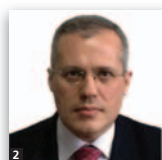


Comparative study of breakwater crown wall – calculation methods

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An investigation was undertaken consisting of a state-of-the-art and comparative analysis of currently available methods for calculating the structural stability of wave walls in sloping breakwaters. A total of six design schemes are addressed. The conditions under which the formulations and ranges of validity are explicitly indicated by their authors, are given. The lack of definition in parameters to be used and aspects not taken into account in their investigations are discussed and the results of this analysis are given in a final table. An investigation proposal based on an energy approach, in which the transmission of waves incident on the porous medium and its effect on the wall face is studied, brings the discussion to its close.

Notation

B_u	coefficient for calculating run-up
F	width of the crown wall foundation
h	half the wave height, wave height = $2h$
h_f	height of wave wall
p_H	uniform horizontal pressure
P_{ra}	uplift pressure at the front of the crown wall foundation
P_{re}	uplift pressure at the extrados of the crown wall foundation
P_{So}	dynamic pressure
So	width of the sheet of water ascending on the slope at level A_c
V_h	maximum horizontal velocity of the crest wave
WL	water level of the experimental set-up
Z	vertical coordinate, positive in an ascending direction (origin is at the design sea level)

1. Introduction

The wave wall has been a usual element in offshore works for decades, due to its great usefulness, especially in sloping breakwaters. It enables the elements in the armour layer to be reduced, reduces the possibility of the breakwater being

overtopped and, therefore, it improves the operability of the quays it protects, provides an access that may be used for maintenance work and the possibility of bringing in service systems. Installing a gallery from which to monitor the status and behaviour of the breakwater inside the wave wall started in recent decades.

Wave study has noticeably evolved with the passing of time. Statistical geometry and the evolution of recorders and measuring and prediction systems have brought advances in spectral analysis and consideration of wave energy, including the application of energy balances and effects of transmission and dissipation. Formulations available to determine the forces that waves produce on sloping breakwater wave walls do not explicitly embody these new energy criteria.

The aim of this study was to analyse and compare existing wave wall calculation methods, determining their ranges of application and detecting their uncertainties. This has led to a reflection on and proposal for new lines of investigation enabling the energy transmitted by waves onto the protection berm to be embodied.

Wave wall failure modes are

- sliding over foundations



Figure 1. Port of the island of Alborán (southern Spain). Hammer shock failure in 2001

- overturning from wave action
- overturning from foot undermining
- foundation plasticisation
- point failure through hammer shock.

This paper addresses the first two modes of failure. Photographs of recent failures are shown in Figures 1 to 3.

2. State of the art

Six methods for wave wall calculation are currently available. The notation used by the authors themselves in their respective papers has been retained and is defined in the text following the equations associated with each method.



Figure 2. Port of Motril, Granada (southern Spain). Slide failure in 2004



Figure 3. Port of Bermeo, Biscay (northern Spain). Slide failure and overturning in 2010

Three give pressures as a result (Iribarren and Nogales, Günbak and Göcke, Martín *et al.*) and the other three give forces (Bradbury and Allsop, Pedersen and Burcharth, Berenguer and Baonza). Each one is briefly addressed herein; they are grouped by the type of result they obtain (forces or pressures).

2.1 Pressure diagrams

2.1.1 Iribarren and Nogales (1954)

Iribarren and Nogales (1954) were the first to define the forces waves exert on the wave wall. Their work provides a pressure diagram as shown in Figure 4.

The representative height of the pressure at the crest is

$$1. \quad EB = 2 \frac{V_h^2}{2g} = h$$

The representative height of the pressure at the trough is

$$2. \quad JC = 2 \frac{V^2}{2g} = 5h$$

The presence of rock fill reduces the pressure to half and the pressure law on the wave wall is defined by ABD (see Figure 4). The definitive law of pressures exerted by the wave would be the line ABH in Figure 4. It considers that the friction between the base of the wave wall and the foundation is 0.50.

2.1.2 Günbak and Göcke (1984)

The model proposed by Günbak and Göcke (1984) assumes a uniform pressure distribution at the wave wall's freeboard

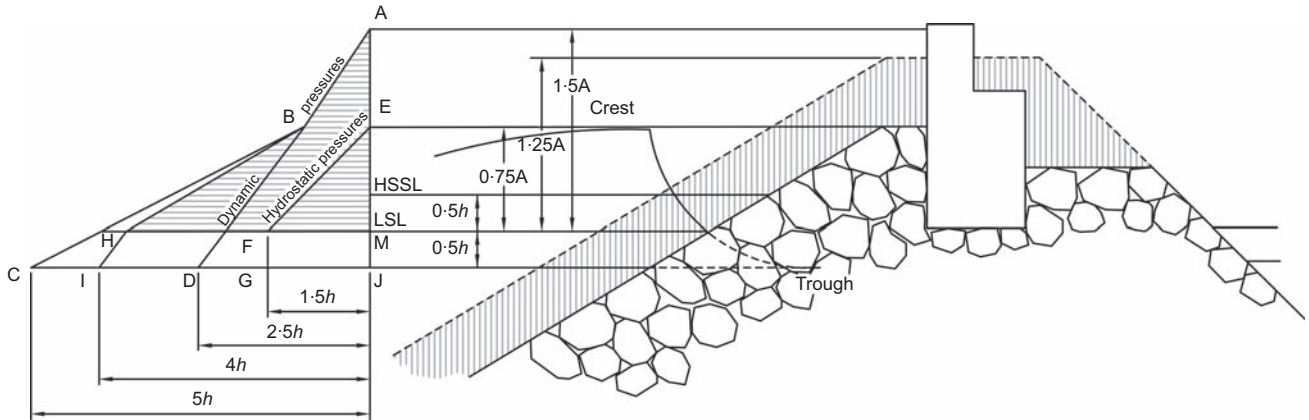


Figure 4. Pressure distribution according to Iribarren and Nogales; HSSL, high still sea level; LSL, low sea level; wave height = $2h$. Source: Negro *et al.* (2008)

which is called shock pressure (P_m). This figure is linearly reduced until reaching 50% at the base of the wave wall because of the presence of the protective layer. To this pressure is added the hydrostatic pressure (P_h) corresponding to run-up. The uplift pressure at the wave wall's base is triangular, as shown in Figure 5.

$$3. \quad P_m = \frac{\gamma_w (\sqrt{g}y)^2}{2g} = \frac{\gamma_w}{2} y \quad (\text{shock pressure})$$

$$4. \quad P_h = \gamma_w (y + s) \quad (\text{hydrostatic pressure})$$

$$5. \quad y = \frac{(R_u - A_c) \sin \beta}{\sin \alpha \cos(\alpha - \beta)}$$

The run-up is calculated with Günbak's formulation (Günbak and Göcke, 1984), according to which

$$6. \quad R_u = 0.4\zeta H \quad \text{if } \zeta < 2.5 \quad \text{with } \zeta = \sqrt{\frac{g}{2\pi H}} T \tan \alpha$$

$$7. \quad R_u = H \quad \text{if } \zeta > 2.5 \quad \text{with } \zeta = \sqrt{\frac{g}{2\pi H}} T \tan \alpha$$

where A_c is the height of the protection berm (m); g is the acceleration due to gravity (m/s^2); H is the wave height (m); R_u is the run-up of the liquid vein (m); T is the wave period (s); y is the distance between the berm's crown and the end of the run-up

(m); s is the stretch of wave wall protected by the armour layer (m); α is the angle of the armour layer's slope to the horizontal (sexagesimal degrees); β is the angle the liquid vein forms (Günbak gives it a value of 15 sexagesimal degrees); and γ is the specific weight of the water (kN/m^3).

2.1.3 Martín *et al.* (1995)

The method described by Martín *et al.* (1995) gives two diagrams for pressures on the wave wall: the *dynamic pressure* corresponds to the deceleration of the wave's front and the *pseudohydrostatic pressure* occurs during the descent of the mass of water accumulated on the structure. The pressures are as shown in Figure 6.

$$8. \quad P_d = \alpha \rho g s \quad A_c < z < A_c + s$$

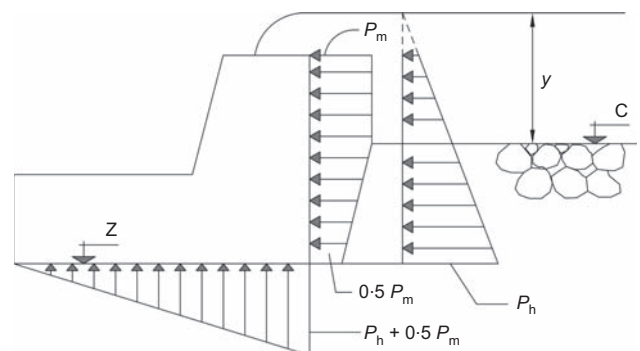


Figure 5. Pressure distribution according to Günbak and Göcke. Source: Negro *et al.* (2008)

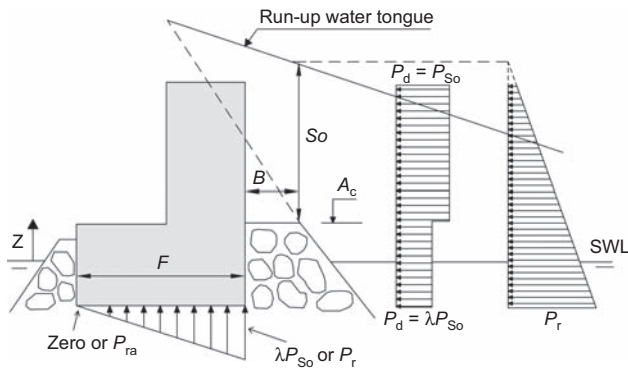


Figure 6. Pressure distribution according to Martín *et al.*; SWL, low sea level. Source: adapted from Martín *et al.* (1995)

$$9. \quad P_d = \lambda \alpha \rho g s \quad \text{foundation level } < z < A_c$$

$$10. \quad \alpha = 2 \frac{R_u}{H} \cos^2 \beta \cos^2 \theta$$

$$11. \quad s = H \left(1 - \frac{A_c}{R_u} \right)$$

$$12. \quad \frac{R_u}{H} = A_u [1 - \exp(-B_u I_r)]$$

(run-up according to Losada (1990))

The λ parameter reduces the pressure due to the presence of the armour layer. It is calculated with Figure 7, which was obtained by means of reduced model tests. The A_u and B_u coefficients are those referred to in Table 1.

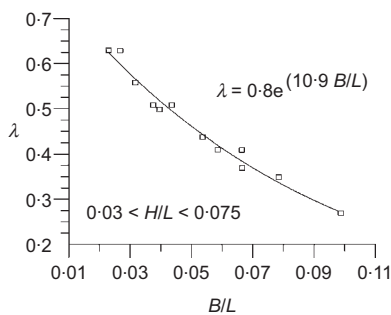


Figure 7. Adjusted values of λ (Martín *et al.*'s method.). Source: Negro *et al.* (2008)

	Rock					
	Rip-rap	fills	Blocks	Cubes	Tetrapods	Dolos
A_u	1.757	1.37	1.152	1.05	0.93	0.70
B_u	-0.435	-0.60	-0.667	-0.67	-0.75	-0.82

Table 1. Parameters A_u and B_u for calculating run-up (Martín *et al.*'s method). Source: Negro *et al.* (2008)

$$13. \quad P_h(z) = \mu \rho g (s + A_c - z) \quad \text{foundation level } < z < A_c + s$$

The value of μ is obtained from Figure 8.

The uplift pressures are calculated by following the condition of continuity at the foot located on the attack side. It indicates the following for the sheltered point.

- There is nil uplift pressure for the dynamic pressures if the foundation level is above high tide. If not, it will be calculated taking into account the buoyancy of the submerged part and the wave transmitted through the porous medium (λP_d).
- In the case of pseudohydrostatic pressure, the uplift pressure at the sheltered point is calculated according to a trapezoidal law on the side of safety as to a parabolic pressure distribution, (P_{ra}), according to the method developed by Martín *et al.* (1995) (Figure 9).

The notation is defined as: α is the non-dimensional parameter containing information on the celerity of the mass of water s wide at level A_c ; β is the angle formed by the main armour layer slope with the horizontal (sexagesimal degrees); λ is the non-dimensional parameter introducing the berm's effect into the pressures on the wave wall's protected area; μ is a non-

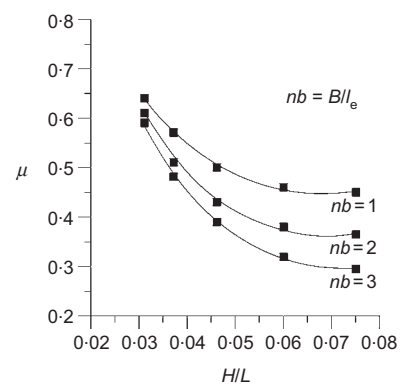


Figure 8. Adjusted values of μ (Martín *et al.*'s method). Source: Negro *et al.* (2008)

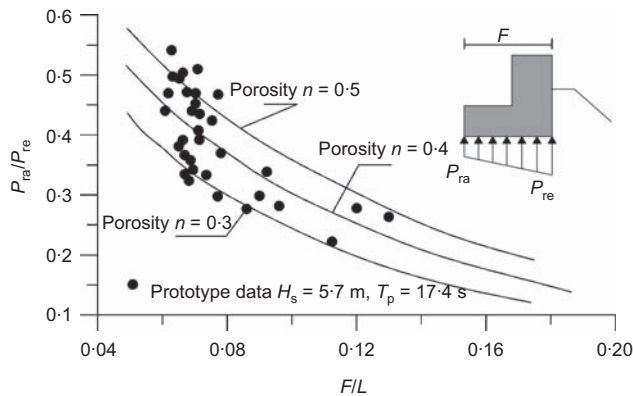


Figure 9. Uplift pressures (Martín *et al.*'s method). Source: adapted from Martín *et al.* (1995)

dimensional parameter less than 1; ρ is the sea water density ($\text{kN}\cdot\text{s}^2/\text{m}^4$) such that $\rho g = \gamma_w$ (kN/m^3); A_c is the level of the rock fill or block berm's crown (m); B is the width of the rock fill or block berm (m); d is the depth (m); g is the acceleration due to gravity (m/s^2) such that $\rho g = \gamma_w$ (kN/m^3); H is the design wave height (m); H_b is the height of breaking wave (m); H_s is the height of significant wave (m); I_r is the Iribarren number in deep water conditions; L is the length of design wave (m); I_e is the equivalent side of the main armour layer's units (m); P_b is the pseudohydrostatic pressure (kN/m^2); P_d is the dynamic pressure (kN/m^2); R_a is the run-up, namely the maximum ascent of the sheet of water on the slope, an undefined case (m); s is the width of the sheet of water ascending on the slope at level A_c (m); T is the design wave period (s); T_p is the peak period (s); and z is the vertical coordinate with its origin at design sea level and positive in an ascending direction (m).

The dynamic and pseudohydrostatic pressures are not added up for calculating the wave wall's stability but each is separately dealt with and the sliding and overturning safety coefficients are obtained for each using the Goda criterion (Goda, 1985).

2.2 Force diagram

2.2.1 Bradbury and Allsop (1988)

This method starts from Jensen's work (Jensen, 1984) and determines the maximum horizontal force F_H on the wave wall.

Jensen does not give a specific equation but, based on test results, points out that there is a practically linear relationship between two factors

$$\frac{F_H}{\gamma_w h b L_p} \text{ and } \frac{H_S}{\Delta h}$$

where F_H is the maximum horizontal force (kN/m); γ_w is the specific weight of sea water (kN/m^3); h is the height of the wall (m); b is the width of the wall (m); L_p is the peak period wave length (m); H_S is the significant wave height (m); Δh is the vertical distance from still-water to the crest of the armour layer (m).

Bradbury and Allsop (1988) propose the following equation from the foregoing

$$14. \quad \frac{F_H}{(\rho g h_f L_p)} = \frac{a H_S}{A_c} - b$$

where F_H is the maximum horizontal force (kN); ρ is the sea water density ($\text{kN}\cdot\text{s}^2/\text{m}^4$); g is the acceleration due to gravity (m/s^2), such that $\rho g = \gamma_w$ (kN/m^3); h_f is the height of wave wall (m); L_p is the peak period wave length (m); H_S is the significant wave height (m); A_c is the protection layer crown height (m); and a, b are empirical coefficients as shown in Figure 10.

It assumes a rectangular horizontal pressure distribution to obtain an estimate on the side of safety

$$15. \quad p_H = F_H/h_f \text{ (uniform horizontal pressure)}$$

The maximum vertical pressure coincides with the maximum horizontal. It takes up a triangular distribution having the maximum value at the front and reduces linearly to zero in the extradors with which the maximum vertical force is

$$16. \quad F_V = (\rho g B_c L_p / S)(a H_S / A_c - b)$$

where S is a safety factor and B_c is the width of the crown wall. The method proposes a coefficient of friction μ with a value of 0.50 for calculating the wave wall's stability. This is the method proposed by CIRIA-CUR (1991).

2.2.2 Pedersen and Burcharth (1992)

The horizontal force is obtained from the pressure records by spatial integration. The study confirms the investigations provided by Jensen, explained in the foregoing section. The equation proposed by Pedersen and Burcharth is as follows

$$17. \quad \frac{F_{h,0.1\%}}{\rho g h_f L_p} = a \left(\frac{H_S}{A_c} + b \right)$$

where: $F_{h,0.1\%}$ is the horizontal force associated with a surplus of 0.1% (kN); ρ is the water density ($\text{kN}\cdot\text{s}^2/\text{m}^4$); g is the acceleration due to gravity (m/s^2), such that $\rho g = \gamma_w$ (kN/m^3); h_f is the height of wave wall (m); L_p is the peak period wave length (m); H_S is the significant wave height (m); A_c is the protection layer's crown height (m); and a, b are non-dimensional coefficients to be determined with specific tests.

Coefficient	Massive concrete blocks				Natural rock			
	No break		Break		No break		Break	
	$\varepsilon_{dp} \leq 3.25$	$\varepsilon_{dp} > 3.25$	$\varepsilon_{dp} \leq 3.25$	$\varepsilon_{dp} > 3.25$	$\varepsilon_{dp} \leq 3.25$	$\varepsilon_{dp} > 3.25$	$\varepsilon_{dp} \leq 3.25$	$\varepsilon_{dp} > 3.25$
<i>a</i>	0.0121	0.0118	0.0100	0.0093	0.0118	0.0103	0.0114	0.0044
<i>b</i>	-0.0094	-0.0119	-0.0067	-0.0084	-0.0115	-0.0129	-0.0103	-0.0024

Table 2. Coefficients for calculating horizontal force (Berenguer and Baonza’s method). Source: Negro *et al.* (2008)

2.2.3 Berenguer and Baonza (2006)

Determining forces on a wave wall calls for calculating the run-up according to the formulation indicated in the method proposed by Berenguer and Baonza (2006)

$$18. R_{u2\%} = 0.86 \zeta_p^{0.54} H_s \gamma_\theta$$

where $R_{u2\%}$ is the ascent of the sheet of water exceeded by 2% of the waves (m); H_s is the significant wave height (m); ζ_p is the Iribarren number referring to the length of wave associated with the peak period; and γ_θ is the obliqueness factor according to De Waal’s criterion (De Waal *et al.*, 1996).

2.2.3.1 HORIZONTAL FORCE

$$19. F_X = \gamma_w h_h^{0.5} L_p^{1.5} \left(a \frac{R_{u2\%}}{A_c^{2/3} B^{1/3}} + b \right) \text{ if } (R_{u2\%} > R_c)$$

$$20. F_X = \gamma_w (R_{u2\%} - W_c)^{0.5} \times L_p^{1.5} \left(a \frac{R_{u2\%}}{A_c^{2/3} B^{1/3}} + b \right) \text{ if } (R_{u2\%} \leq R_c)$$

The *a* and *b* coefficients of the foregoing equations are obtained from Table 2.

2.2.3.2 VERTICAL FORCE (UPLIFT PRESSURE)

$$21. F_Y = \gamma_w h_h^{0.5} L_p^{1.5} \left(a \frac{R_{u2\%} - W_c}{A_c^{2/3} B^{1/3}} + b \right) \text{ if } (R_{u2\%} > R_c)$$

$$22. F_Y = \gamma_w (R_{u2\%} - W_c)^{0.5} \times L_p^{1.5} \left(a \frac{R_{u2\%} - W_c}{A_c^{2/3} B^{1/3}} + b \right) \text{ if } (R_{u2\%} \leq R_c)$$

The *a* and *b* coefficients of the foregoing equations are obtained from Table 3.

If a wave wall with a base *F* is considered, the total uplift pressure will be

$$23. F_{YT} = F_Y + F_Y = F_Y + (0.017L_p - 0.109F)(F - 0.043L_p)$$

2.2.3.3 MOMENT DUE TO THE HORIZONTAL FORCE

$$24. M_X = \gamma_w h_f L_p^2 \left(a \frac{F_X}{\gamma_w h_f^{0.5} L_p^{1.5}} + b \right) \text{ if } (R_{u2\%} > R_c)$$

Coefficient	Massive concrete blocks				Natural rock			
	No break		Break		No break		Break	
	$\varepsilon_{dp} \leq 3.25$	$\varepsilon_{dp} > 3.25$	$\varepsilon_{dp} \leq 3.25$	$\varepsilon_{dp} > 3.25$	$\varepsilon_{dp} \leq 3.25$	$\varepsilon_{dp} > 3.25$	$\varepsilon_{dp} \leq 3.25$	$\varepsilon_{dp} > 3.25$
<i>a</i>	0.0015	0.0004	0.0001	0.0014	0.0024	0.0014	0.0016	0.0001
<i>b</i>	0.0020	0.0028	0.0037	0.0017	0.0013	0.0012	0.0025	0.0034

Table 3. Coefficients for calculating vertical force (Berenguer and Baonza’s method). Source: Negro *et al.* (2008)

Coefficient	Massive concrete blocks				Natural rock			
	No break		Break		No break		Break	
	$\varepsilon_{dp} \leq 3.25$	$\varepsilon_{dp} > 3.25$	$\varepsilon_{dp} > 3.25$	$\varepsilon_{dp} > 3.25$	$\varepsilon_{dp} \leq 3.25$	$\varepsilon_{dp} > 3.25$	$\varepsilon_{dp} \leq 3.25$	$\varepsilon_{dp} > 3.25$
<i>a</i>	0.113370	0.109490	0.119270	0.062150	0.123997	0.096651	0.121971	0.071884
<i>b</i>	0.000190	-0.000080	0.000040	0.000060	-0.000002	-0.000067	-0.000072	0.000008

Table 4. Coefficients for calculating the moment of the horizontal force (Berenguer and Baonza's method). Source: Negro *et al.* (2008)

25.

$$M_X = \gamma_w (R_{u2\%} - W_c) \times L_p^2 \left[a \frac{F_X}{\gamma_w (R_{u2\%} - W_c)^{0.5} L_p^{1.5}} + b \right] \text{ if } (R_{u2\%} \leq R_c)$$

The *a* and *b* coefficients of the foregoing equations are obtained from Table 4.

2.2.3.4 MOMENT DUE TO THE VERTICAL FORCE (UPLIFT PRESSURE)

$$M_{YT} = F_Y (F - 0.018L_p) + (F_{YT} - F_Y)$$

26.
$$\times \left(\frac{0.046L_p - 0.217F}{0.102L_p - 0.651F} \right) (F - 0.043L_p)$$

where F_X is the horizontal force exerted by waves on the wave wall (kN); F_{YT} is the vertical force (uplift pressure) exerted by waves on the wave wall (kN); M_X is the moment due to horizontal force (kN.m); M_Y is the moment due to vertical force (uplift pressure) (kN.m); γ_w is the specific weight of water (kN/m³); W_c is the wave wall's foundation level as to sea level (m); R_c is the wave wall's crown level as to sea level (m); L_p is the wave length at foot of breakwater as to peak period (m); A_c is the berm crown level as to sea level (m); B is the width of crown berm (m); h_f is the height of the wave wall (m); $\text{tg } \alpha$ is the tangent of the armour layer slope angle with the horizontal; and θ is the waves' angle of incidence (sexagesimal degrees).

3. Critical analysis of the methods

The foregoing methods have their ranges of use and application. They have been generally obtained under laboratory conditions, which should be borne in mind to be able to properly interpret the results. The peculiarities of each method as taken from the pertinent investigations are discussed hereafter. The analysis is carried out in a chronological order to observe the variation of the parameters used in the calculations in time.

3.1 Iribarren and Nogales (1954)

- The method proposed by Iribarren and Nogales (1954) stands out for being the first wave wall calculation method.

It dates from 1954 and no other alternative calculation method appeared until 1984 (Günbak and Göcke, 1984).

- The breakwater on which the pressures indicated are applied corresponds to a specific geometry recommended by

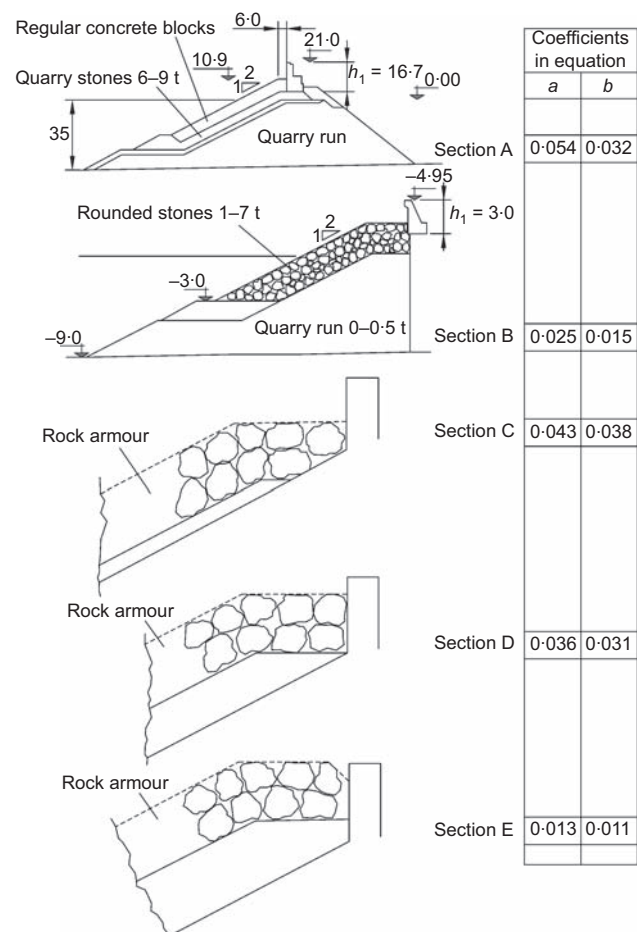


Figure 10. Values of empirical parameters according to geometries tested for the Bradbury and Allsop method. Source: Negro *et al.* (2008)

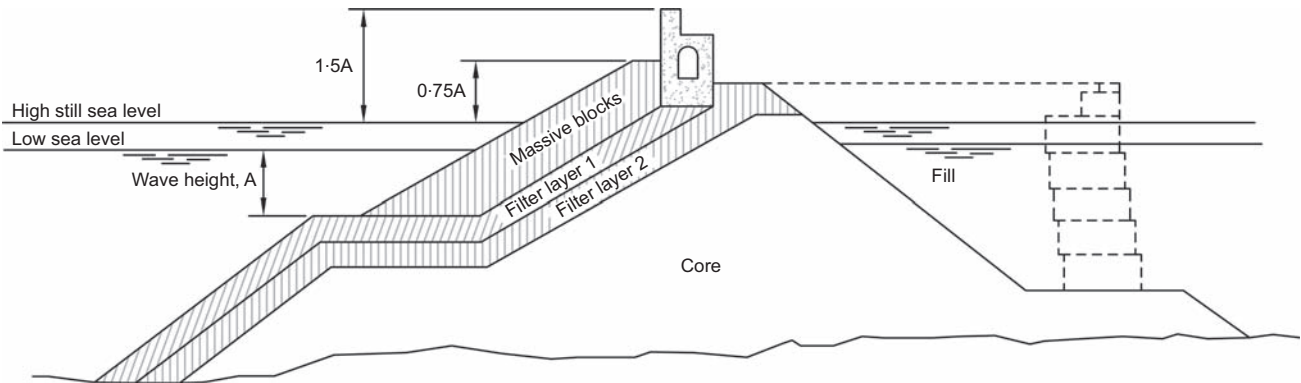


Figure 11. Typical breakwater cross-section according to Iribarren and Nogales. Source: Negro *et al.* (2008)

Iribarren and Nogales. Applying the method for another type of geometry would not be recommendable. The geometry Iribarren and Nogales indicate is that as represented by Figure 11. The method requires the waves to arrive at the wave wall broken, whether because they are broken when incident on the breakwater or because they break on the armour layer's slope.

- It only indicates horizontal pressures and does not specify the uplift pressures appearing on the wave wall with a wave height of $A = 1.25H$, where $H = 2h$.
- It poses passing pressures that represent the horizontal speed of the wave crest and dynamic pressures corresponding to the fall of molecules into the trough. The resulting horizontal pressure is the sum of them both.
- The block layer located in front of the wave wall reduces the total pressures 50%.
- The parameters intervening do not include the length of the

protection berm located facing the wave wall. It does not assess the influence of the number of units in the armour layer facing the wall on reducing pressure.

- It does not indicate the wave height to be considered in the calculation ($H_{1/250}$, $H_{2\%}$, H_S , $H_{1/10}$).

3.2 Günbak and Göcke (1984)

- Günbak and Göcke (1984) devised this procedure in order to calculate 'wave screen' type structures. Nevertheless, it does not specify the range of validity or application.
- In the tests on which it is based, waves break before becoming incident on the wave wall.
- The authors point out that the method has been thought up to be applied in Mediterranean ports. This aspect is relevant since each sea behaves differently and is not entirely picked up by the Günbak and Göcke method parameters, such as

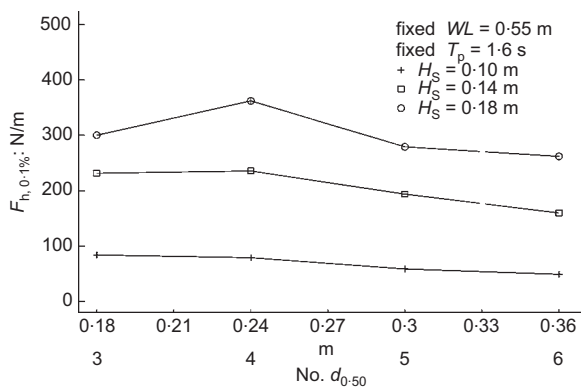


Figure 12. Influence of the berm's width on the total horizontal force on the wave wall according to Pedersen and Burcharth. Source: adapted from Pedersen and Burcharth (1992)

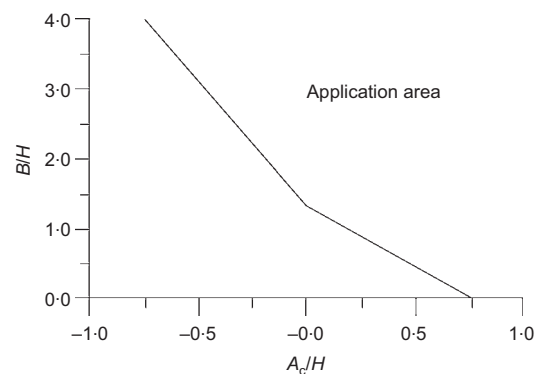


Figure 13. Range of validity of Martin *et al.*'s method. Source: Negro *et al.* (2008)

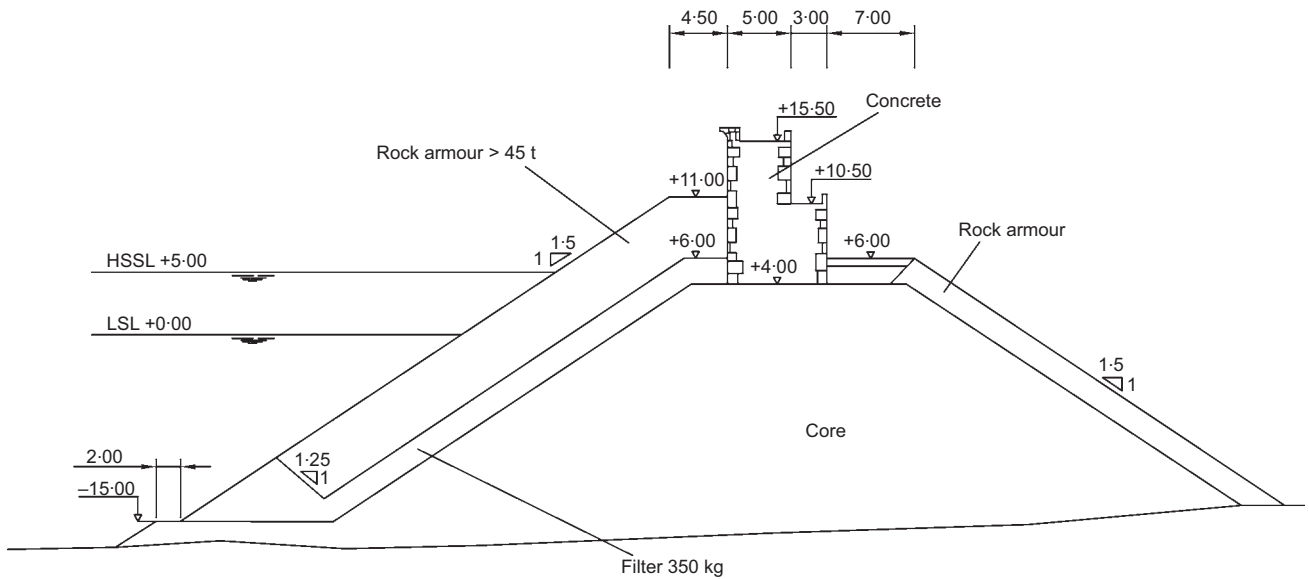


Figure 14. Mutriku new breakwater section S-3 (dimensions and elevations in m); HSSL, high still sea level; LSL, low sea level

the wave length and number of waves (a particularly relevant factor in breakwaters as made patent in Van der Meer's equation (Van der Meer, 1993). Applying it to ports located in other masses of water, the North Atlantic for example, should be previously studied.

- There are two pressures: shock and hydrostatic. The resulting pressure is the sum of them both.
- It considers that the existence of a berm facing the wave wall reduces the shock pressure at the foot of the wall 50%. This reduction is gradual, with no leap.
- It does not define the wave height which has to be used nor

to what location the Iribarren number refers, which is that used to calculate run-up and pressures.

- It carries out a number of tests on eight different cross-sections, though with the same slope ($1V/2H$). Nevertheless, the actual ports where it contrasts the results (Tripoli and Antalya) have slopes of $1V/1.5H$ and $1V/2.5H$.
- Based on the tests, it concludes that there should be at least three units of the armour layer facing the wave wall in order to be able to consider the 50% reduction in the impact pressures. The length of the armour layer located facing the wave wall does not intervene in the formulation and it is

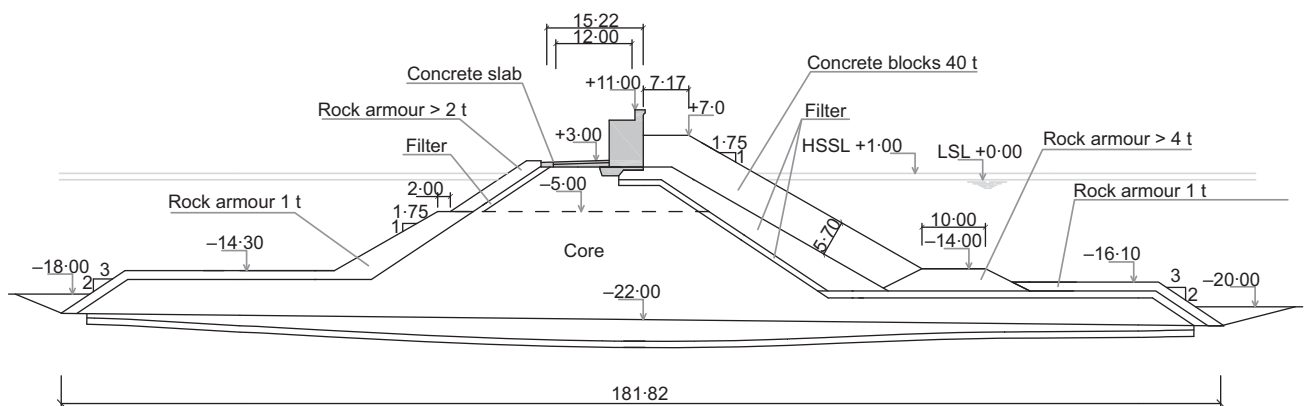


Figure 15. Barcelona South breakwater section (dimensions and elevations in m); HSSL, high still sea level; LSL, low sea level

Mutriku new breakwater

Crown wall weight ($\gamma_{HM} = 2.3 \text{ t/m}^3$)	169 t/m
Crown wall weight arm	4.51 m
Stability moment due to crown wall weight	762 t.m
Width of the foundation	8.00 m
Specific weight of water	1.02 t/m ³
Acceleration due to gravity, such that $\rho g = \gamma_w$ (kN/m ³)	9.81 m/s ²
Specific weight of armour elements	2.3 t/m ³
Maximum variability of sea water level	5.00 m
Return period of the structure	112 years
Calculation wave height (Iribarren and Nogales, Günbak–Göcke and Martín <i>et al.</i>). $H_{2\%} \approx 1.416H_s$	10.62 m
Significant wave height	7.50 m
Peak period wave length in toe berm	182.65 m
Calculation wave period	15.7 s
Peak period	15.7 s
Wave incidence angle	0.0 degrees
Main armour elements equivalent length	> 45 t → 2.6 m
Friction coefficient	0.5

Barcelona South breakwater

Crown wall weight ($\gamma_{HM} = 23 \text{ kN/m}^3$)	133 t/m
Crown wall weight arm	2.88 m (without slab)
Stability moment due to crown wall weight	383 t.m
Width of the foundation	6.85 m (without slab)
Specific weight of water	1.02 t/m ³
Acceleration due to gravity, such that $\rho g = \gamma_w$ (kN/m ³)	9.81 m/s ²
Specific weight of armour elements	2.3 t/m ³
Maximum variability of sea water level	1.00 m
Return period of the structure	500 years
Calculation wave height (Iribarren and Nogales, Günbak–Göcke and Martín <i>et al.</i>). $H_{2\%} \approx 1.416H_s$	10.62 m
Significant wave height	7.5 m
Peak period wave length in toe berm	182.62 m
Calculation wave period	14 s
Peak period	14 s
Wave incidence angle	0.0 degrees
Main armour elements equivalent length	50 t → 2.79 m
Depth	20 m
Friction coefficient without key	0.5
Friction coefficient with key	0.6

Table 5. Case data

therefore assumed that it must always be equal to the length equivalent to three units of the armour layer.

3.3 Bradbury and Allsop (1988)

- The method proposed by Bradbury and Allsop (1988) is based on Jensen's investigations.
- The formulation offers the horizontal force as the result.

From there it passes to horizontal pressure, adopting a uniform pressure distribution over the whole height of the wave wall, and to uplift pressures, assuming a triangular distribution although it indicates that rectangular should be taken to be on the side of safety. It is supposed, with these pressures, that the arms of the forces would be obtained in order to calculate the moment, but no specification is given.

Example	Method	Loads				
		F_H	F_V	M_{FH}	M_{FV}	
Mutriku new breakwater	Iribarren and Nogales	962	531	3414	2832	
	Günbak and Göcke	943	530	4267	2826	
	Bradbury and Allsop	913	318	5252	847	
	Pedersen and Burcharth	–	–	–	–	
	Martín <i>et al.</i>	Dynamic	550	153	3547	817
		Pseudohydrostatic	265	277	1015	1231
	Berenguer and Baonza		1067	251	4313	1184
Barcelona South breakwater $\mu = 0.5$	Iribarren and Nogales	694	455	1874	2076	
	Günbak and Göcke	397	295	1179	1349	
	Bradbury and Allsop	1409	536	6342	1225	
	Pedersen and Burcharth	–	–	–	–	
	Martín <i>et al.</i>	Dynamic	382	103	1996	472
		Pseudohydrostatic	165	187	499	712
	Berenguer and Baonza		916	294	4795	1068
Barcelona South breakwater $\mu = 0.6$	Iribarren and Nogales	694	455	1874	2076	
	Günbak and Göcke	397.2	295.4	1179.1	1349	
	Bradbury and Allsop	1409	536	6342	1225	
	Pedersen and Burcharth	–	–	–	–	
	Martín <i>et al.</i>	Dynamic	382	103	1996	472
		Pseudohydrostatic	165	187	499	712
	Berenguer and Baonza		916	294	4795	1068

F_H , horizontal force (kN/m); F_V , vertical force (kN/m); M_{FH} , instability moment associated with F_H (mKN/m); M_{FV} , instability moment associated with F_V (mKN/m).

Table 6. Example structures. Results obtained: forces

- It defines the wave height and wave length to be used in the calculations.
- The formulation has empirically obtained parameters that depend on the geometry of the berm and the filter and it is therefore assumed that all the peculiarities of those elements are taken into account. The reference coefficients for cases which do not match the cross-sections indicated would have to be obtained.
- The breakwater cross-sections have a $1V/2H$ slope. It points out that this slope usually takes a more unfavourable run-up.
- The formula does not show the armour layer's width. Nevertheless, the cross-sections in Figure 10 show the following: section A shows a filter of 6 to 9 t, which indicates that the armour layer must be formed by blocks between 60 and 140 t. This section encloses a berm distance of 6 m, which allows two blocks, not three, to be placed. Sections C, D and E show a berm with a width corresponding to three units.
- It does not include any reduction in horizontal pressures because of the existence of an armour layer facing the wave wall.

3.4 Pedersen and Burcharth (1992)

- The Pedersen and Burcharth (1992) method calculates the horizontal force with a surplus of 0.1% obtained by integrating the pressures recorded in laboratory tests. This formula has to be adjusted with the a and b parameters.
- The tests carried out hardly have any overtopping and the calculation method is therefore more reliable in breakwaters displaying a low overtopping rate.
- It is a formulation that is very similar to Bradbury and Allsop's. It does not pretend to be a new equation but confirms that given by Jensen (1984).
- It observes that the influence of the berm's width on the intensity of the pressures is low, as shown in Figure 12.
- It assumes that most of the load is due to hydrostatic pressure.
- It does not say how to calculate the vertical force to be considered.
- The empirical a and b parameters it introduces into the equation show the characteristics of the berm and the filter, the same as the Bradbury and Allsop (1988) method. However, these parameters have to be determined in laboratory tests. It does not provide predetermined values as in Bradbury and Allsop's method.

Example	Method	Stability analysis		
		CSD	CSV	
Mutriku new breakwater	Iribarren and Nogales	0.60	1.40	
	Günbak and Göcke	0.61	1.12	
	Bradbury and Allsop	0.75	1.29	
	Pedersen and Burcharth	–	–	
	Martín <i>et al.</i>	Dynamic	1.40	1.92
		Pseudohydrostatic	2.66	6.30
	Berenguer and Baonza		0.67	1.49
Barcelona South breakwater $\mu = 0.5$	Iribarren and Nogales	0.63	0.94	
	Günbak and Göcke	1.30	2.11	
	Bradbury and Allsop	0.28	0.41	
	Pedersen and Burcharth	–	–	
	Martín <i>et al.</i>	Dynamic	1.60	1.68
		Pseudohydrostatic	3.46	6.26
	Berenguer and Baonza		0.56	0.58
Barcelona South breakwater $\mu = 0.6$	Iribarren and Nogales	0.76	0.94	
	Günbak and Göcke	1.56	2.11	
	Bradbury and Allsop	0.34	0.41	
	Pedersen and Burcharth	–	–	
	Martín <i>et al.</i>	Dynamic	1.93	1.68
		Pseudohydrostatic	4.15	6.26
	Berenguer and Baonza		0.68	0.58

CSD, sliding safety coefficient; CSV, overturning safety coefficient.

Table 7. Example structures. Results obtained: stability analyses

3.5 Martín *et al.* (1995)

- The Martín *et al.* (1995) method gives two pressures: a dynamic one (due to the deceleration of the wave's front) and a pseudohydrostatic one (due to the descent of the mass of water accumulated on the structure). It does not consider them to be concomitant, and, therefore, they are not added together and it studies the wave wall's stability in each case separately. It is the only calculation method operating in this way. The vertical pressure is parallelepiped or triangular depending on whether it is due to dynamic or pseudohydrostatic pressures.
- The methodology is applicable while the following conditions are fulfilled
 - (a) waves reach the wave wall either broken or in run-up
 - (b) the angle of incidence may be up to ± 20 sexagesimal degrees
 - (c) must be inside the application region marked in Figure 13.
- It is an exhaustive method that considers a large number of factors not taken into account in the other procedures or included together with others in empirical parameters.
- It does not clearly define the wave height, wave length or period to be used in the calculation, and it is not known whether $H_{2\%}$, H_S , $H_{1/10}$, H_{\max} are involved.
- It considers a reduction in horizontal pressures, both dynamic and pseudohydrostatic, in the part protected by an armour layer. A coefficient of reduction (λ) is applied in the dynamic ones that depends on the armour layer's width. The pseudohydrostatic pressures have a coefficient of reduction (μ) that depends on the number of units in the armour layer. It therefore assesses the length of the rock layer. The graphs where these coefficients are obtained give values close to 0.50. It considers the reduction in horizontal pressures with one or two units in the armour layer, unlike the indications of Günbak–Göcke and Pedersen–Burcharth, who only consider the reduction in pressures as from three units.
- The A_u and B_u parameters necessary to calculate the run-up display incoherence in the values adopted for cubes and blocks. A lesser run-up is obtained with the values for the cube-type elements than with the block-type elements. It should be the opposite since cubes break the sheet of water less and they also shore up, presenting a smoother surface and, therefore, increasing the run-up figure.

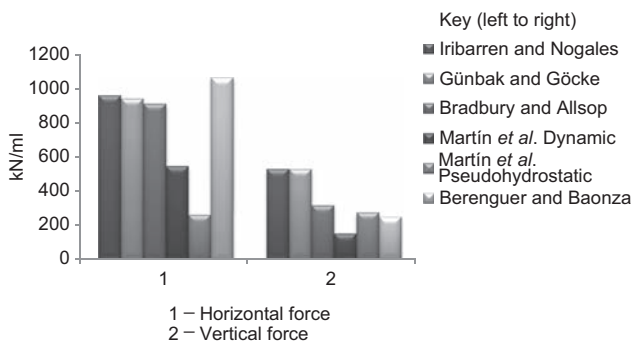


Figure 16. Mutriku new breakwater – forces

- The λ and μ parameters are obtained for 120 t blocks since they are calculated as from the data-taking campaign on the Príncipe de Asturias breakwater in Gijón.
- As indicated in the foregoing point, the method has been proven for the Príncipe de Asturias breakwater, which is not completely representative due to the peculiar cross-section thereof since the wave wall is built and has foundations directly on the core, formed by 90 t blocks with a high gap index between them. It is not the classic wave wall foundation.

3.6 Berenguer and Baonza (2006)

- The Berenguer and Baonza (2006) method obtains horizontal and vertical pressure laws with tests with irregular waves. Using these pressures, it calculates the horizontal and vertical force as well as the moments associated with each one.
- It has been contrasted with numerous actual cases located along the whole Spanish coast.
- The waves do not break in all the tests carried out until reaching the breakwater.
- The method for calculating the forces on the wall is designed for the case of the breakwater armour layer's elements being natural rock or cubes. The run-up intervenes in this calculation with its own formulation obtained from tests with cross-sections where perforated cubes and perforated antifers are used.
- Some blocks in the armour layer were deliberately placed in the tests relating to forces on the wave wall with a certain displacement to reflect the actual state of a breakwater that has withstood storms throughout its useful life.

4. Example structures

The six methods related have been applied to three example structures

- Mutriku new breakwater (Guipúzcoa) (Figure 14)
- Barcelona South breakwater (Figure 15) with key
- Barcelona South breakwater (Figure 15) without key

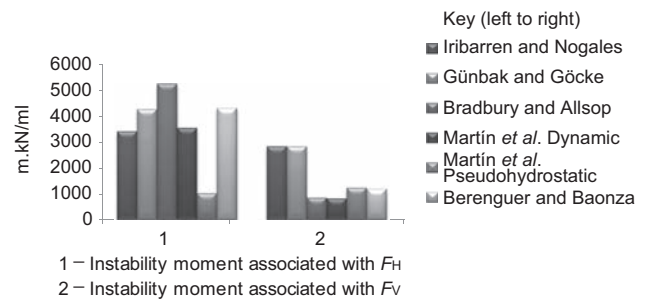


Figure 17. Mutriku new breakwater – instability moments

The case data are presented in Table 5 and results obtained are given in Tables 6 and 7. These results are also shown graphically in Figures 16 to 22.

Results obtained show that the sections given do not fit in the sections defined by Bradbury and Allsop. Quite different results were obtained depending on the section chosen. Section A was chosen for the calculations shown in Tables 6 and 7.

Wave actions were first calculated for each of the methods shown. Coefficients of sliding and overturn stability were calculated with the results.

In general, a high dispersion in results can be observed.

In the Mutriku example the following characteristics were observed.

- Horizontal forces have similar values in all methods except in Martín *et al.*'s, which gives less forces. The moment associated presents more variation depending on the method.
- Vertical forces present more dispersion than horizontal ones. Moments associated have even more dispersion.
- Iribarren and Nogales' is the most conservative method. New methods give more accurate actions because, on

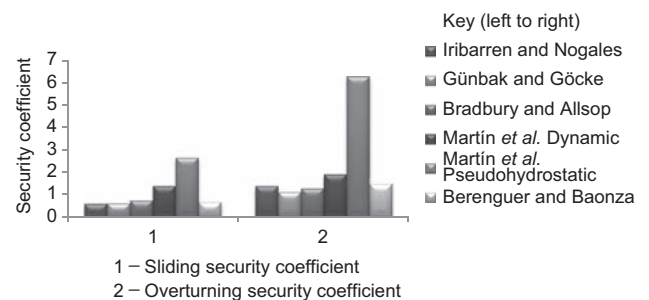


Figure 18. Mutriku new breakwater – safety coefficients

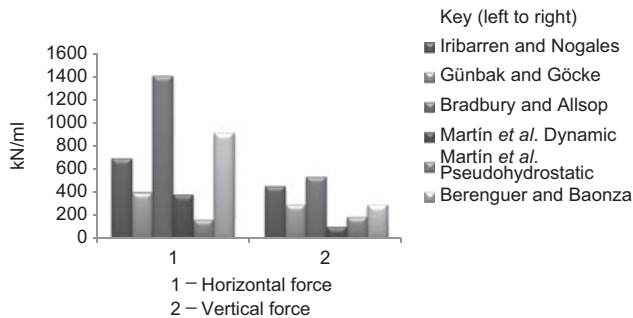


Figure 19. Barcelona South breakwater – forces

including more calculation parameters, the results are more particularised to the specific study case.

In the Barcelona South breakwater, the following characteristics can be observed.

- There is a high dispersion in horizontal and vertical forces.
- As far as the stability calculation is concerned, methods have a different sensitivity to a variation in the friction coefficient.
- Recent methods (Martín *et al.* and Berenguer and Baonza) do not give similar results.

5. Proposal for a new formulation

Methods currently used to determine wave actions on a wave wall are based on geometric and wave criteria. The existing formulations do not embody the waves' energy treatment.

This is the reason why a study is envisaged on a new formulation in which the incident waves on the wall intervene in the form of energy transmitted through the porous medium which is the armour layer, determining the diagram of pressures above the block berm in the case of run-up and the dissipated one through the units making up the slope. The first schemes used employ the

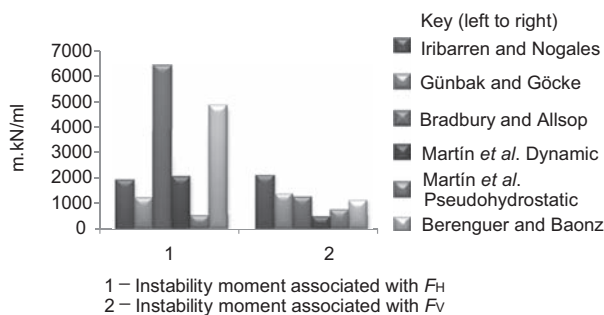


Figure 20. Barcelona South breakwater – instability moments

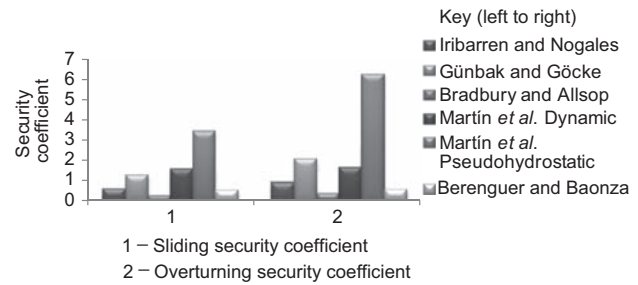


Figure 21. Barcelona South breakwater – safety coefficients ($\mu = 0.5$)

DELOS (Burcharth *et al.*, 2007) formulae for transmission and run-up and pressure adjustments with rectangular laws above the berm of protection units.

6. Conclusions

The following conclusions may be drawn in the light of these state-of-the-art review analyses.

- All methods are based on more or less extensive laboratory tests except for Iribarren and Nogales' which, on the other hand, is very old.
- The tests on which the methods analysed are based cover a broad series of states of the sea (significant wave height, periods and so on) although the same does not apply for the geometry of the armour layer's crown, such that the slopes considered are of only two types: $1V/1.5H$ or $1V/2H$ and the range of the number of armour layer units is limited except in Martín *et al.* (Table 8). This leads to the application of a method to slopes that largely diverge from those used in the tests on which those methods are based, possibly giving rise to erroneous results. The same occurs in those cases where the actual geometry cannot be fitted into that of the profile tested and the coefficients obtained therefrom (the case of Bradbury and Allsop). In such a case, reduced scale tests have to be carried out (the case of Pedersen and Burcharth), and this involves a high cost due to the formulations for

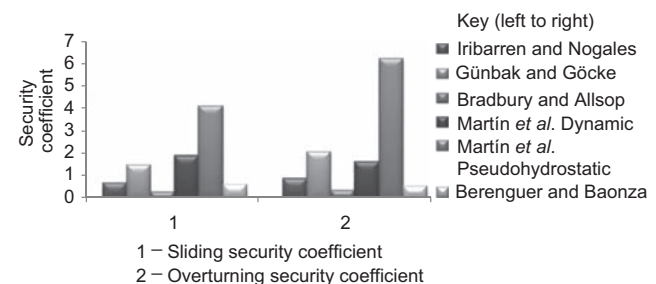


Figure 22. Barcelona South breakwater – safety coefficients ($\mu = 0.6$)

		Iribarren and Nogales	Günbak– Göcke	Bradbury– Allsop	Pedersen– Burcharth	Martin <i>et al.</i>	Berenguer– Baonza
Year published		1954	1984	1988	1992	1995	2006
Test based on:	Waves	No	Regular	–	–	In situ measured campaigns (Gijón)	Irregular JONSWAP
	Slope	No	Run-up with $5V/2H$ Pressures with $1V/2H$	$1V/2H$	$1V/1.5H$	$1V/1.5H$	Run-up with $1V/1.5H$ and $1V/2H$ Forces with $1V/1.5H$
	Armour layer units	No	Rock fill	Rock fill and blocks as per figures	Unspecified	Rock fill, cubes, tetrapods, quadrapods, tribar and dolos	Run-up with cubes and perforated antifers. Forces with blocks and rock fill
Formulation in:	Horizontal pressures	Horizontal and vertical pressures	Horizontal and vertical forces	Horizontal forces	Horizontal and vertical pressures	Horizontal and vertical forces with their respective moments	Horizontal and vertical forces with their respective moments
Criteria		Geometric and undulatory	Geometric and undulatory	Geometric and undulatory	Geometric and undulatory	Geometric and undulatory	Geometric and undulatory
Variables intervening	Wave wall height	X	X	X	X	X	X
	Width of wave wall foundations	X	X	X	–	X	X
	Height of emerged berm	X	X	X	X	X	X
	Width of emerged berm	–	–	–	–	X (in the form of no. of units)	X
	Slope angle	–	X	–	–	X	X
	Wave height	X	X	Significant wave height	Significant wave height	X	Significant wave height
	Period	–	X	–	–	X	Average and peak periods
	Wave length	–	–	X (referring to the peak period)	X (referring to the peak period)	X	X (at foot of breakwater referring to T_p and T_m)

Table 8. Comparison of the methods analysed (continued
on next page)

	Iribarren and Nogales	Günbak– Göcke	Bradbury– Allsop	Pedersen– Burcharth	Martín <i>et al.</i>	Berenguer– Baonza
Year published	1954	1984	1988	1992	1995	2006
Wave's angle of incidence with the perpendicular to the breakwater	–	–	–	–	X	X
Type of armour layer units	–	–	–	–	X	–
Armour layer's porosity	–	–	–	–	X	–
Others	–	–	Predetermined empirical coefficients	Empirical coefficients to be determined in each case	–	–

Table 8. Continued

pre-sizing the wave wall while tests allow the design to be optimised. Numerous systematic tests would need to be performed in order to increase the number of cases and obtain the adjustment parameters.

- Some methods do not offer equations for all the actions necessary to calculate the wave wall's stability (Table 8); Iribarren and Nogales do not indicate the uplift pressure distribution; Bradbury and Allsop adopt a triangular uplift pressure as an approximation from which they obtain the vertical force, but they say that a rectangular distribution on the side of safety may also be taken; and Pedersen and Burcharth only determine the horizontal force.
- In some cases, there is a lack of definition in the undulatory parameters to be used in the calculations.
- The methods do not include all characteristics in the formulations that determine the breakwater. The most complete in this aspect are those of Martín *et al.* and Berenguer and Baonza.

7. Recommendations

- The engineer must bear in mind that there is a heavy dispersion of results between methods and it is therefore advisable to use more than one method to determine results coming closer to reality.
- Studies by Camus Braña and Flores Guillén (2004) also show the dispersion of results obtained with different methods but point out that Pedersen's formulation is more

reliable. In the case of being outside the range of application, good results are also obtained with this method.

- The foregoing studies (Camus Braña and Flores Guillén, 2004) also claim that a better approximation to the physical process is obtained with the method of Martín *et al.* because it separately analyses the dynamic and pseudohydrostatic forces.
- Some calculation methods (Iribarren and Nogales, Martín *et al.*) indicate their range of validity. The others do not and it is therefore understood that they are applicable in any case. Nevertheless, it is recommended to know the conditions under which the said formulations were obtained before applying them to a case far from the original parameters.
- Existing calculation methods are recommendable for prior sizing. In any event, tests on a physical model are recommended to confirm the final design. These tests may even lead to the cross-section's optimisation.

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