

## ORAN HIGHWAY REVETMENT DESIGN

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### ABSTRACT

The aim of this study is to give detailed information about the revetment stability of the highway located in the City of Oran, in Algeria. Oran highway is one of the most important highways in Algeria due to its location. The existence of cliffs in the region and the settlement of the City of Oran arose a necessity to re-align the highway; therefore, the highway was shifted towards the Gulf of Oran. Approximately 1km of the highway is to be constructed on the Mediterranean Sea due to the new alignment. In order to protect the sea side of the road from the adverse effects of the sea, a revetment was designed. Wave climate analysis, wave forecasting and wave transformation studies were carried out to obtain the design wave height in front of the toe of the revetment. After the application of the mathematical calculations, a revetment composed of 23 tons of antifer units on armour layer was designed. In order to check the hydraulic stability of the proposed section, physical model tests were performed in the laboratory of LEM (Laboratoire d'Etudes Maritimes) in Algeria. As a result of the physical model tests, the proposed section was found insufficient and thus some modifications had to be made. The antifer units used in the armour layer were increased from 23 tons to 26 tons, the toe length of the structure is increased from 12 meters to 17 meters and the weight of the rocks which were used in the toe is decreased from 6-8 tons to 4-6 tons. After the modifications, the new section was tested physically and it was selected as the best option.

**Keywords:** revetment, highway, antifer, design wave, wave transformation, physical model

### INTRODUCTION

Oran is an important coastal city that is located in the north-west of Algeria, shown in Figure 1. It is considered to be the second most important city after the capital Algiers due to its commercial, industrial, and cultural importance.



**Figure 1: Location and side view of the City of Oran and the harbour**

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Oran highway is located on northern side of the City of Oran, and it is one of the most important highways in Algeria due to its location (Figure 2). The highway is currently under construction for rehabilitation purposes and is expected to create a fast and safe link between the other two major cities of Algeria which are Arzew and Mostaganem. In addition to providing fast and safe transportation, after its construction is completed, the highway is also to serve as a safe trade route that connects the ports in the City of Oran, Arzew and Mostaganem.

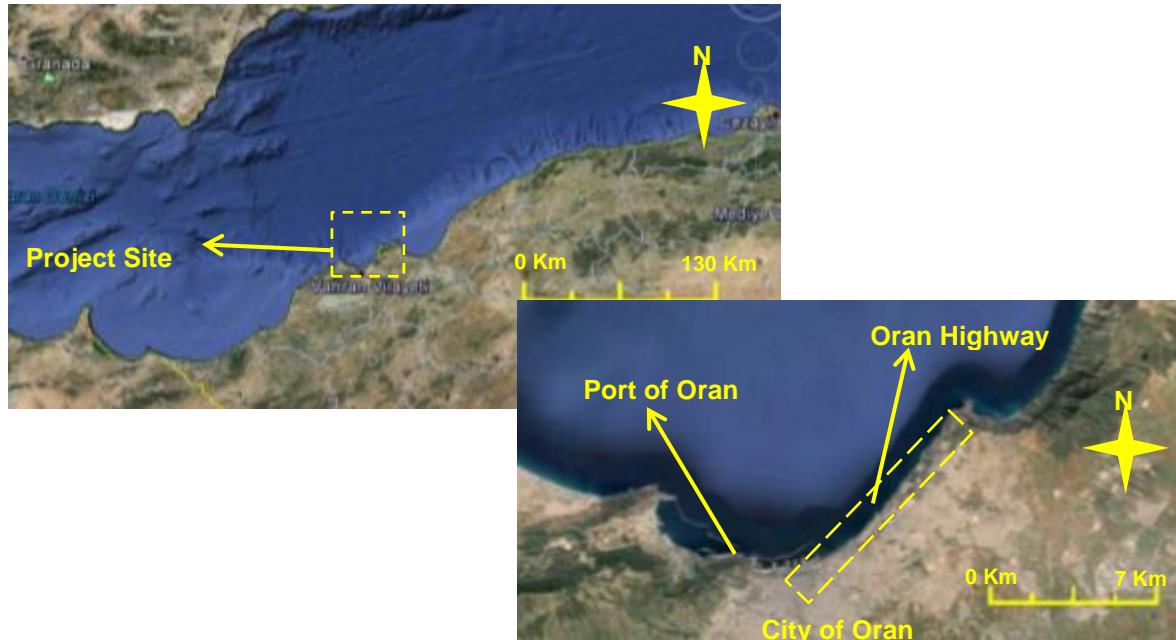


Figure 2: Location of Oran

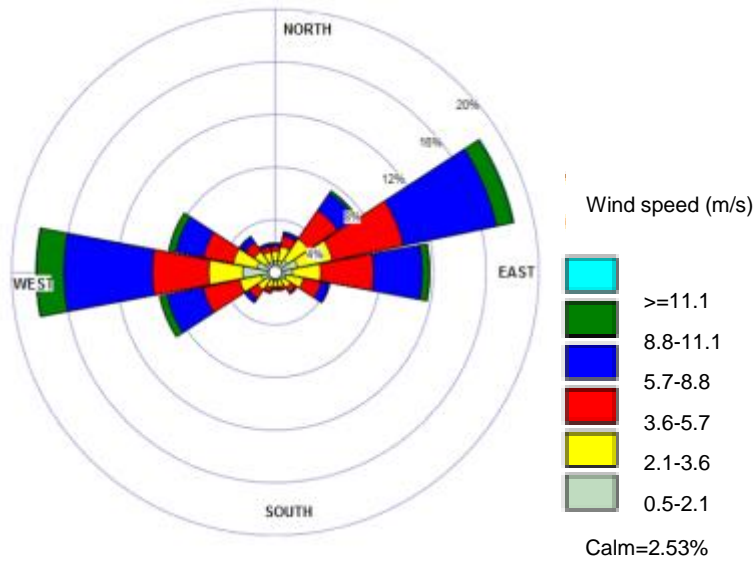
Depending on both the existence of cliffs in the region and the settlement of the City of Oran, the new alignment of the highway was shifted towards the Gulf of Oran, which made it necessary to construct approximately 1 km of the highway on the sea. The depth of the toe of the revetment varies between -4.0 m and -9.0 m. For the defence structure, the use of antifer armour units is preferred and the revetment stability is checked via Hudson equation for various water depths.

## WIND STATISTICS AND WAVE HINDCASTING

### Wind Data

In this study, ECMWF (European Centre for Medium-Range Wave Forecasts) wind data between the years 1983-2011 are used for the coordinates 36.00N 0.70W.

Wind rose and wind velocity class frequency histogram, which are obtained using ECMWF wind data considering all directions, are presented in Figure 3.



**Figure 3: Wind rose (Obtained using ECMWF wind data at coordinates 36.00N 0.70W)**

Effective fetch directions and effective fetch lengths are given in Figure 4 and Table 1, respectively.



**Figure 4: Effective fetch lengths for the project area**

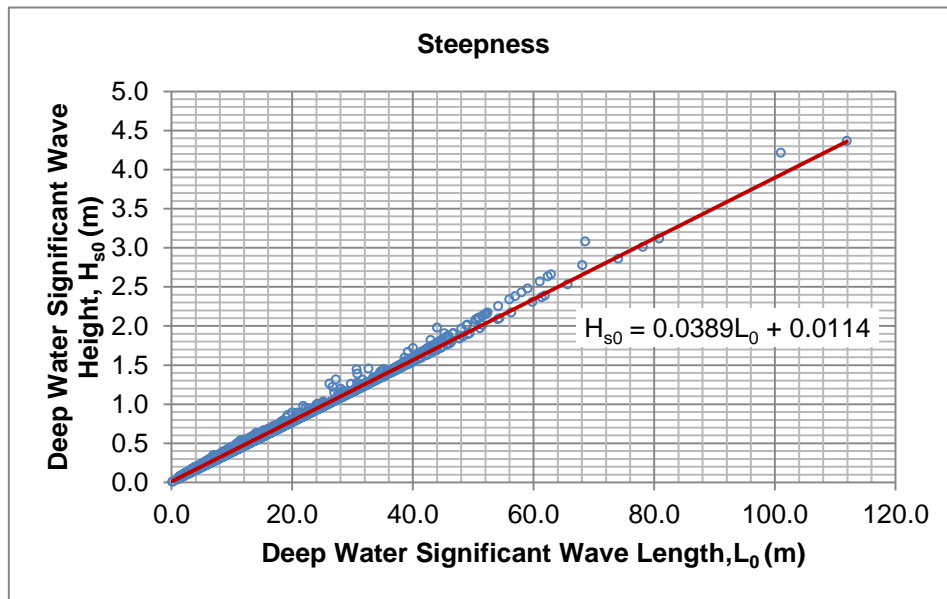
Direction	Effective Fetch Lengths (km)
E	312.4
ENE	625.5
NE	705.3
NNE	416.1
N	236.9
NNW	203.9
NW	273.7
WNW	357.1
W	266.9

**Table 1: Effective Fetch Lengths**

### Wave Hindcasting Study

A mathematical model, W61, (developed by Middle East Technical University) is used in wave hindcasting study, which is based on wind velocity data taken from ECMWF database for 36.00N 0.70E between 1983 and 2011. Deep water significant wave characteristics are defined using the mathematical model. According to the obtained results, deep water significant wave steepness ( $H_{s0}/L_0$ )

is calculated as 0.0389. Deep water significant wave heights versus deep water significant wave lengths are given in **Figure 5**.



**Figure 5: Deep water significant wave heights vs deep water significant wave lengths**

### Extreme Wave Statistics

Design wave height is calculated using extreme wave statistics for a return period of 100 years. Using the methods of extreme wave statistics such as FT-I (Gumble), FT-II, Weibull and Log-normal distributions, the data are fitted for yearly maximum significant wave characteristics. The best fit selection criteria, which are given in Goda (2000), are satisfied by Weibull  $k=1.0$  distribution.

The results of extreme wave statistics for 5 different return periods within 90% confidence interval limits and wave steepness values are given in Table 2.

Return Period (RP) (year)	Deep water significant wave heights $H_{s0}$ (m)	Deep water significant wave periods $T_s$ (sec)
		$(H/L)_0=0.0389$
5	$2.08 \pm 0.25$	$5.86 \pm 0.35$
10	$2.76 \pm 0.48$	$6.75 \pm 0.57$
20	$3.28 \pm 0.69$	$7.36 \pm 0.74$
50	$4.48 \pm 1.20$	$8.60 \pm 1.09$
100	$5.00 \pm 1.43$	$9.09 \pm 1.21$

**Table 2: Extreme wave statistics results (Deep water)**

For a return period of 100 years, deep water significant height ( $H_{s0}$ ) has a range between 5.00 m and 6.43 m; the significant periods ( $T_s$ ) range between 9.09 sec and 10.30 sec. The deep water significant wave height and wave period are selected as  $H_{s0}=6.43$  m,  $T_s=10.30$  sec, respectively.

Another study for extreme wave statistics was carried out by LEM. LEM used the data obtained from France meteorological station. The deep water wave characteristics suggested by LEM as a result of extreme wave analysis are given in Table 3.

$H_{s0}$ (m)	10.40
$T_s$ (sec)	12.30
$H_{s0}/L_0$	0.044

**Table 3: Deep water design wave parameters provided by LEM for a return period of 100 years**

Comparing Table 2 and Table 3, it is seen that for a return period of 100 years, the design wave heights differ considerably. This difference results from the data source. In general, ECMWF gives lower values depending on the location of data source and boundary conditions. Due to the importance of the project, the experience of the local authority in Algeria (LEM) was taken into consideration in the determination of the deep water wave characteristics. Thus, the deep water wave characteristics ( $H_{s0}=10.4$  m,  $T_p=12.9$  sec) provided by this authority were used in the project.

### Extreme Water Level Variation

A structure may be exposed (and possibly vulnerable) to different risks for different water levels. To illustrate, the wave height is limited due to breaking before arriving at a structure depending on water level. Also, construction and maintenance of coastal structures are generally affected by the overall water level regime.

Therefore, various components of water level, which are given below, are to be considered.

### *Tidal Oscillations, Seasonal Variations and Sea-Level Rise*

The difference between the highest and lowest monthly mean sea levels can be taken as 1.2 ft = 37 cm (Department of Commerce, USA, 2013). Tidal change can be taken as 1.4 ft = 43 cm (Department of Commerce, USA, 2013). Sea level rise can be taken as 1 mm/year (National Intelligence Council, 2009).

### *Wind Set-up*

The wind set-up values ( $\eta_0$ ) are calculated for each direction (Table 1) using Eq.1 (OCDI, 2002): In wind set-up calculations, since only deep water wave characteristics are provided by LEM, there is no wind data available. Thus, in order to calculate wind set-up, wind data of ECMWF (36.0N 0.7W) are used.

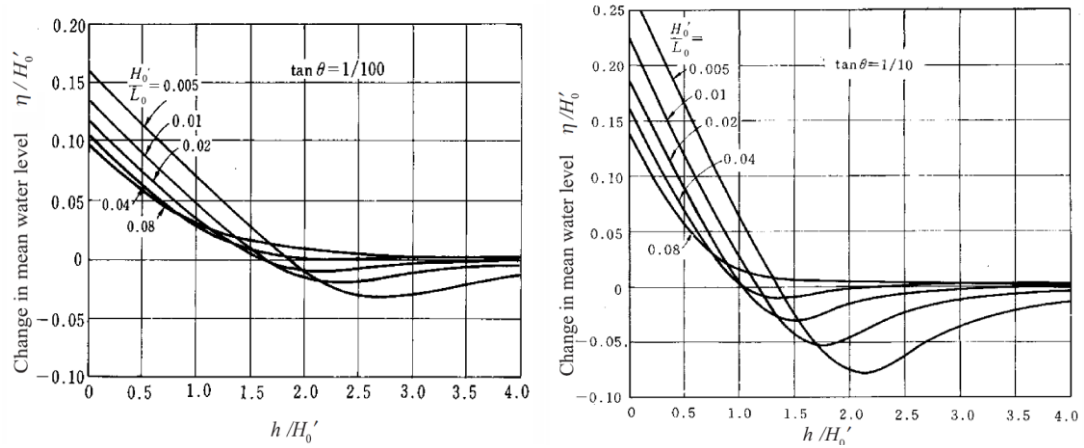
$$\eta_0 = 4.8 \times 10^{-2} \times \frac{F}{h_{mean}} \times U_{ave,10}^2 \quad (1)$$

where, F: effective fetch length (km),  $h_{mean}$ : average water depth (m),  $U_{ave,10}$ : average wind speed 10 m above the sea surface (m/s)

### *Wave Set up*

The total rise in the mean water level at the shoreline due to random wave breaking (total wave set-up) is the sum of static (mean wave set-up) and dynamic (surf beat) components of wave set-up.

The static component of wave set-up values ( $\eta$ ) at relative water depths ( $h/H_0$ ) for different sea bottom slopes are obtained from Figure 7 a-b. (OCDI, 2002)



**Figure 6: a-b: Change in mean water level (a: Bottom Slope 1/10, b: Bottom Slope 1/100; OCDI, 2002)**

The dynamic (oscillating) component of wave set-up, surf beat amplitudes ( $\zeta_{rms}$ ) at shorelines are computed from Eq. 2 (OCDI, 2002) where  $h$  is the water depth.

$$\zeta_{rms} = \frac{0.01 \times H_0'}{\sqrt{\frac{H_0'}{L_0'} \times \left(1 + \frac{h}{H_0'}\right)}}, H_0' = H_0 \times K_r, \quad (2)$$

where,  $H_0'$ : equivalent deep water wave height (m),  $K_r$ : refraction coefficient,  $h$ : water depth (m) and  $L_0$ : deep water wave length (m)

In the calculation of wave set up, sea bottom slope is taken as 1/20.

### Coriolis and Barometric Effects

The additional rise of sea water level due to the effects of coriolis and barometric pressure differences is assumed to be 10 percent of the total rise above SWL due to other components of storm surge.

In summary, the total extreme storm surge water level rise (Table 4) for an extreme storm event for a return period of 100 years is calculated as 1.45 m above the present SWL in front of the revetment.

Tidal Rise (m)	0.43 m
Seasonal Variations (m)	0.37 m
Sea Level Rise (m)	0.10 m
Wind Set-up (m)	0.18 m
Wave Set-up (m)	0.24 m
Barometric and Coriolis Effects (+10% of the total rise above SWL)	0.13 m
Total Rise	1.45 m

**Table 4: Extreme storm surge water levels**

### Wave Transformation

In order to determine the near shore wave characteristics at the project area, deep water significant wave height ( $H_{s0} = 10.40$  m) has to be transformed to structure design depth (at toe) which is defined as  $d_{SWL} = 9.0$  m in this study. To determine the near shore wave characteristics, well-known open source program SWAN (Simulation Waves Near-shore) is used. The basic inputs of the mathematical model (SWAN) are the bathymetry of the near shore area, wave directions and the deep water wave characteristics.



Wave transformation analysis is performed for 5 different cases by considering the present still water level, high and low water levels for both the present and 100 years later (Table 5).

Case 1: Water level (WL) = Still Water Level (SWL)

Case 2: Water level (WL) = Still Water Level (SWL) + Seasonal Changes (SC) + Tidal Changes (TC) + Barometric and Coriolis Effects (BCE)

SC+TC+BCE = 0.93 m (HWL=SWL+0.93 m)

Case 3: Water level (WL) = SWL - SC - TC - BCE (LWL=SWL-0.93 m)

Case 4: Water level (WL) = SWL + Global Warming (GW) + SC + TC + BCE

GW+SC+TC+BCE = 1.03 m

Case 5: Water level (WL) = SWL + GW - SC - TC - BCE (LWL=SWL-1.03 m)

Case No	Seasonal Changes	Tidal Changes	Global Warming	Barometric and Coriolis Effects	Result
Case 1	Not Included	Not Included	Not Included	Not Included	Still Water Level
Case 2	Added	Added	Not Included	Added	HWL (Today)
Case 3	Subtracted	Subtracted	Not Included	Subtracted	LWL (Today)
Case 4	Added	Added	Added	Added	HWL (100 years later)
Case 5	Subtracted	Subtracted	Subtracted	Subtracted	LWL (100 years later)

**Table 5: Wave transformation performed for 5 cases**

As a result of the wave transformation analysis, NW direction is found as the dominant wave direction. The summary for the five cases is provided in Table 6.

Case-1	6.3 m
Case-2	6.9 m
Case-3	5.9 m
Case-4	7.0 m
Case-5	5.9 m

**Table 6: Extreme storm surge water levels**

It is decided that Case 4 (Water level (WL) = SW + GW + SC + TC + BCE) is the most critical case. Therefore, the design depth and design wave height are selected as 10.03 m and 7.0 m, respectively.

### Hydraulic Stability Analysis of Revetment

In addition to the analyses described above, the existing Oran highway revetment project which was prepared by the client (using 15 m<sup>3</sup> Accropode units) was evaluated in detail. After the evaluation, one important question about the construction capabilities of these interlocking units in Algeria has arisen depending on the importance of the highway. In order to understand whether such units can be satisfactorily applied in Algeria, a detailed literature survey was carried out. The literature survey revealed that 20 m<sup>3</sup> tetrapode units, which have interlocking behaviour like accropode units, were used in the Port of Oran breakwater, which is located next to the new alignment of Oran highway. However, more interestingly, it was discovered that this breakwater was damaged in a storm that had a significant wave height of 7.0 m, which has approximately the same wave characteristics as the design wave which occurs at the toe of the new revetment for the Oran highway. In order to stay on

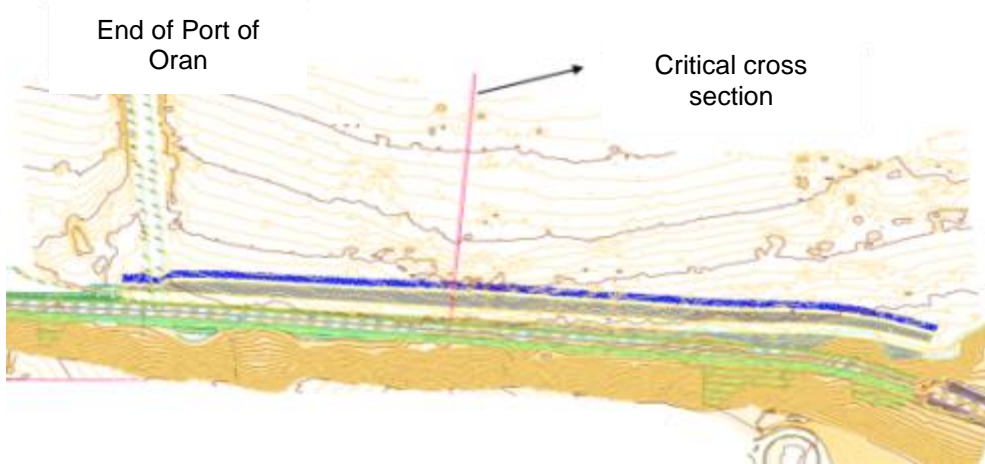
the safe side, it is decided (with LEM) that the use of more robust units is much more suitable for the new revetment. The previous construction experience in Algeria was the main issue that was considered while selecting the type of the armour layer unit. As a result of the detailed survey, the most commonly applied armour layer unit in Algeria was found as antifer armour units.

The design parameters which are used in stability analysis for the revetment are summarized in Table 7.

Deep water significant wave height	$H_{s0}$	10.4 m
Deep water significant wave period	$T_s$	12.3 sec
Selected depth of construction at the toe at SWL	$d_{SWL}$	9.0 m
Sea bottom slope	$m$	1/20
Significant wave height at the toe of structure	$H_s$	7.0 m
Peak wave period	$T_p$	12.9 sec
Spectral wave period	$T_{m-1,0}$	11.7 sec
Wave height exceeded by 2% of the waves at the toe	$H_{2\%}$	9.3 m

**Table 7: Design parameters used in the stability calculations**

In order to perform hydraulic stability analysis, the critical section shown in Figure 7 is selected.



**Figure 7: The location of critical section used in hydraulic stability analysis**

### Breaking Conditions Check

It is observed that Case 4 is the most critical one. Thus, in stability calculations, wave height is taken as  $H_s = 7.0$  m at  $d_{HWL} = 10.03$  m.

Breaking conditions are checked for the most critical case by using the formulations given in CEM, 2003 and the breaking wave depth is calculated as 7.6 m. Therefore, non-breaking wave condition is valid.

### Stability Calculations

Hudson method is used in the stability calculations of the antifer armour units.

$$W_{50} = \frac{\gamma_{antifer} \times H_s^3}{\left( \frac{\gamma_{antifer}}{\gamma_w} - 1 \right)^3 \times \Delta \times \cot \alpha} \quad (3)$$

$W_{50}$ : Weight of armour unit (tons)

$H_s$ : Significant wave height at the toe of structure (m)

$\gamma_{antifer}$ : Specific weight of antifer block (taken as 2.4 t/m<sup>3</sup>)



$\gamma_w$ : Specific weight of sea water (taken as 1.025 t/m<sup>3</sup>)

$K_D$ : Stability coefficient

$\alpha$ : Structure slope

$K_D$  values which are suggested by LEM for the stability calculations of antifer armour units are given in Table 8.

cota	$K_D$
1.5	6.0
2.0	7.5

**Table 8:  $K_D$  values suggested by LEM**

### Armour Layer Design for Antifer Units

For structure slope 1/1.5 and 1/2, the weights of antifer armour units are calculated using Breakwater 3.3 software. For structure slope 1/1.5 and 1/2, weights of antifer armour units are summarized in Table 9.

$H_s$ (m)	Structure Slope	$K_D$	$W_{50}$ (tons)
7.0	1/1.5	6.0	37.9
7.0	1/2.0	7.5	22.7

**Table 9:  $W_{50}$  values calculated for structure slope 1/1.5 and 1/2**

$W_{50}$  = 23 tons is selected for structure slope,  $cota = 1/2$ .

### Wave Run up

Wave run up is calculated using Rock Manual, 2007, EurOtop, 2007 and Breakwat software. Wave run-up values are summarized in Table 10 for each method.

Armour Layer	Method Used	$R_{u,2\%}$ (m)	$R_{u,2\%}$ (m) with $R_p=100$ years
Antifer	Rock Manual	12.11	12.53 (12.11+0.42)
	EurOtop	10.98	11.40 (10.98+0.42)
	TAW (From Breakwat 3.3)	12.43	12.85 (12.43+0.42)
	Van Gent (From Breakwat 3.3)	10.59	11.01 (10.59+0.42)

**Table 10: Wave run-up for antifer blocks for each method**

0.42\*: sum of wind set-up and wave set-up values

### Splash Zone Length

For the calculation of splash zone length Cox and Machemehl (1986) method is used.

$$L_s = 0.2 \times T \times \psi \times \sqrt{g \times (R_u - R_c)} \quad (4)$$

$\Psi$ : Importance of structure factor ( $1 < \Psi < 2$ )

T: Wave period ( $T_{m-1,0}$ ) (sec)

$R_u$ : Fictitious wave run up (m) : 12.9 m for antifer armour units

$R_c$ : Crest Level (m): 10 m from still water level

g: Gravitational acceleration ( $\text{m/s}^2$ )

For two different values of importance factor, splash zone lengths are summarized in Table 11.

Armour Layer Type	Importance Factor, $\Psi$	$L_s$ (m)
Antifer Block	1.0	12.5
	2.0	25.0

Table 11: Splash zone lengths

From the calculated values of splash zone length, average of  $L_s$  value is suggested as 18.8 m for antifer blocks (Figure 8)

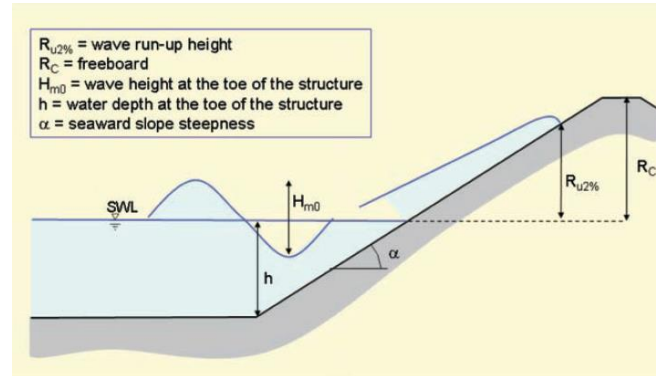


Figure 8: Splash zone length (EurOtop)

### Wave Overtopping

Wave overtopping is calculated as the amount of water overtopping the crest freeboard per meter ( $\text{m}^3/\text{s}/\text{m}$  or  $\text{l}/\text{s}/\text{m}$ ). The critical values of the design overtopping rates are given in Table 12.

Usage (Rear side of the structure)	Mean Overtopping Discharge ( $q$ ) ( $\text{m}^3/\text{s}/\text{m}$ )	$V_{\max}$ Peak Overtopping Volume ( $\text{m}^3/\text{m}$ )
<b>Vehicles</b>		
Unsafe for driving at moderate or high speed, impulsive overtopping giving falling or high velocity jets	$q > 1.10^{-5} - 5.10^{-5}$	$V_{\max} > 5.10^{-3}$
Unsafe for driving at low speed, overtopping by pulsating flows at low levels only, no falling jets	$q > 0.01 - 0.05$	$V_{\max} > 1.10^{-3}$
<b>Buildings</b>		
No Damage	$q < 1.10^{-6}$	
Minor Damage to Fittings etc.	$1.10^{-6} < q < 3.10^{-5}$	
Structural Damage	$q < 3.10^{-5}$	

Table 12: Critical wave overtopping (Rock Manual, 2007)

Wave overtopping is calculated using TAW (2002a) and Breakwat (TAW-Neural Network Methods).

In Table 13, overtopping values calculated by using different methods are summarized.

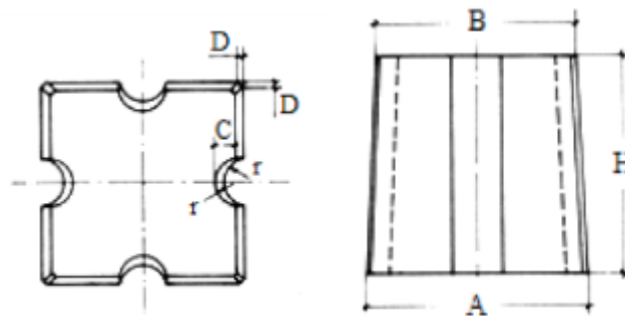
Method Used	$q$ (m <sup>3</sup> /sec/m)
TAW	0.011
TAW (Breakwat 3.3)	0.024
Neural Network (Breakwat 3.3)	0.021

**Table 13: Summary of overtopping values**

Although, the calculated overtopping values are large, no problems are expected along the highway due to overtopping since the elevation of the highway is high enough.

### Size of Antifer Blocks

The dimensions of a typical antifer armour unit are given in Figure 9.



**Figure 9: Dimensions of a typical antifer armour unit (a: Top View, b: Front View)**

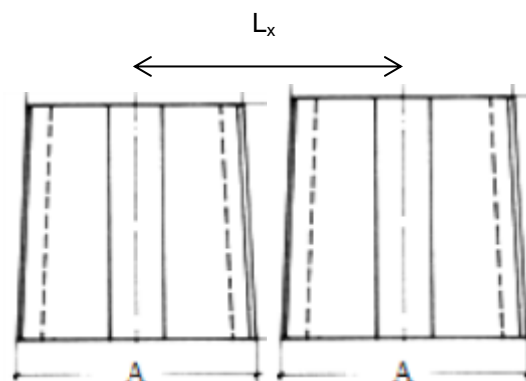
The dimensions of a 23 tons-antifer armour unit are given in Table 14.

	$V_{\text{antifer}}$ (m <sup>3</sup> )	A (m)	H (m)	B (m)	C (m)	D (m)	R (m)
Calculated	9.58	2.29	2.11	2.12	0.20	0.05	0.26

**Table 14: Dimensions of antifer blocks**

### General Layout of Antifer Blocks

The placement of antifer armour units is very important in terms of stability; therefore, the regular placement of antifer blocks should be made according to the general layout given in Figure 10 and Figure 11.



**Figure 10: Distance between the centers of each antifer block**

By using the value of A given in Table 13,  $L_x$  is calculated approximately as 2.85 meters.

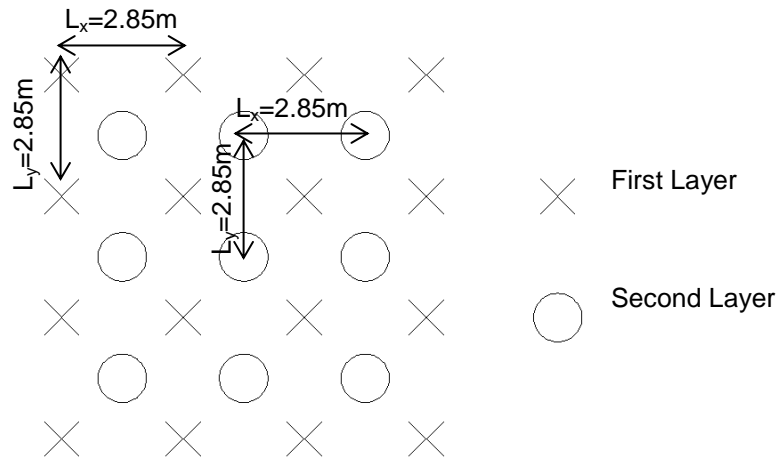


Figure 11: General layout of regular placement of antifer blocks

### Conclusion of Mathematical Calculations

The results of the stability analysis are summarized in Table 15.

Type of Armour Layer	Antifer Cube
Method Used	Hudson
Wave Height at the toe of Structure, $H_{s,toe}$ (m)	7.0
$T_s$ (sec)	12.3
Structure Slope ( $\alpha$ )	1/2
$K_D$	7.5
$\gamma_{antifer}$ (t/m <sup>3</sup> )	2.4
$\gamma_{rock}$ (t/m <sup>3</sup> )	2.65
$\gamma_{water}$ (t/m <sup>3</sup> )	1.025
$W_{50}$ (tons)	23
Under Layer $W_{50,A}$ (tons)	2.0 – 4.0
Core $W_{50,C}$ (kg)	20 - 400
Wave Run-up (m)	12.9
Crest Level from SWL(m)	10.00
Splash Zone Length (m)	18.8
Crest Width (m)	19.5

Table 15: Summary of stability analysis

The typical cross section of the revetment before the physical model tests is given in Figure 12.

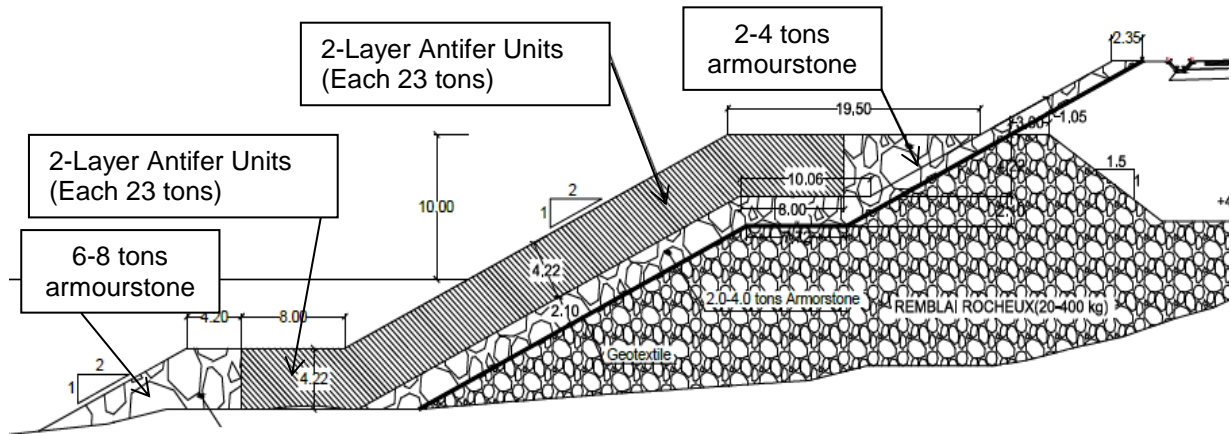


Figure 12: Suggested cross section of the revetment

## PHYSICAL MODEL TESTS

Although, regular placement of antifer armour units is suggested in the design, depending on the previous applications in Algeria, irregular placement is preferred instead of regular placement of antifer armour units.

The prototype is scaled according to Froude model and selected as 1/60.4.

The test procedure is presented below:

*Progressive Series:* These test series are carried out using two different water levels, 0.00 m and - 0.85 m, respectively. For each test, waves are generated with the shown order.

*Endurance Series:* These test series are carried out using the wave characteristics of a 100 years return period wave, with a water level of +0.85 m and a duration equivalent to 12 hours in reality.

*Reserve Stability Test:* These test series are carried out using the  $H_s = 1.1 \times H_{s,100 \text{ years}}$  with a water level of +0.85 m. The duration of the test is equal to the cycle time.

The wave characteristics that are used in the physical model tests are shown in Table 16.

Physical model, which is shown in Figure 14, is constructed at the laboratory for irregular placement.

Test No	Return Period	$H_s$ (m)	$H_s$ generated in -30m (m)	$T_p$ (sec)
1	1 year	3.0	2.5	6.32
2	2 years	4.7	4.00	8.00
3	5 years	6.1	5.00	8.94
4	10 years	7.0	6.00	9.79
5	20 years	8.1	7.00	10.58
6	50 years	9.5	8.00	11.31
7	100 years	10.4	9.00	12.00

Table 16: Wave characteristics that are used in the physical model tests



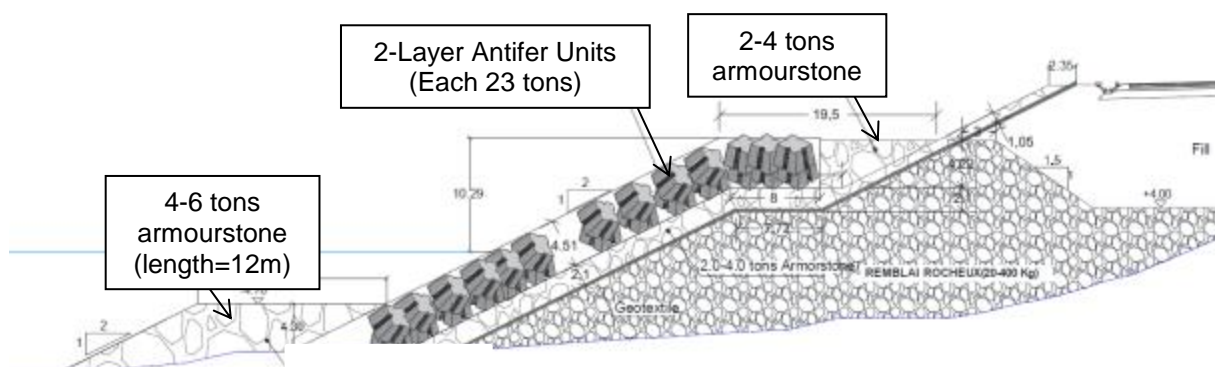
**Figure 13: Suggested cross section of the revetment in laboratory conditions**

The physical model tests for the suggested cross section given in Figure 13 (23 tons of antifer blocks are used in the armour layer and toe; 6-8 tons of rock are used in the toe) were repeated for SWL, HWL and LWL. After a series of tests using the wave characteristics given in Table 14, it was seen that there was damage on the trunk; some antifer armour units rolled down and stayed in front of the toe (Figure 14).

The tests revealed that the stability of the armour unit was insufficient due to wave breaking at the toe. Thus, the cross-section of the revetment was modified into the section given in Figure 15.



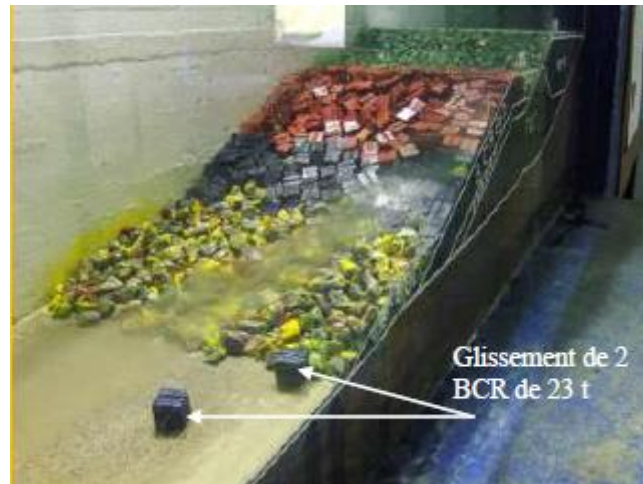
**Figure 14: Damage of irregular placement of antifer block after a series of tests**



**Figure 15: Modified cross section of the revetment**



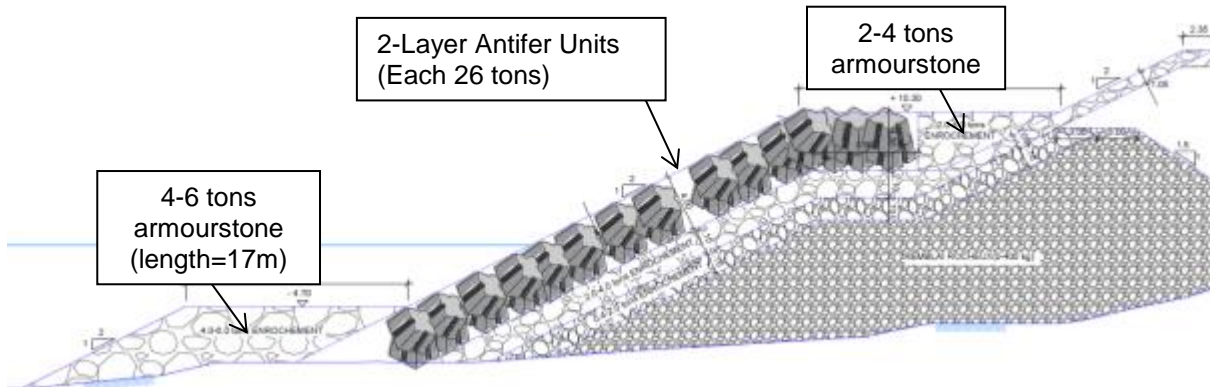
The only difference between the two sections (Figure 12 and Figure 15) is the modification made on the toe of the structure. Instead of 12.0 meters long toe consisting of 6-8 tons of rock and antifer units, a 12.0 meters long toe composed of 4-6 tons of rock is used. The physical model tests were repeated for the section given in Figure 16. After a series of tests, damage was observed at the armour layer. The results of the physical model test are given in Figure 16.



**Figure 16: Sample picture from the end of physical model tests for modified section**

After the evaluation of all the test results, the size of antifer armour units used in the cross section is increased from 23 tons to 26 tons. The placement of antifer is placed randomly. In addition to the increase in the size of armour units, a further modification is made on the toe. The length of the toe is increased from 12.0 m to 17.0 m. The physical model tests for the suggested cross section were repeated for SWL, HWL and LWL and are given in Figure 17.

As a result of the tests, it is decided that the armour unit is stable.



**Figure 17: Final (recommended) cross section of revetment**

The modifications on the cross section as a result of physical model test are summarized in Table 17.

	Before Physical Model Tests (Design cross section)	After Physical Model Tests (Modified cross section)
<b>Armour Layer</b>	23 tons of antifer blocks	26 tons of antifer blocks
<b>Placement of armour layer</b>	Regular	Irregular
<b>Length of toe</b>	12 m	17 m
<b>Stone size used at toe</b>	6-8 tons of rock and 23 tons of antifer units	4-6 tons of rock

**Table 17: Modifications in cross section as a result of physical model tests**

## CONCLUSION

The properties of the final cross section of the revetment after the physical model tests are given in **Table 18**.

Type of Armour Layer	Antifer Cube
Method Used	Hudson
Wave Height at the toe of Structure, $H_{s,toe}$ (m)	7.0
$T_s$ (sec)	12.3
Structure Slope ( $\alpha$ )	1/2
$K_D$	7.5
$\gamma_{antifer}^3$ (t/m <sup>3</sup> )	2.4
$\gamma_{rock}^3$ (t/m <sup>3</sup> )	2.65
$\gamma_{water}^3$ (t/m <sup>3</sup> )	1.025
$W_{50}$ (tons)	26
Filter Layer-1 $W_{50,F1}$ (tons)	2.0 – 4.0
Filter Layer-2 $W_{50,F2}$ (tons)	0.4 – 2.0
Core $W_{50,C}$ (kg)	0 - 400
Wave Run-up (m)	12.85
Crest Level from SWL(m)	10.3
Splash Zone Length (m)	18.8
Crest Width (m)	19.5

**Table 18: Properties of the final cross section of the revetment**

By performing physical model tests, the behaviour of the section was clearly understood. The random placement (irregular placement) of antifer blocks is found to be safe from the hydraulic stability point of view. Furthermore, the physical model tests showed that the packing density should be increased in order to prevent local settlements and loss of armour unit.

Although relatively high overtopping was observed, due to the elevation difference between the crest level (10.3 m) and the highway (average 20.0 m), overtopping is expected to cause no harm.

The suggested section in Figure 17 is found to be hydraulically stable for this revetment. 26.0 tons of antifer blocks are used in the armour layer. The crest level of the revetment is selected as 10.30 m. 4-6 tons of rocks are used at the toe, and toe length is selected as 17.0 m.

Oran highway revetment is currently under construction. The construction site is shown in Figures 19 and 20.



**Figure 18: Antifer stocking site in Oran highway construction site**



**Figure 19: End of the revetment in the construction site (under construction)**

For similar projects, it is strongly recommended to perform physical model tests to clearly understand the behaviour of the proposed section (design section) in nature.

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