CHAPTER 119

H_o Parameter for Preliminary Design of Conventional Breakwater Structural Head. Data Analysis of Spanish North Coast Harbours

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SUMMARY

The determination of the response of a Coastal Structures to waves is almost determined by model tests of using formulae fitted to model data. Using results of Aalborg University, 1987-88, and Collecting Spanish data from the North Coast, 1988, we approach for preliminary design the dimensionless wave height parameter, H_o , Van der Meer, 1988 for conventional breakwater, structural head, and start of damage and collapse of the structure.

The effect of wave length, storm duration, and the influence of the breakwater geometry, interlocking blocks and water depth will be include in the future for better calibration of Van der Meer parameter

1. INTRODUCTION

The determination of the response of a Coastal Structure or a breakwater, trunk or head, to waves is almost always determined by model tests. Sometimes, Hydraulics Engineers use formulae fitted to model tests. The main problem depends on the stochastic nature of loads and random placement of the armour units in breakwater.

Brosen et al, 1974, demostrated that the Hudson formula is not valid for complex interlocking types of armour blocks with respect the slope angle. Van der Meer, 1988, proposes the dimensionless wave height parameter, H_o , $H_s/\Delta D$, to give the relationship between different structures and represents an important variable in a stability formula.

The agreement will be conclude:

$$\frac{u^2}{\Delta gD} \simeq \frac{H_s}{\Delta D} = (cotg \ \alpha \ . \ K_D)^{1/3} = N_s$$

Shield parameter Van der Meer Hudson Brebner and Donelly

Using model tests of Aalborg University and collecting the Spanish Data, we approach for preliminary design the H_o parameter and

$$\frac{H^{2/3} \cdot L^{1/3}}{\Delta D}$$

for conventional breakwater, structural head, start of damage and collapse of the structure.

2. CLASSIFICATION OF RUBBLE MOUND STRUCTURES

Rubble mound structures can be classified by use of the static parameter, H_{o} ,

$$H_o = \frac{H_s}{\Delta D_{n50}}$$

where:

H_s Significant wave height (m)

 Δ Relative mass density (-)

 $\frac{\gamma}{\gamma_{w}} - 1 \qquad \begin{array}{c} \gamma & \text{Specific weight of an individual unit (t/m^3)} \\ \gamma_{w} & \text{Specific weight of seawater (t/m^3)} \end{array}$

D_{n50}Nominal diameter (m)

Dynamically stable structures are characterized by the forming of a profile under wave attack. In this case, conventional breakwater can be classified with the combined wave heightwave period parameter H_{a} . T_{a} , defined by,

$$H_o = \frac{H_s}{\Delta D_{n50}}$$
, dimensionless wave height parameter

$$T_o = T_m \cdot \sqrt{\frac{g}{D_{n50}}}, \quad dimensionless \text{ wave period parameter}$$

As a simple remark, rubble mound structures can be classified (Fig. 1).



Fig. 1. Type of structure as a function of $\frac{H}{\Delta D}$, Van der Meer, 1988

| $H_{o} < 1$ | Caissons, seawall, verti | ical breakwater. |
|--------------|--------------------------|------------------|
| | Instantaneous Failure. | |
| U < 1 | Conventional breakwater | Gradual Failura |

| 1 | < H _o < | 4 | Conventio | onal breakwat | er. | Gradual Fa | ilure. |
|---|--------------------|---|-----------|---------------|-----|------------|--------|
| 3 | $< H_{\circ} <$ | 6 | Special | breakwater, | S | shaped, | berm |

| $6 < H_{o} < 20$ | Rock slopes. Nominal | diameter | relatively |
|------------------------|----------------------|----------|------------|
| | small. | | |
| $15 < H_{\circ} < 500$ | Gravel beaches. | | |
| $H_{o} > 500$ | Sand beaches. | | |

The hydraulic stability can be classified including static and dynamic profiles,

| Η, . | T, | < | 100 | Static criteria |
|---------------|----|---|-----|------------------|
| H_{\circ} . | T, | > | 100 | Dynamic criteria |

This classification is a great contribution to the total uncertainty of the coastal structures response and follows single estimation like Professor Suarez Bores 1968-74-78 and the evolution of breakwater structures, Goda and Tanimoto, 1991 (fig. 2).



Fig. 2. Evolution of breakwater structures, Goda-Tanimoto, 1991

- Structural response and design

Deterministic

- . Univariate: $H_{1/n}$, return period
- . Multivariate: $H_{1/n}$, T, N, S, P, risk analysis
- Failure response
 - . Instantaneous failure
 - . Gradual failure

- Stochastic nature of the loads

- . Conventional breakwaters, $H_{1/n}$. Gradual failure by the action of waves train.
- . Rigid breakwaters, H_{max} , N Collapse by the action of a single wave.
- Optimum design
 - . Risk analysis
 - . Economic analysis
 - . Stochastic analysis
 - . Multivariate risk analysis

Burcharth and Frigaard, Aalborg University, studied the front profile development, erosion and deformation of slopes, when exposed to oblique waves. Their range of tests correspond:

$$3,50 < H_{o} < 7,10$$

the following stability factor values were proposed by Burcharth et al, 1988, for the start of significant transport of stones along the structures (angle of wave incidence $\leq 30^{\circ}$).

| | H_s |
|---|------------------|
| | ΔD_{n50} |
| . Trunks exposed to steep oblique waves | 4,50 |
| . Trunks exposed to long oblique waves | 3,50 |
| . Round heads | 3 |

This research and study have been the basis to preliminary design of conventional breakwaters, structural head, $1 < H_o < 4$, with a theoretical approach and using the long series of

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Spanish Data from the North Atlantic Sea, the North Coast, and collecting model tests results.

3. DATA ANALYSIS AND ENGINEERING METHODS FOR PREDICTING THE DESIGN OF STRUCTURAL HEAD. START OF DAMAGE AND COLLAPSE OF THE STRUCTURE. SPANISH NORTH COAST EXAMPLES.

The waves reaching the Cantabrian Sea Coast, Spanish North Coast, are generated in the North Atlantic by the action of extratropical storm. The polar mass oscillations through the year define the path of the storms through the Bay of Biscay. Under such conditions, the winds are from West to North, including North West and NNW with the dominant gradient wind speed of 40 m/sg.

In that case, there are maximum frequency and intensities of waves, reaching values of significant wave height over ten meters with peak period of eighteen seconds. This is representative of the rough conditions in deepwater.

The studies reported here were done using the Spanish Data of conventional breakwaters, more than 100 ports and harbours, in the Cantabrian Sea Coast, and with a theoretical approach, obtained the following results,

| Tetrápodos | 2 | "en desorden" | 1.20 | 1,25 | 1,5 |
|--------------------|---|---------------|------|------|-----------|
| - | 2 | •. | 1,30 | 1,35 | 2,0 |
| " | 2 | | 1,80 | 1,90 | 3.0 |
| Tribars | 2 | "en desorden" | 1,10 | 1,15 | 1,5 |
| ** | 2 | | 1.15 | 1,20 | 2.0 |
| • | 2 | - | 1,30 | 1,35 | 3.0 |
| Dolos | 2 | "en desorden" | 1,50 | 1,50 | 2,0 |
| " | 2 | " | 1.60 | 1.65 | 3.0 |
| Cubo modificado | 2 | "en desorden" | - | 1,55 | 1,5 a 3,0 |
| Hexápodos | 2 | "en desorden" | 1,65 | 1,35 | 1.5 a 3.0 |
| Tribars . | 1 | uniforme | 1,60 | 1,60 | 1,5 a 3.0 |

Fig. 3. Coefficients for conventional breakwaters head

| Type of Breakwater | Weight of the main armour layer (t) | Nominal Diameter (11) | Significant wave height (m) | H。 (~) |
|------------------------------|--|-----------------------------|-----------------------------------|-----------|
| Fuenterrabía, quarrystone | 9 t | 1,50 m | 5,00 m | 2,04 |
| Orio, blocks $\gamma = 2,80$ | 13 t | 1,66 m | 5,85 m | 2,02 |
| Getaria, quarrystone | 20 t | 1,96 m | 6,63 m | 2,12 |
| Bermeo, 1 ^a Phase | 50 t | 2,75 m | 7,13 m | 1,93 |
| Bermeo, 2ª Phase | 85 t | 3,30 m | 9,00 m | 2,03 |
| Bilbao, Punta Lucero | 150 t | 4,00 m | 10,10 m | 1,88 |
| Lastres | 40 t | 2,55 m | 6,80 m | 1,98 |
| Gijón | 120 t | 3,70 m | 9,60 m | 1,94 |
| Candás (Failure) | 27 t | 2,25 m | 4,95 m | 1,64 |
| San Esteban | 125 t | 3,70 m | 8,75 m | 1,77 |
| Cudillero | 60 t | 2,92 m | 8,00 m | 2,04 |
| Burela | 72 t | 3,10 m | 7,45 m | 1,79 |
| San Ciprián | 90 t | 3,35 m | 8,90 m | 1,98 |
| Cillero | 28 t | 2,25 m | 6,25 m | 2,06 |
| Cariño, quarrystone | 12 t | 2,05 m | 5,00 m | 1,96 |
| Malpica | 120 t | 3,68 m | 10,70 m | 2,16 |
| Lage | 20 t | 2,05 m | 5,00 m | 1,96 |
| Finisterre | 15 t | 1,85 m | 5,00 m | 2,09 |
| Panjón | 5 t | 1,28 m | 3,18 m | 1,85 |

Another conventional breakwaters were designed after this preliminary analysis in the Spanish North Coast. The results were:

| Type of Breakwater | Weight of the main armour layer (t) | Nominal Diameter (m) | Significant wave height (m) | H _。 (-) |
|----------------------|--|----------------------------|--------------------------------------|-----------------------|
| Zurriola, blocks | 45 t | 2,55 m | 8,20 m | 1,96 |
| Cala Bens, La Coruña | 125 t | 3,75 m | 10,00 m | 1,98 |

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The relative mass density coefficient (\triangle) oscilate between 1,63 for quarrystone, 1,34 for concrete blocks, tetrapod, acropod ... etc, and 1.731 for high density natural blocks.

This analysis reported before is relation with the classic point of view for calculating head in conventional breakwater,

$$W_{head}$$
 = Coefficient (> 1,00) x W_{Hudson}

The dimensionless coefficient accounts for all variables like (Fig. 3)

- Shape of the armour layer
- Number of units of the main armour layer
- Manner of placing armour layer
- Type of wave attacking structure (breaking or nonbreaking)
- Slope

Looking at the statistics of the present analysis and "only" for preliminary design in conventional breakwater head, start of damage, we can conclude in first step, fitting the model with nature test (Fig. 4-9),

$$H_o = \frac{H_s}{\Delta D_{Dn50}} \approx 2,00$$

Iribarren's model tests, 1965, presented in PIANC Congress Stockolm, demonstrated the evolution of a slope affected with groups of waves, obtaining the complete results of the evolution of damage in the armour layer of a conventional breakwater. The three phases are:

- Total stability
- Partial stability, start of damage
- Inestability, moderate damage to collapse of the structure

Stablishing the state of damage under the action of a monochromatic wave train, the slope is indefinite and the units of the main armour layer are quarrystone, concrete blocks and tetrapod (Fig. 10).

The results of damage levels with two diameter thick armour layer, Van der Meer, 1988, show the limits of the adimensional parameter $S = A/D^2$, a cross sectional eroded area, D nominal diameter of the armour layer (Fig. 11).

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Fig. 4. Cross section Bilbao breakwater







Fig. 6. Cross section principe de Asturias, Gijón breakwater









Fig. 9. Cross section San Ciprian breakwater

| | | Damage Levels | | | |
|--------------------------|-----------|-----------------|--------------------|--------------------|--|
| l ype of armour layer | Slope | Start of damage | Moderate damage | Filter visible | |
| Stone armour layer | 1,5/3,0 | S = 2 | S = 4/8 | S <u>></u> 8/12 | |
| Block | 1,5/2,0 | Nod = 0 | Nod=0,50/1,50 | Nod = 2 | |
| Tetrapod | 1,50/1 | Nod = 0 | Nod=0,50/1,00 | Nod=1,50 | |
| Acropod | 1,33/1,50 | Nod = 0 | | Nod=0,50 | |

Losada et al (1986) defined three hydrodinamic damage criteria,

- 1. Incipient damage
- 2. lribarren's damage
- 3. Destruction

In order to be more precise, another stage of damage is included as a visual response, it is called incipient destruction, Vidal et al (1991). For a mound breakwater consisting of a main layer, secondary layer and core, $P \approx 0.40$, the definitions of the four damage criteria are as follows:

- 1. Initiation of damage. This level of damages defines the condition attained when a certain number of armour units are displaced from their original position to a new one at a distance equal to or larger than a unit length. Holes larger than average porous size are clearly appreciable.
- 2. Iribarren's damage. This damage occurs when the extension of the failure area on the main layer is so large that the wave action may extract armour units placed on the lower layer of the main armour layer.
- 3. Initiation of destruction. A small number of units, two or three, in the lower armour layer are forced out and the waves work directly on pieces of the secondary layer.
- 4. Destruction. Pieces of the secondary layer are removed. If the wave height does not change the mound will definitely be destroyed and it will cease to give the level of service defined in the design. It is called Filter visible.

As a consecuence, for a preliminary design of mound breakwaters, and start of damage, we propose,

$$H_o = \frac{H_s}{\Delta D_{n50}} = 2,00$$

After the model tests performed by Iribarren, 1965, the medium relationship between the wave height of start of damage and the wave height of destruction or filter visible, is equal for stone armour layer, blocks and tetrapods and the value is 0,62.

Using the adimensional parameter for preliminary design and start of damage, $H_o \approx 2,00$ and with the relation between the wave height start of damage and filter visible-destruction, the value of H_o will be,

$$\frac{H_{start of domage}}{H_{filter visible, destruction}} = 0,62$$

 $H_o \approx 1.25$ Filter visible, collapse of structure thus, the following point will emphasized, with the concept of weight of the main armour layer,

$$W_{destruction} = (\frac{1}{0.62})^3 \quad W_{start of damage}$$

 $W_{destruction} = 4,10 \quad W_{start of damage}$

The results with oblique incidence do not show the same value for every Kind of armour piece, obtaining relation of 0,69 in rip-rap, armour layer with stone, and 0,73 with blocks and tetrapods.

However, the first analysis for a preliminary design can be consider stable.

The effect of wave length, storm duration, and the influence of the breakwater geometry, interlocking blocks, and water depth,

$$H_o = \frac{H_s}{\Delta D_{nso}} = 2,00$$
 Start of damage

$$H_o = \frac{H_s}{\Delta D_{n50}} = 1,25$$
 Filter visible, destruction

The effect of wave length, storm duration, and the influence of the breakwater geometry, interlocking blocks, and water depth, slope and other factors will be search in a future. Model test will provide the following conclusions,

$$H_{o} = \frac{H_{s}}{\Delta D_{n50}} = \frac{H_{s}^{2/3} \cdot L^{1/3}}{\Delta D_{n50}} = (cotg \ \alpha \cdot K_{D})^{1/3} = N_{s}$$

$$\frac{H_{s}^{2/3} \, . \, L^{1/3}}{\Delta D_{n50}} \, . \, f(N)$$

where,

| L | Wave length, $\frac{1}{25} < \frac{d}{L} < \frac{1}{2}$ | (m) |
|----------------|---|-----|
| N | Number of active waves | (-) |
| α | Slope of the breakwater | (°) |
| K _D | Stability coefficient, Hudson | (-) |
| Ns | Stability coefficient, Brebner and Donelly | (-) |
| H。 | Van der Meer's parameter | (-) |
| H | Significant wave height | (m) |
| D _n | Nominal diameter | (m) |
| | $Dn = {}^{3}\overline{\left(\frac{W}{L}\right)},$ | |

$$Dn = \sqrt[3]{\left(\frac{W}{\gamma}\right)},$$

W Weight of an individual unit of the main armour layer (t) Specific weight of an individual unit(t/m³)

γ



Fig. 10. Stability curve, Iribarren, 1965



Fig. 11. Value of parameter "S", A cross sectional eroded area; D, nominal diameter of the armour layer

4. CONCLUDING REMARKS

According the model tests of Burcharth and Frigaard, and the data analysis and only for preliminary design and of no other more qualified information is available, the Spanish experience in the Cantabrian Sea Coast recommends for conventional breakwater, structural head, the value of the dimensionless wave height parameter $H_o = H_s/\Delta D$.

$$H_o = \frac{H_s}{\Delta D_{n50}} \approx 2,00$$
 Start of damage

$$H_o = \frac{H_s}{\Delta D_{nso}} \simeq 1,25$$
 Destruction, collapse

The effect of wave length, storm duration, and the influence of the breakwater geometry, interlocking blocks and water depth are now in study and they will be include in the future as a function of L, N, D, H_s, D, d and α .

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