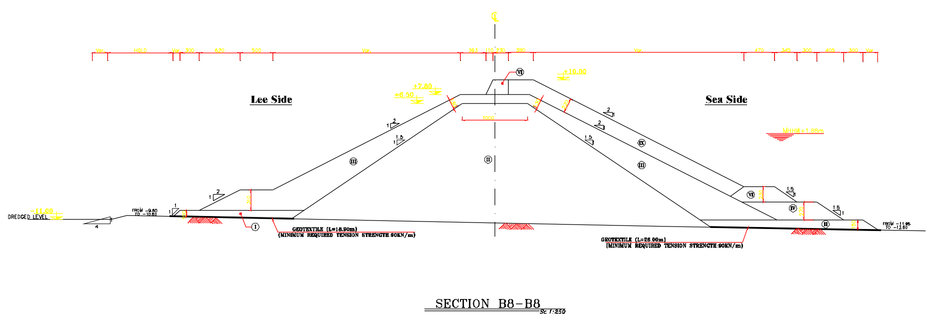


**Fig. 2) IRAN-LNG Port breakwater section (B8-B8-B).**

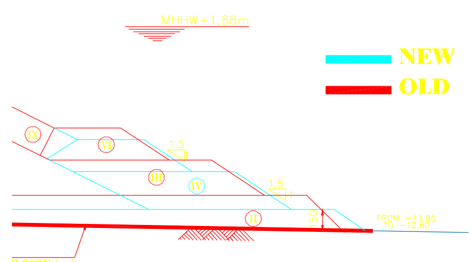


**Fig. 3) Comparison between new (B8-B8-B) and previous (B8-B8-A) breakwater sections.**

**Materials and design condition**

The core and other layers material weight ranges and related grading for section B8-B8-A are presented in Table 1. The added Layer (IV) material weight ranges and related grading for section B8-B8-B are presented in Table 2.

**Table 1. Materials Weight ranges and grading for section B8-B8-A.**

| Layer | Weight | Specification |
|---|---|---|
| I | 0.1 – 20 Kg | 50% > 10 Kg |
| II | 1 – 200 Kg | 50% > 100 Kg |
| III | 0.2 – 1 ton | 50% > 650 Kg |
| VI | 3 – 5 ton | 50% > 4000 Kg |
| IX | Xbloc® | 4m3 |

**Table 2. Materials Weight ranges and grading for section B8-B8-B.**

| Layer | Weight | Specification |
|---|---|---|
| IV | 1 – 3 ton | 50% > 2000 Kg |

The wave conditions which were considered for model tests are presented in Table 3. Storm duration for all conditions is 6 hours. The water levels at toe of structure presented in Table 4.

**Table 3. Design wave conditions.**

| Return Period | Significant Wave height | Peak Wave Period | Angle of incidence |
|---|---|---|---|
| (Year) | Hs (meter) | Tp (second) | (degree) |
| 1 | 1.92 | 7.9 | 247.5 |
| 50 | 4.57 | 11 | 225 |
| 100 | 5.35 | 11.8 | 225 |

**Table 4. Water levels**

| Water Elevation | Level |
|---|---|
| | (Meter related to C.D.) |
| MLLW | 0.41 |
| MSL | 1.23 |
| MHHW | 1.88 |
| DWL | 2.57 |

**Model Scale**

According to the dominant rules in physical modeling and the designing conditions and also laboratory limitations, the appropriate scale was selected. Therefore the model's section has been designed and implemented in the wave's flume. The basis of all physical modeling is the idea that the model behaves in a manner similar to the prototype it is intended to emulate. Ideally, a properly designed laboratory model should behave in all respects like a controlled version of the prototype. Completely similar models are models in which the values of all relevant dimensionless parameters in the prototype are maintained in the model. A prerequisite for complete similarity is that the model be geometrically similar to the prototype. Other types of model are kinematically similar models and dynamically similar models. Correspondence between prototype and model parameters is denoted by the scale ratio or simply the scale (see Hughes, Steven E. 1993).

The inertial nature of the determining phenomena in a hydraulic study of this type of structure (subjected to wave action) made it necessary to choose Froude's hydraulic similarity. The choice of geometric reduction scale was guided mainly by:

· The need to obtain satisfactory hydraulic and hydrodynamic similarity,
· The dimensions and performance of the wave flume.

Thus by considering the abovementioned rules the 1:35 scale was chosen.

Method for compensating for the increased buoyancy of salt water relative to the fresh water used in most scale models is to adjust weight of model Armour units. The scaling requirement is based on preserving the value of a "Stability Number" between prototype and model. The ratio of stability number based on Hudson formula in prototype and model should be unit in order to simulation confirmation as Equation 1.

$$\frac{W_m}{W_p} = \frac{1}{\lambda^3}\frac{\rho_{am}}{\rho_{ap}}\frac{\left(\frac{\rho_{ap}}{\rho_{wp}}-1\right)^3}{\left(\frac{\rho_{am}}{\rho_{wm}}-1\right)^3} \qquad (1)$$

Based on weight ranges of materials that have been presented in Tables 1 and 2, both I and II sections materials have been modeled by size of materials geometrically. By using of Equation 1 and considering 1:35 scale, the nominal diameters of core and filter layer materials are presented in Table 5. Either stone materials of layers III, IV and VI have been modeled and are present in Table 5 too.

**Table 5. Armor stones Diameter and weight ranges in model.**

| Layer | | | | |
|---|---|---|---|---|
| Diameter (mm) | | Weight (gr) | | |
| I | II | III | IV | VI |
| 1.0 – 5.7 | 2.1 – 12.3 | 4 – 20 | 20 - 60 | 60 - 100 |

Geometrical simulation of Xbloc® yield to a nominal diameter (Dn) of 4.54 cm for 9.6 ton ($4m^3$) Xbloc® in prototype. In order to yield Dynamic simultaneous the Equation 1 have been simplified and used as follows:

$$\rho_{am} = \frac{\rho_{ap}}{\rho_{wp}}\rho_{wm} \qquad (2)$$

With considering following data:

$\rho(\text{Xbloc in prototype}) = \rho_{bp} = 2.31\,\text{ton/m}^3$, $\rho_{wm} = 1.0\,\text{ton/m}^3$, $\rho_{wp} = 1.03\,\text{ton/m}^3$ and $\lambda = 35$,

Thus: $\rho_{am} = \frac{2.31}{1.03}(1.00) = 2.24\,\text{ton/m}^3$

By considering volume of Xbloc® in model ($V = 93.29\ cm^3$) and the above mass density, mass of each block was calculated of 209.2 gr. Plot of Xbloc® model is present in Figure 4.

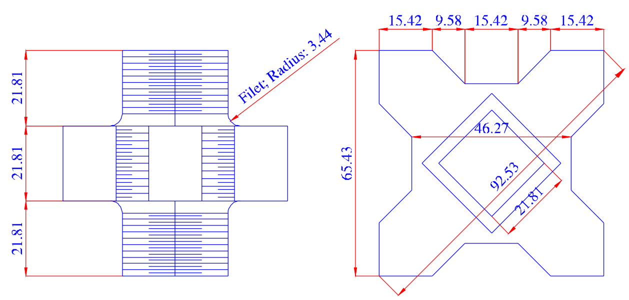


**Fig. 4) Plot of Xbloc® model**

**Tests Procedure**

According to the hydrodynamic conditions considered in the designing process and by considering design and extreme wave heights and the related wave periods and also extreme conditions of the water depth and the storm duration, the hydrodynamic parameters was determined and the test implementation plan was presented.

The basis of presenting physical modeling study is considering 3 wave heights with related periods for 3 water depths with JONSWAP wave energy spectrum that totally first 27 tests have been performed to control the designing of the structure (1000, 2000 and 3000 series tests). Also three wave heights with related periods for 1 water depth (MLLW) have been performed to control the designing of the new designed toe Berm (series tests 4000). Of course 100 and 50 year return period wave heights have been considered as extreme condition and yearly wave height has been considered as operating condition.

Physical model tests final program with related carried out reflection coefficients (Cr), incident significant wave heights (Hso), significant and 2% run-up levels (Rus, Ru2%) and wave overtopping discharges (Qbar) presented in Table 6.

**Table 6. Tests program**

| No. | Test Number | Hsi (m) | Tp (Sec) | Cr | Hso (m) | Water Depth (cm) in model | Rus (m) | Ru2% (m) | Qbar (lit/m/sec) prototype |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 1001 | 1.92 | 7.9 | 0.29 | 0.98 | 37.16 (MLLW) | 0.76 | 1.27 | 0 |
| 2 | 1002 | 1.92 | 7.9 | 0.27 | 1.76 | 37.16 (MLLW) | 1.38 | 2.11 | 0 |
| 3 | 1003 | 1.92 | 7.9 | 0.27 | 1.91 | 37.16 (MLLW) | 1.30 | 2.05 | 0 |
| 4 | 1004 | 1.92 | 7.9 | 0.27 | 1.94 | 37.16 (MLLW) | 1.59 | 2.22 | 0 |
| 5 | 1005 | 1.92 | 7.9 | 0.27 | 1.93 | 37.16 (MLLW) | 1.47 | 2.12 | 0 |
| 6 | 1006 | 1.92 | 7.9 | 0.26 | 1.93 | 39.50 (MSL) | 1.49 | 2.29 | 0 |
| 7 | 1007 | 1.92 | 7.9 | 0.26 | 1.85 | 39.50 (MSL) | 1.57 | 2.32 | 0 |
| 8 | 1008 | 1.92 | 7.9 | 0.26 | 1.92 | 39.50 (MSL) | 1.53 | 2.29 | 0 |
| 9 | 1009 | 1.92 | 7.9 | 0.24 | 1.92 | 43.3 (DWL) | 1.43 | 2.34 | 0 |
| 10 | 1010 | 1.92 | 7.9 | 0.24 | 1.93 | 43.3 (DWL) | 1.50 | 2.42 | 0 |
| 11 | 1011 | 1.92 | 7.9 | 0.24 | 1.93 | 43.3 (DWL) | 1.44 | 2.31 | 0 |
| 12 | 2001 | 4.57 | 11 | 0.33 | 4.87 | 37.16 (MLLW) | 3.87 | 6.18 | 0.411 |
| 13 | 2002 | 4.57 | 11 | 0.33 | 4.91 | 37.16 (MLLW) | 3.82 | 6.15 | 0.369 |
| 14 | 2003 | 4.57 | 11 | 0.33 | 4.88 | 37.16 (MLLW) | 3.99 | 6.37 | 0.269 |
| 15 | 2004 | 4.57 | 11 | 0.33 | 4.77 | 37.16 (MLLW) | 3.62 | 5.92 | 0.153 |
| 16 | 2005 | 4.57 | 11 | 0.33 | 4.84 | 39.50 (MSL) | 3.89 | 6.31 | 0.512 |
| 17 | 2006 | 4.57 | 11 | 0.33 | 4.84 | 39.50 (MSL) | 3.74 | 6.01 | 0.509 |
| 18 | 2007 | 4.57 | 11 | 0.33 | 4.93 | 39.50 (MSL) | 3.84 | 6.47 | 0.553 |
| 19 | 2008 | 4.57 | 11 | 0.34 | 4.92 | 43.3 (DWL) | | | 0.896 |
| 20 | 2009 | 4.57 | 11 | 0.34 | 4.72 | 43.3 (DWL) | 4.14 | 6.34 | 0.805 |
| 21 | 2010 | 4.57 | 11 | 0.34 | 4.81 | 43.3 (DWL) | 4.10 | 6.33 | 0.851 |
| 22 | 3001 | 5.35 | 11.8 | 0.35 | 5.14 | 37.16 (MLLW) | | | 0.822 |
| 23 | 3002 | 5.35 | 11.8 | 0.35 | 5.29 | 37.16 (MLLW) | 4.95 | 7.83 | 1.625 |
| 24 | 3003 | 5.35 | 11.8 | 0.35 | 5.29 | 37.16 (MLLW) | 5.11 | 7.91 | 1.645 |
| 25 | 3004 | 5.35 | 11.8 | 0.35 | 5.27 | 37.16 (MLLW) | 4.83 | 8.21 | 1.526 |
| 26 | 3005 | 5.35 | 11.8 | 0.36 | 5.65 | 39.50 (MSL) | 4.82 | 7.22 | 2.223 |
| 27 | 3006 | 5.35 | 11.8 | 0.36 | 5.40 | 39.50 (MSL) | 4.98 | 7.46 | 1.940 |
| 28 | 3007 | 5.35 | 11.8 | 0.36 | 5.45 | 39.50 (MSL) | 4.93 | 7.32 | 1.991 |
| 28 | 3008 | 5.35 | 11.8 | 0.36 | 5.55 | 43.3 (DWL) | 5.38 | 7.14 | 5.640 |
| 30 | 3009 | 5.35 | 11.8 | 0.36 | 5.41 | 43.3 (DWL) | 5.07 | 6.97 | 3.760 |
| 31 | 3010 | 5.35 | 11.8 | 0.36 | 5.47 | 43.3 (DWL) | 5.17 | 6.99 | 4.642 |
| 32 | 4001 | 1.92 | 7.9 | 0.24 | 1.90 | 37.16 (MLLW) | | | |
| 33 | 4002 | 4.57 | 11 | 0.32 | 4.82 | 37.16 (MLLW) | | | |
| 34 | 4003 | 5.35 | 11.8 | 0.36 | 5.60 | 37.16 (MLLW) | | | |

Totally 34 tests were performed during this study.

**Model Implementation**

All experiments were performed in the wave flume of Soil Conservation and Watershed Management Research Institute (SCWMRI). The wave flume has 33 meters length, 5.5 meters width and one meter depth. The wave flume was separated in three parts. The model of structure was constructed in the end of the middle part. Plan view and Cross section of the wave flume with all instruments and their layouts showed in Figure 5.

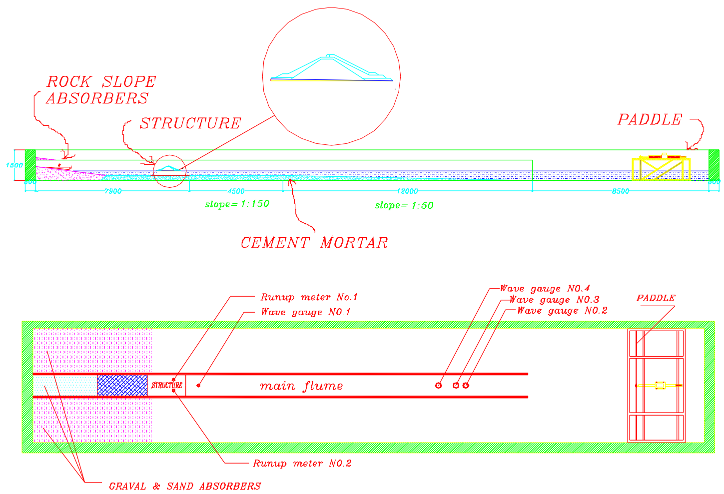


**Fig. 5) Cross section and Plan view of wave flume and setup of sensors**

Based on bathymetry of IRAN-LNG harbour zone and design wave directions, slope of sea bed was carried out about 1:80. In order to use of scale ratio ($\lambda$=35) and length of wave flume, the slope of sea bed have been constructed in middle part of wave flume. The Constructed sea bed model is presented in Figure 6.

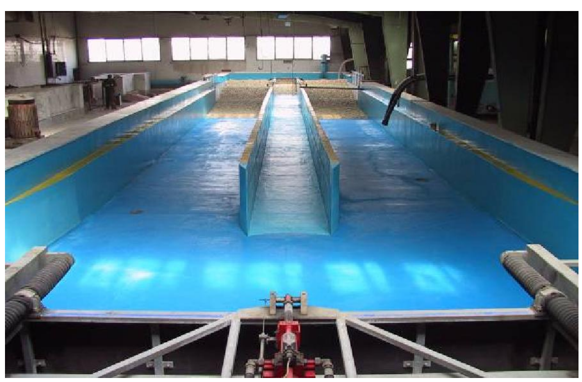

**Fig. 6) Constructed sea bed model in the wave flume (middle part)**

The needed stone materials in order to use in model have been crushed and graded from their original materials which have been provided by client from rock quarries in the project site. The procedure of crushed rocks grading by weight in is presented in Figure 7. By using of Xbloc® model ingots, the Xbloc® models were constructed by concrete filing of ingots. Breakwater model under construction by Xbloc®s is shown in Figures 8. Model section B8-B8-B of the breakwater was constructed in visible part of main wave flume. The new B8-B8-B breakwater model section that has been plotted in visible part of wave flume is showed in Figure 9 in comparison with section B8-B8-A.

**Fig. 7) Stone materials grading by weight**

**Fig. 8) Xbloc® armor layer under Construction**

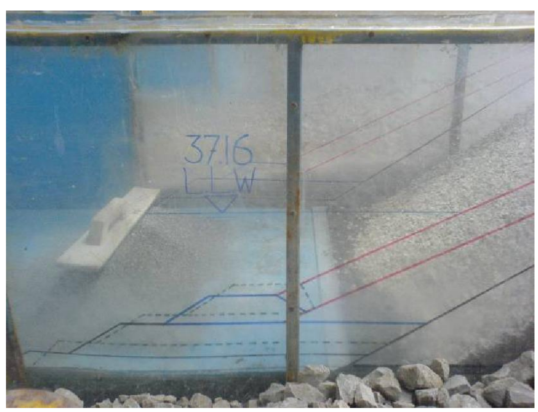


**Fig. 9) Drawing of breakwater section model in visible part of wave flume (**section B8-B8-B)
(Dashed lines shows the section B8-B8-A in comparison with section B8-B8-A**)**

One wave height meter was placed at toe of structure in order to record of incident wave with a distance of less than 1/4 wave length from the structure. Three wave height meters have been placed in the middle of the flume for finding out the reflection coefficient of structure based on Mansard method (see Mansard et al. 1979). Two wave height meters also have been placed inclined align at the armour layer slope (1:2.0) in order to record of wave run-up levels (Figure 10). A 20 cm width channel with related water reservoir has been used behind the crest of the model structure for wave overtopping measurements.

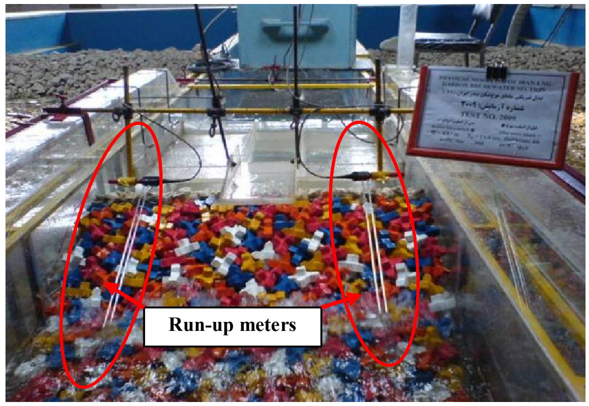


**Fig. 10) Run-up meter sensors**

**Stability Assessment**

In order to investigation of structural stability under storm conditions and waves attack, three methods have been used simultaneously for each test as the following:

**Profiling**

A profiling system has been used in order to carry out movement and reshaping of armour layer materials (Xbloc®s) and toe berm armour stones. The profiling system consists of a vertical point gage with 0.1 mm accuracy that has been mounted on a 4wheel cart (Figure 11). Profiling of armour layer and toe berm of structure has been done before and after of each test in order to comparison and finding out the reshaping and movements of armour units and toe berm stones.

**Fig. 11) Profiling system**

**Color photographs**

For each test, color photographs for both before and after wave attacks are made in order to finding out movement of armour units by comparison of them.

**Video recordings**

For each test, video recording are made in order to stability and hydraulic responses investigations.

## CONCLUSIONS

### Stability Assessment of Xbloc® Armor Layer

Comparison of armour layer profiles before and after wave attack by use of both profiling and related photographs shows that the breakwater is statically stable and no movement of armour stones occurred (e.g. Number of displaced units, $N_{od} = 0$). A sample of profiling plot is presented in Figure 12. Both of before and after wave attack profiles are plotted together in order to comparison. Down side difference between these two profiles is due to upward movement of toe berm stones. It demonstrate that flatter slope of 1 : 2 related to default 3 : 4 slope which has been recommended by Delta Marine Consultants (Xbloc® designer) cause to more stable structure. It is noticeable that choose of flatter slope (1 : 2 instead of 3 : 4) for armour layer by designer was based on some geotechnical problems of sea bed at the base of the structure.

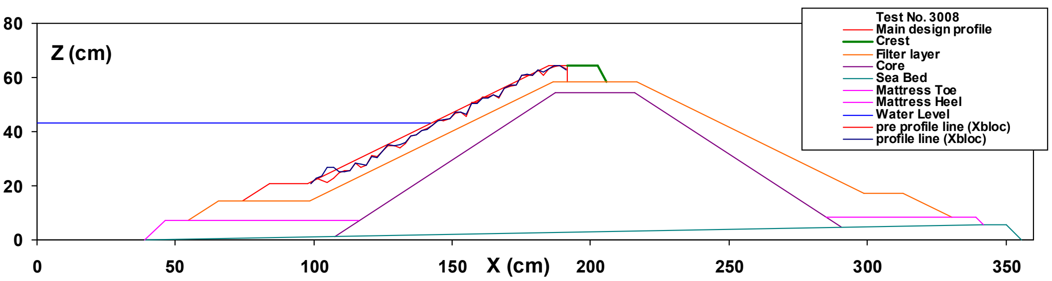


**Fig. 9) Profiling Plot of Test No. 3008 (Average profile of Xbloc® armour layer)**

### Hydraulic Responses

Wave run-up has been recorded during tests as described before by two wave height meters with frequency of 40Hz. Significant and 2% Run-up values presented in table 6. Higher run-ups occurred when higher and longer waves were attack to the structure. Wave overtopping occurred when run-up levels exceeded from the level of the structure's crest. Based on measured overtopping discharges which are presented in Table 6, it is clear that the wave overtopping discharges are not considerable.

### Toe Berm Stability

Toe berm profiling was performed for both before and after wave attack. Toe berm stability was evaluated by comparison of profiles. Reshaped profiles of toe berm have been shown the higher waves with related longer periods have more effect on reshaping of the toe berm materials. Lighter stones in filter layer which have been located under toe berm stones moved upward and downward of structure slope for main section of B8-B8-A. This movement causes instability of toe berm stones. Wave runups moved filter layer materials into armour layer and therefore these moved materials filled the holes between Xbloc® armour units. By reduction of filter layer materials which was located under the toe berm stones, toe berm materials moved easily and then in storms with higher return periods (50 and 100 years) more reshaping of the toe berm stones occurred. Thus Lower water levels for higher wave condition caused the toe berm failure.
In order to choosing more stable toe berm, complementary 4000 series tests have been done for comparison of section B8-B8-B with previous section B8-B8-A. Therefore, three tests (4001, 4002 and 4003) were performed as complementary model study for toe berm stability. In order to investigation of toe Berm stability, the reshaping profiles compared with the previous simultaneous tests of section B8-B8-A. Toe berm profiles show that the yearly wave condition has no effect on stability for both new and previous breakwater sections (tests No. 1003 and 4001). Toe berm profiling plots of tests No. 4003 and 3003 which have approximately simultaneous hydrodynamic condition are presented in Figure 13. These two profiles show the

modified toe Berm (section B8-B8-B) is more stable related to previous breakwater section (B8-B8-A). Figure 14 shows the final reshape photographs of the tests 3001 and 4003 respectively which have either approximately simultaneous wave condition. Therefore, based on the mentioned Figures it is clear that the reduction of toe Berm level instantaneously with increasing the weight of toe Berm under layer materials has the most useful effect on increasing of the toe Berm stability. Thus the new modified toe Berm (section B8-B8-B) is more stable than the previous one (section B8-B8-A). Thus the section B8-B8-A was reported to client as final modified and corrected section in order to construction in prototype.

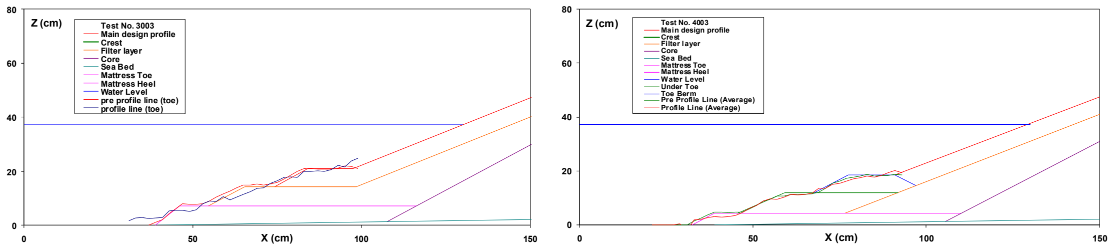


**Fig. 11) Comparison of reshaping profiles of the two toe Berms** (Left, Test No. 3003, section B8-B8-A and Right, Test No. 4003, section B8-B8-B)

**Fig. 11) Comparison of the Reshaped toe Berms** (Left, Test No. 3001, section B8-B8-A and Right, Test No. 4003, section B8-B8-B)

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