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PHYSICAL MODELLING OF A CAISSON BREAKWATER UNDER IMPULSIVE CYCLONIC CONDITIONS: CASE OF PORT EAST (LA REUNION ISLAND)

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ABSTRACT

The GPMDLR (Port Authority of La Réunion Island, France) has requested ARTELIA to carry out the preliminary studies for the enlargement of the containers terminal in Port East. In this context, the laboratory of ARTELIA has performed 2D physical model testing to evaluate the behavior of caissons exposed to cyclonic conditions. The solution based on a vertical breakwater has been proposed as an alternative to a rubble mound breakwater given the lack of quarries suitable to provide big quantities of large rocks. Main objectives of the study were: the evaluation of overtopping rates (mean and max), the verification of the stability of foot protection blocks and stones, and the recording of wave pressures (gauges) and loads (scale) on the caissons. Given the fact that most of the waves broke violently upon the caisson, the nature of efforts was impulsive and the scaling to prototype by classical Froude similarity principles was not appropriate. Results were therefore amended following Cuomo's correction factors for the Froude scaling law (Cuomo et al. 2010). Times series of efforts (forces and pressures) were used to feed numerical models (static and dynamic) allowing for the verification of caissons' geotechnical stability and the design of concrete parts. This article collects the concerns, feedback and further questioning related to the conducted experiment.

KEYWORDS: Vertical breakwater, Caisson, Physical model, Impact waves, Stability, Cyclonic conditions.

1 INTRODUCTION TO THE STRUCTURE

The breakwater subject of the study is made of precast reinforced concrete caissons crowned by a three-level stepped superstructure built with in-situ concrete. The reclamation, made of quarry run, leans on the backside of the structure, which serves as a containing support. Main dimensions and location of the cross section are indicated in the figure below.

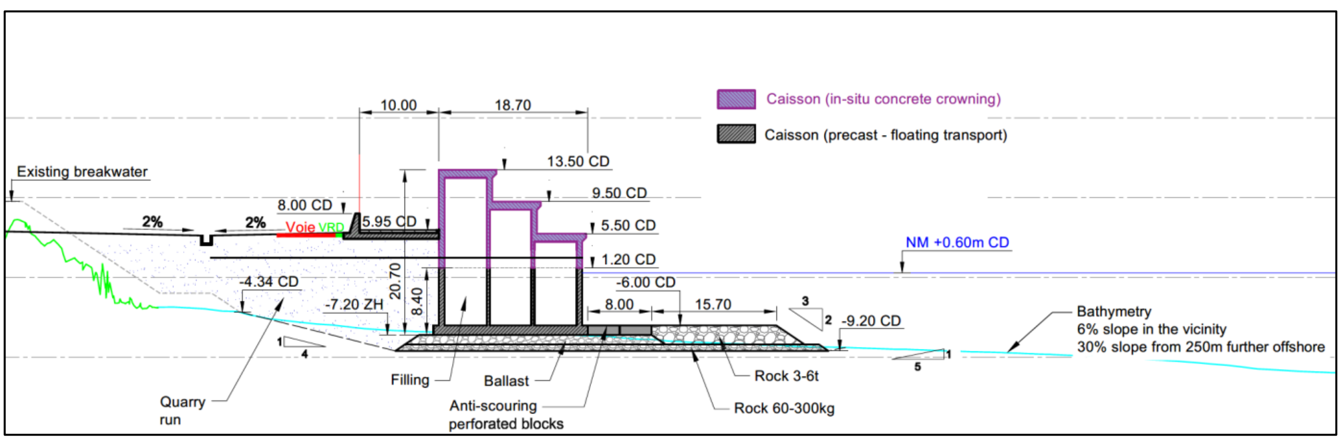

Figure 1. Tested cross-section

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2 DESIGN CONDITIONS

1.1 Generalities

Thanks to its privileged location on the North West coast of Reunion Island, Port East is very well sheltered from austral and trade waves all year round. Yet, waves raised by tropical storms and cyclones passing by the north of the island can occasionally jeopardize the breakwaters (ca. once a year). Resulting swells due to squalls can present very important wave heights and long peak periods. The features of the waves depend mainly on the distance between the trajectory of the depressions and the port. Design wave characteristics (Hm0, Tp, duration and spectrum) and associated water levels corresponding to the dimensioning extreme wave conditions have been provided by the Client. Waves and water level conditions used for design are shown in the table below.

Table 1. Relevant conditions

N°	Description	Water level [mCD]	Hm0 at the toe (-8,5 mCD) [m]	Hm0 offshore (-50 mCD) [m]	Tp [s]	Duration [h]
1	Setting	+ 1,02	2,0	-	12	2 h
2	Low level conditions	+ 1,28	7,9	11,8	16	6h+6h
3	High level conditions 2070	+ 2,50	9,2	11,9	16	6h+6h

For testing time series have been derived from a synthetic Jonswap spectrum with γ=3.3 peak enhancement parameter.

1.2 Influence of bathymetry in wave break conditions

The toe of the breakwater is located at -8.50mCD, which is a shallow depth with regard to the size of cyclonic waves. The seabed slope in front of the breakwater is about 6% until an inflection point 250m offshore (depth about -20mCD), where the slope becomes much steeper (30%) and plunges very quickly to a very important depth.

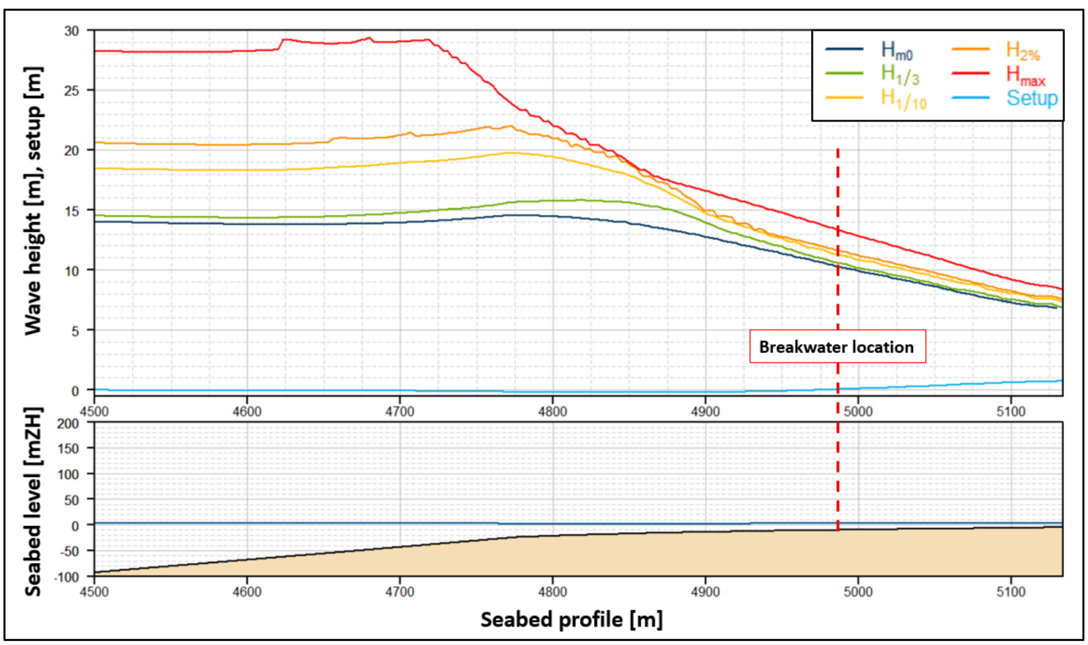

Figure 2. 1D wave propagation general pattern from offshore up to breakwater location

Under these conditions, the long swell induced by the offshore cyclone propagates quite freely up to the location of the slope inflection, where it is slowed-down by the seabed, becomes very steep, and ends up violently collapsing upon the caisson, as shown in the pictures below.

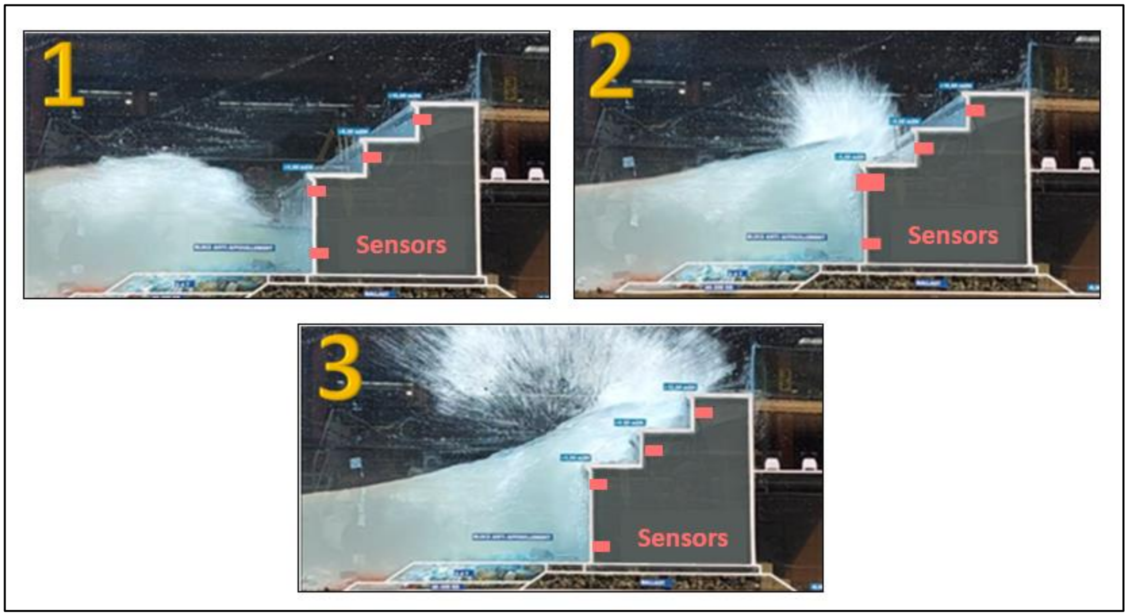

Figure 3. Wave impacting against the caisson

3 PHYSICAL MODELLING

3.1 Description of the modelling

The modelling has been implemented in one of the two wave flumes of ARTELIA laboratory with main dimensions being the following:

- Width: 1 m
- Maximum depth: 2.0 m at the wave generator
- Length: 41 m

The flume is equipped with a wave generator able to produce random waves according to a predefined wave energy spectrum, consistent with the concerned ranges of cyclonic wave heights and periods.

The caisson has been scaled to 1/59, which was found to be the largest geometrical fitting compatible with the size of the flume, the capacity of the wave-maker, and the range of measuring devices such as load cells and gauges. The seabed has been reproduced by a non-erodible wood surface, following the most critical profile with regard to wave impacts.

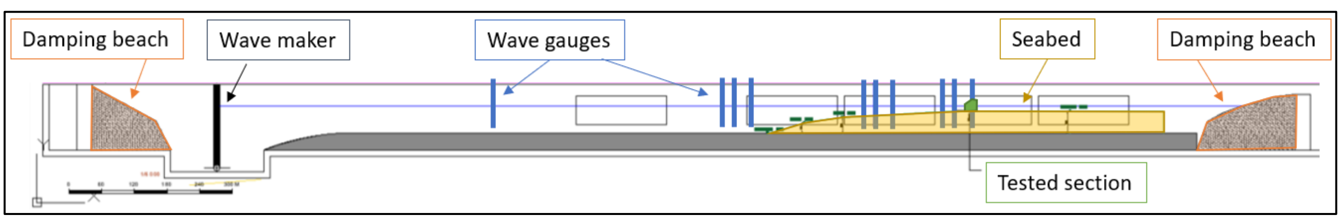

Figure 4. Model implementation in the flume

The setup of the model was conceived to allow the evaluation of overtopping rates, toe protection stability and wave efforts upon a one only assembly. Therefore, the model was split into two different regions, each of them accounting for the study of certain parameters as shown in the picture below. It has to be noted that the left half of the caisson was artificially founded on rougher material (2-4t stones instead of 60-300kg) in order to avoid the possible underestimations of water pressures underneath the structure caused by the insufficient permeability induced by the scaling of smaller gradings.

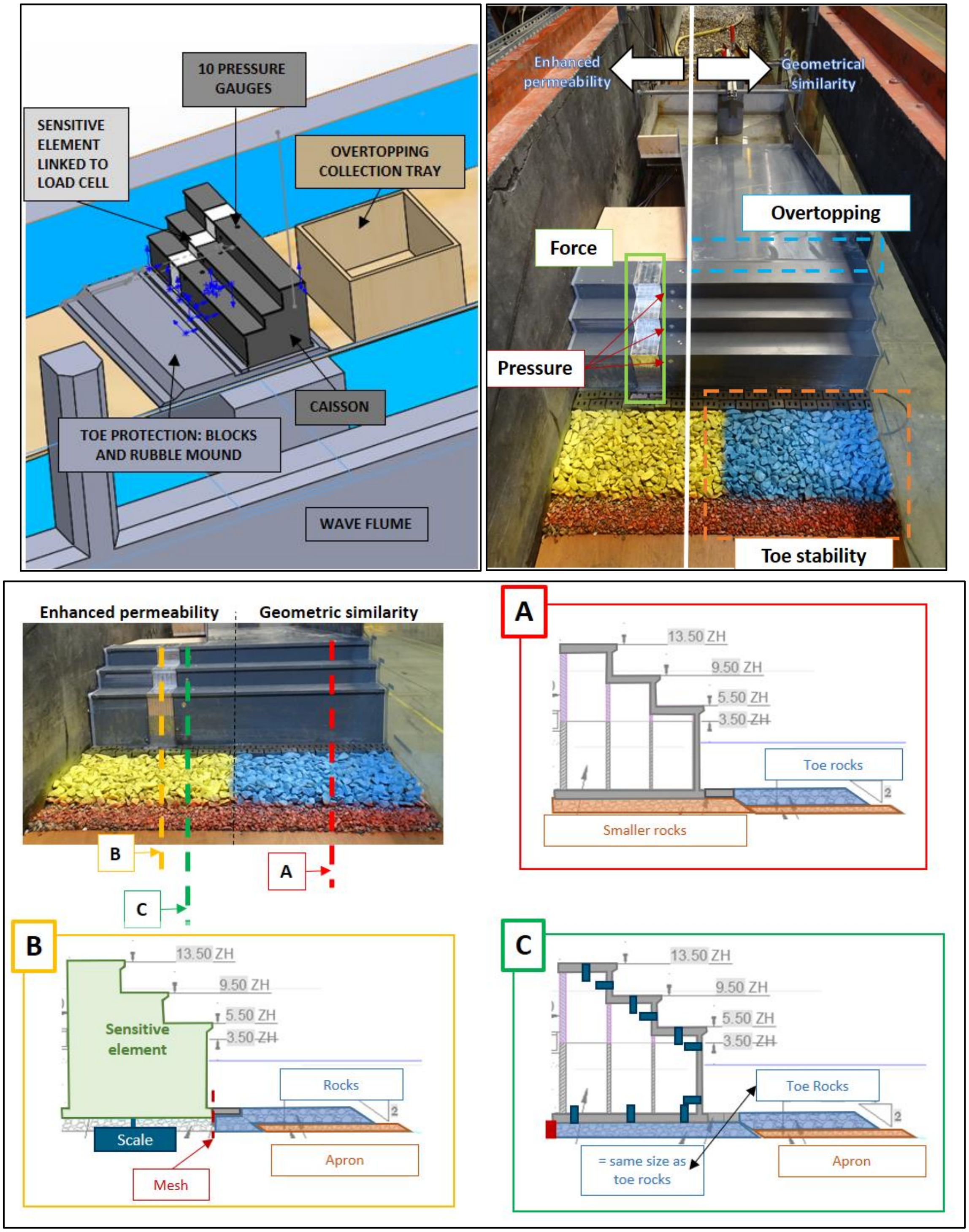

Figure 5. Set up of model in ARTELIA's 2D wave-flume

3.2 Overtopping rates

The breakwater under study shall be designed such as the overtopping discharge does not jeopardize the physical integrity of the breakwater in case of massive overtopping. The three steps constituting the crowning top were equipped each with 50cm protuberant bullnoses in order to counteract the ease of green water to run-up over the caisson. The overtopping water volume has been collected in a tray during the runs and weighed. The recording of cumulated volumes with a rear gauge has allowed to identify maximum instant overtopping. The mean overtopping discharge (l/m/s) is derived by dividing the total overtopping volume by the total test duration.

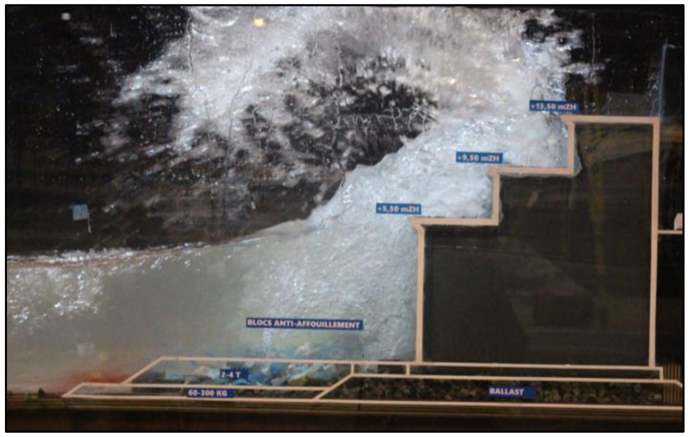

Figure 6. Breaking wave splashing and bouncing back on the caisson

Targeted order of magnitude for reasonable cyclonic mean overtopping was set along with the Client to be about 200l/s/m. Recorded rates after high-level conditions were yet between 260 l/s/m and 275 l/s/m. We note however, that splashes did not reach a long distance behind the caisson and that green water was very scarce since the bullnoses proofed to be quite efficient for the task of intercepting water jets. Drainage gutters have been sized for the proper evacuation of water back to the sea (by gravity).

3.3 Verification of stability of foot protection

Several configurations of the toe berm have been tested under low water levels (which is a very pessimistic approach under squall conditions) until the finding of a satisfactory arrangement in terms of stability. Visual observations and counting of movements have been used for the evaluation. The original toe was made of 2 rows of 15tons blocks and rubble mound apron of 2-4t. Both the breaking of the waves on the toe and the run-down proved to be very aggressive and the toe was not stable. To increase the toe stability, four modifications have been tested, especially: lowering of the toe level by 1m, increase of the toe rocks size (3-6t), increase of the blocks size by joining them two by two (30t).

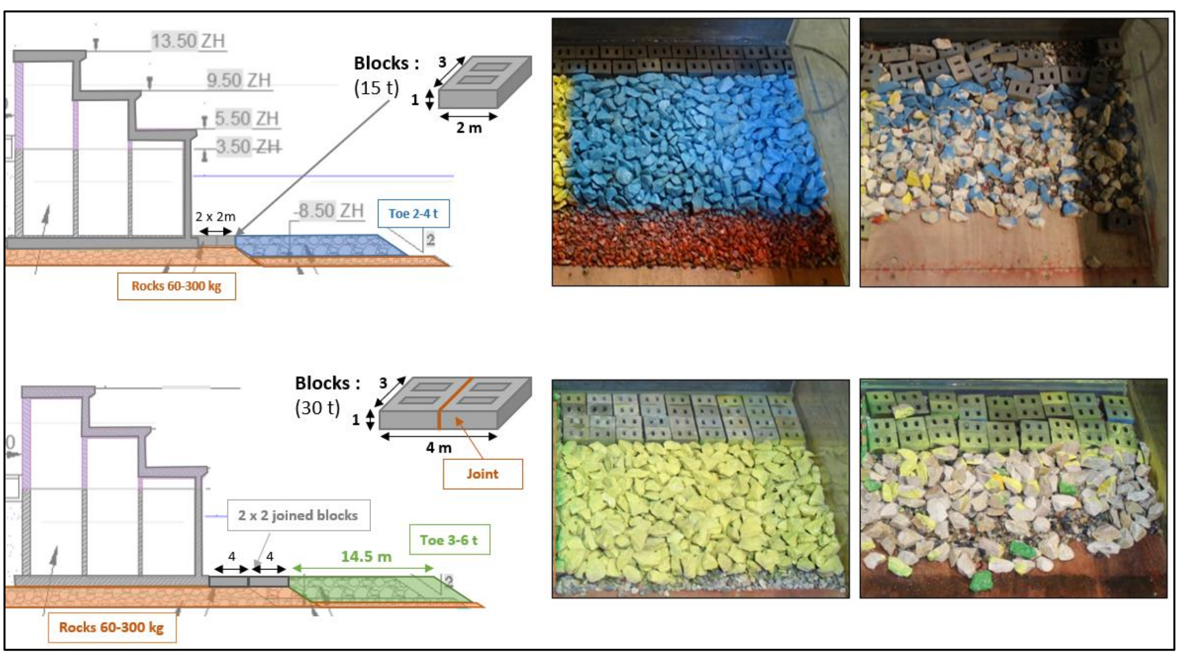

Figure 7. First (up) and final (below) toe configuration – After 12h hours of cyclonic waves (low water level)

The amended section shown in the upper figure has provisionally been validated attending for a future verification upon a 3D model test (not yet performed).

3.4 Wave pressures and loads

3.4.1 Setup of instrumentation

The model was equipped with the following gauges for the of record wave pressures on the front, on the bottom and on the top of the caisson:

- 4 horizontal gauges (H1 to H4, see figure below),
- 6 vertical gauges (V1 to V6, see figure below).

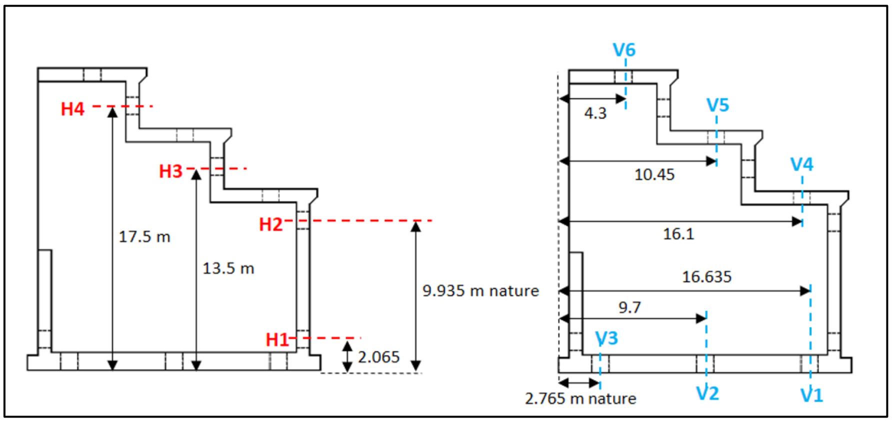

Figure 8. Location of horizontal (H) and vertical (V) gauges

A 6D scale (used in 3D mode) was installed for the recording the overall forces and torque acting over a slice of the structure at any time during the simulation.

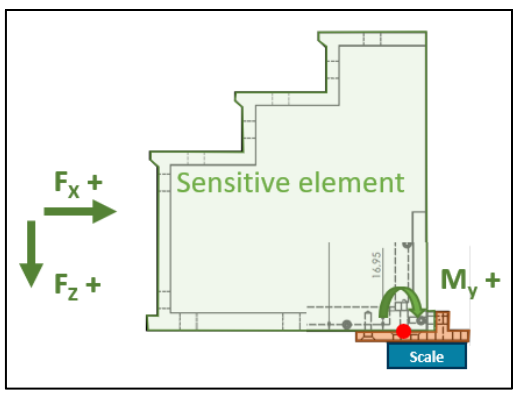

Figure 9. Installation of load cell

3.4.2 Expected order of magnitude of efforts upon the caisson

Preliminary calculations of expected efforts have been carried out with Goda-Takahashi formulae (*Random Seas and Design of Maritime Structures; Yoshimi GODA; 2010*) leading to the results below.

Table 2. Table of results with Goda-Takahashi

Pressures:

- $p_1 = 149.56\ kPa$
- $p_2 = 137.5\ kPa$
- $p_3 = 140\ kPa$
- $p_4 = 60\ kPa$
- $p_u = 109\ kPa$

Forces and torque:

- $P = 2\,520\ kN/m$
- $M_P = 22\,840\ kN.m/m$
- $U = 980\ kN/m$
- $M_U = 11765\ kN.m/m$

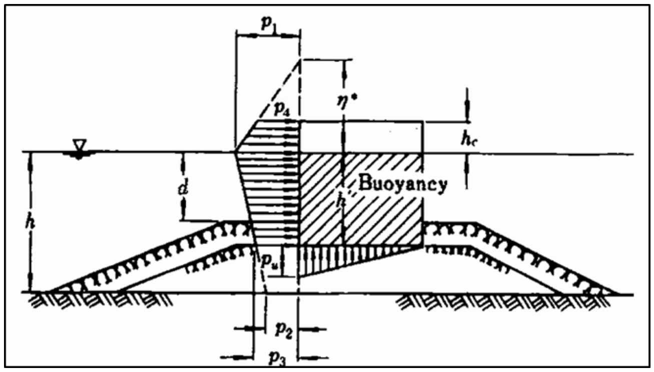

Figure 10. Goda-Takahashi pressures pattern for wave crests

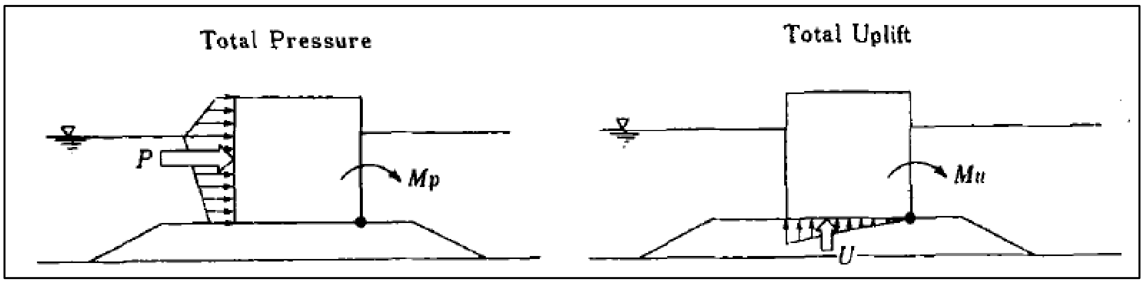

Figure 11. Sketch of forces and torque

3.4.3 Envelope of recorded efforts

The particular tiered geometry of the topping of the caisson is intended to reduce the maximum peak of effort by spreading over three consecutive impact times the action of cyclonic waves breaking upon the structure. The most important effort upon the structure takes place during the first impact. Recorded pressures have been therefore integrated at this moment over the surface of the caisson following the blue pattern shown in the sketch here below.

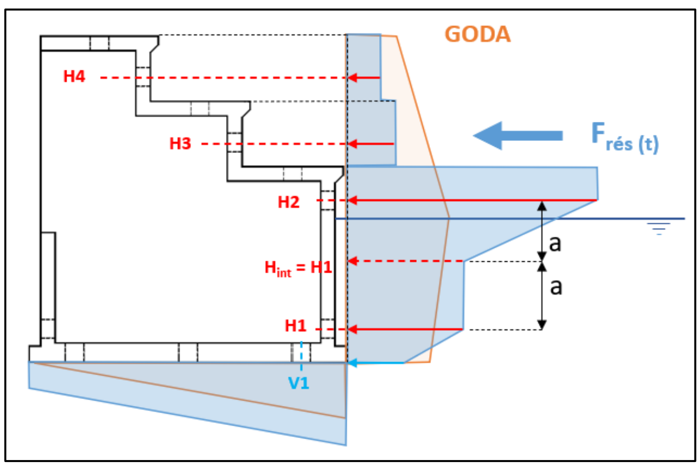

Figure 12. Sketch of pressures diagram used for the integration of pressures recorded

As a crosscheck, we have compared the integrated efforts against the recordings of forces (both scaled taking into account Cuomo 2010 assumptions, see chapter below). The comparison shows that both approaches lead to very similar values, validates the conceptual pattern of pressures shown in the figure above.

Table 3. Table of Comparison between integrated pressures and load cell recordings

Statistical analysis	FX [kN/ml]	Fres [kN/ml]	Δ
Max value	2488	2628	6%
1/250 value	2277	2243	-1%
1/10 value	1613	1675	4%
1/3 value	1265	1367	8%

3.4.4 Amendment of wave impact loads

Given the fact that waves were "impacting" on the caisson (what can be proven by observation and by the chart given in Proverbs) the prototype maximum horizontal efforts scaled by Froude's law have been amended according to (Cuomo et al.; 2010). Cuomo et al. (2010) mentioned that the use of Froude similarity for scaling up wave impact pressures recorded during physical model tests may lead to a significant over-estimation of impact maxima. *"It is well accepted that "pulsating" or "quasi-static" loads can be scaled by simple Froude relationships without any need for adjustment (scale correction). In contrast, it is generally agreed that the magnitudes and durations of "impulsive" wave loads may be strongly influenced by air effects in ways that cannot be scaled by Froude".* According to (Cuomo 2010) impulse can be scaled by Froude, even where pressures or rise times cannot. Under this assumption, the pattern presented in the figure below has been used. The principle has consisted of 1/ amending (reducing) the maximum peak recorded according to Cuomo 2010 and afterwards 2/ enlarging the time of the action in such a way that the area over the curve Force-time is equivalent (same impulsion). This exercise has lead us to rework the time series around the maximum recorded peak of effort in order to match with the corrected load (following Cuomo 2010). Rise-times and total peak times are then derived based on the conservation of pressure impulse.

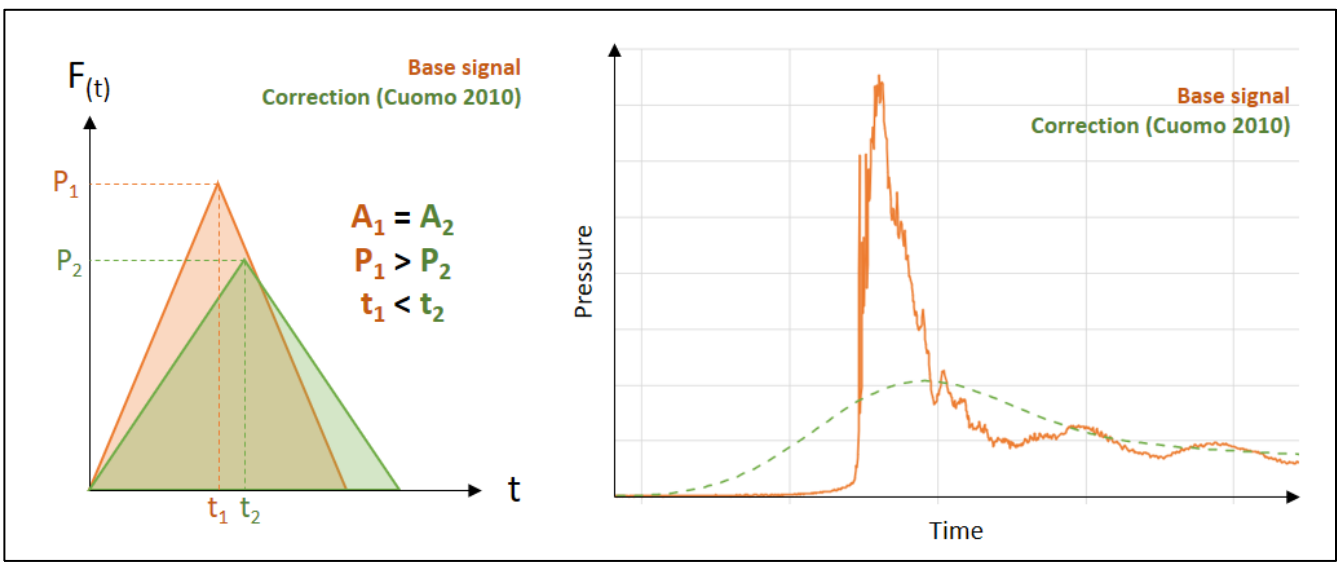

Figure 13. Principle of equivalence of impulse (area under the graph Force-Time) once the peak load has been amended.

The recorded (Froude) and amended values (Coumo 2010) of efforts at prototype scale are given in the tables below:

Table 4. Table of statistical load results according to FROUDE scaling (raw)

Statistical analysis	FX [kN/ml]		FZ [kN/ml]		MY [kN.m/ml]	
	+	-	+	-	+	-
Max value	7215	-361.6	897.4	-7083	125700	-5391
1/250 value	6956	-356	850.4	-6262	118900	-4923
1/10 value	3501	-318.6	611.6	-2598	58110	-4026
1/3 value	2204	-287.6	550.3	-1439	34850	-3712

Table 5. Table of statistical load results flowing to CUOMO 2010 correction factors

Statistical analysis	FX [kN/ml]		FZ [kN/ml]		MY [kN.m/ml]	
	+	-	+	-	+	-
Max value	2488	-361.6	897.4	-1364	37060	-5391
1/250 value	2277	-356	850.4	-1204	33130	-4923
1/10 value	1613	-318.6	611.6	-761.1	23100	-4026
1/3 value	1265	-287.6	550.3	-534.9	17790	-3712

3.4.5 Comparison of amended results (Cuomo et al., 2010) to preliminary calculations based on (Goda ; 2010)

If we compare the results obtained after amending the recordings with Cuomo methodology with the expected values of effort foreseen with Goda 2010 (§2.4.2) we can conclude that the overall order of magnitude of forces and torque are very close to predictions.

Table 6. Table of Comparison between integrated pressures and load cell recordings

Statistical analysis	Horizontal Forces	Torque
Goda-Takahashi *(cf. Table 2)*	$2\,520\ kN/m$	$M_P + M_U = 22\,840 + 11\,765$ $M = 34\,605\ kN.m/m$
2D Model measures (amended by Cuomo 2010)	$2\,488\ kN/m$	$30760\ kN.m/m$

4 NUMERICAL MODELS

4.1 Geotechnical stability

The caisson stability could not be justified by means of a rigid block approach without considering the reaction of the backfill which is proportional to the applied load. We wanted to verify as well if mobilizing a reaction force would not result in unacceptable residual displacements. A 3D non-linear (interfaces and hardening soil model) stress-strain numerical model was then developed with the software FLAC3D. Initially, this numerical model has been used to update the results of the stability analysis by means of a pseudo-static approach. Then, a dynamic analysis involving time-series wave loads was performed to check if the cyclic and final residual (if any) displacements were acceptable. The time series used are those obtained from the treatment detailed on §3.4 (Cuomo 2020 amendment). Considering the pressure recordings, it was possible justify the geotechnical stability of the caisson (sliding, overturning, and bearing capacity).

Displacements were calculated with a dynamic stress-strain non-linear numerical model. Three different tests were performed: 1/ single horizontal pressure series around the maximum peak (duration around 12s prototype), 2/ the cyclic repetition of this same wave, 3/ a 17-minutes (prototype) pressure recordings. The dynamic analysis highlighted a reversible and acceptable caisson crest displacement (order of magnitude few millimetres) without any significant impact in the reaction capacity of the backfill.

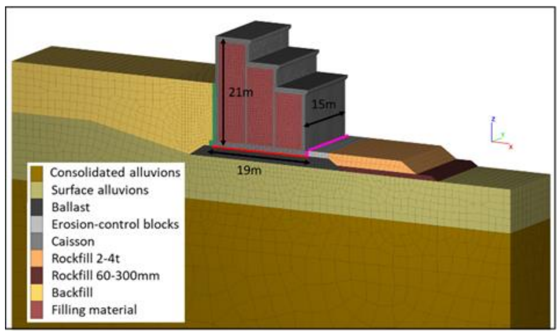

Figure 14. Flac 3D model

Improvements on the scaling of time along the whole length of the recorded series would give more confidence on the findings of the numerical models and allow a more refined analysis. The resonance frequency of the caisson for instance did not affect in our calculations the results of the stability analysis with the used signal. We note however that depending on the type of wave attack, the amendment introduced with *Cuomo 2010* methodology might modify the scaling of time and lead to a lengthening or a shortening of the interval between two consecutive peaks of pressure. This incertitude might distort the detection of a possible amplification of displacements due to resonance effects (when the natural vibration frequency of the structure is similar to the frequency of the applied load, which is the wave pressure in this case).

4.2 Structural design

The structural design verification did not require the development of complex models and calculation approaches. Measured pressures (amended by Cuomo 2010) have been used to feed a Robot 3D model of the caisson to evaluate by static approach the internal resistance of the structure and to pre-define the required steel quantities for reinforcements.

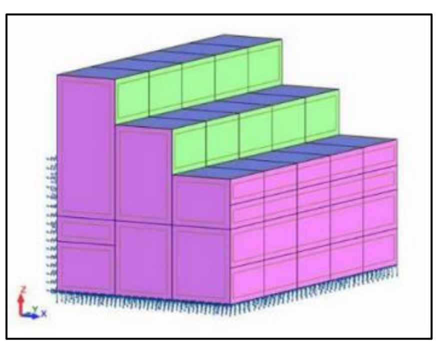

Figure 15. Illustration of the Robot model considered for the reinforcement calculations

5 CONCLUSIONS AND FURTHER RESEARCH

The 2D physical modelling of the suitability of the design of a caisson exposed to (impact) waves needs the scaling of elements and actions of different nature, responding each of them to diverse laws that cannot be reproduced on the same support and premises, and require specific considerations.

Thereby:

- Enhanced permeability shall be considered for the adequate recording of pressures underneath the caisson, otherwise the recordings might be underestimated if using geometrical similarity on foundation rocks.
- Peaks related to the impact of waves cannot be scaled by Froude law since prevailing loads are of elastic nature. An amendment in terms of load and time distribution is necessary. Otherwise the recordings would be overestimated.

The use of Cuomo 2010 amendments on recorded efforts has led to an order of magnitude of model load results that is fully coherent to the preliminary expectations based on Goda-Takahashi formulae. The installation of several pressure gauges has allowed for a better comprehension of the pressure distribution patters at the most critical moment with regard to stability.

With regard to the time series used for the dynamic numerical models, the amendment of maximum impulsive peaks with *Cumo 2010* methodology based on keeping the impulsion equivalent to that of Froude similitude, seems fully applicable for short time periods (around one peak) but needs to be further studied for an extended use over the whole time series.

About overtopping, it is eased on the structure by the presence of a stepped crowing that facilitate the ramp-up of the mass of water. On the other hand, bullnoses have proven to be very efficient for deviating backwards and upwards the overtopping splashes.

The validation of the geotechnical stability of the caisson required the employment of complex numerical calculations. This new approach paves the way to the necessity of definition of acceptable displacements (reversible or irreversible) as a basis of design of major breakwater projects. An enhanced monitoring of existing structures and the creation of an extensive database of their behavior and the relating acceptance is a first stage that may be considered to move forward.

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Cofinancé par le mécanisme pour l'interconnexion
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