# RECENT ITALIAN EXPERIENCE IN THE DESIGN AND CONSTRUCTION OF VERTICAL BREAKWATERS

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## 1. INTRODUCTION

Italy is often considered as a mother country of vertical breakwaters for harbour protection, since they have been widely used all along our coasts long since. An updated location map is given in fig. 1, which also shows the position of the directional wave recording stations of the existing Italian network.

The main reasons for this popularity, despite the usual availability of rock for mounds, are:
- the frequent favourable geotechnical conditions and large water depths (which can make them less expensive than rubble mound breakwaters),
- the small tidal range and not too severe wave conditions of the Mediterranean Sea (which reduce the risks of large breaking wave impact loads on the wall),
- a traditional familiarity since the Roman age with marine concrete structures (which are made with good pozzolan or slag cement).

The most common vertical breakwaters are in fact composed by prefabricated monolithic cellular r.c. caissons, which are typically floated and sunk with seawater ballast upon a rubble mound foundation and then filled with sand and/or concrete.
They are also called "upright" or "composite" breakwaters.

The present improvement of the construction technologies (also promoted by some new large projects of the offshore oil industry) ensures a high durability and low maintenance of these structures and allow a rapid installation on site with reduced risks of damage in the construction phase.

The speed and safety of construction and an increased control of the structural performance are in fact assuming a greater importance in the breakwater design choices. The use of floating construction equipment can be very cost effective in remote locations, such as islands.

Moreover, the present environmental constraints related to the use of rock quarries, the air/water/acoustic pollution and heavy traffic due to rock transport inland and dumping on site and a smaller visual impact compared to a wider rubble mound structure are now favouring the caisson breakwater solution even for shallow-water applications (eg. coastal marinas).

Another advantage of the caisson technique in sensitive coastal environments is the flexibility, due to the potential removability of the structure by pumping out the cell fill.

A "revival" of the vertical breakwater concept is now going on also outside the traditional countries (Italy, Spain,Japan) after some recent catastrophic failures of large rubble mound breakwaters (OUMERACI et al.,1991).

New research work (including large-scale physical models and advanced numerical models) is being addressed to the dynamic behaviour of vertical structures, particularly under breaking wave impact loading, as within the EC-funded MAST G6-S project (1990-92), which already produced many new contributions (see references). Also a new PIANC working group (n.28) is just about to start.

The following paragraphs give useful information on the most recent caisson breakwaters designed and constructed in Italy, which include particular solutions for the configuration of the caisson walls and superstructure in order to reduce the main "drawbacks" of vertical structures, such as wave reflection and toe scour, wave forces and wave overtopping.

The hydraulic behaviour of these structures has been optimized with the aid of laboratory model tests. Further insight on the performance of vertical breakwaters will also be gained from the results of prototype measurements which are starting in late 1992 at two instrumented caissons at Porto Torres industrial harbour and are also planned for similar structures in other Italian locations.

Before describing the most interesting new developments, it is worth to recall the historical evolution of vertical type breakwaters in Italy, which has been also influenced by the occurrence of a number of major failures suffered in the past. Most of these failures are well known among breakwater engineers and they are typically more catastrophic than those of rubble mounds due to the "fragile" stability behaviour of vertical structures.

## 2. HISTORICAL ITALIAN VERTICAL BREAKWATERS AND LESSONS FROM FAILURES

The main vertical breakwaters which were constructed before the last World War (some of them even in the past century) are located in the harbours of Genoa, Naples, Palermo, Catania, Trapani and Bari (fig. 2-3-4). In the figures and in the text, elevations are always referred to the Mean Sea Level (L.M.M.) as usual in Italy.

The type of the vertical structures was very variable mainly following the evolution of more powerful construction equipment. The first vertical walls were made with overlapping parallelepiped concrete blocks weighting 50 to 150 t (concrete masonry blockwork) (fig. 4A).

Then hollow cellular concrete-filled blocks (up to 400 t) were used to cover the whole structure width (fig. 4.B-C). However these cellular blocks suffered from the bad quality of the in-situ mass concrete filling and its imperfect bond with the frame elements. The next step was represented by the use of full-width monolithic overlapped cyclopean blocks weighting up to 1000 t. The vertical holes needed for handling them with the larger cranes were then filled in-situ with concrete and rails or steel bars to ensure the compactness of the column (fig. 3). The final most efficient solution is represented by the reinforced concrete cellular caissons, the first application of which dates back to 1925-27 in minor works at the harbour of Naples, Genoa and Capri, although their use became generalized on industrial basis after the last war.

Application of the caisson technology has been successfully carried out for more than 20 breakwaters in Italy. The design of these composite structures was based on empirical concepts and the evolution of the structural shapes was a consequence of the analysis of the behaviour in time.

In the harbour of **Catania** the main breakwater failed on 26 March 1933 (soon after its construction) during a severe storm the characteristics of which were not recorded: MINIKIN (1950) stated that the waves were 7.6 m high and about 150 m long. The 320 t concrete blocks (12x4x3.25 m), simply placed above a rubble base at -12,5 m, slided over one another in successive courses due to the unforeseen forces induced by waves breaking on the wall. The breakwater of Catania was repaired by transforming it to a rubble mound structure which has successfully worked so far (Fig. 2).

This failure, together with the collapse of the Mustapha breakwater at Algiers (February 1934), deeply worried the experts in harbour engineering: it can be regarded as the first shock in the history of breakwater construction and partly explains why rubble mound structures are still favoured. The consequences of these failures are apparent in the conclusions of the next PIANC Conference of Bruxelles in 1935, which gave precise guidelines on the limits to be respected in order to guarantee the occurrence of a standing wave in front of the structure.

Even if just referred to a generic maximum wave height (the knowledge of wave statistics was then practically non-existent) these guidelines are still valid: in particular if H is the design wave height, d is the water depth at the toe of the wall and d' is the depth at the toe of the rubble mound foundation then:

$$1.5\,H \le d\,;\quad 2.0\,H \le d' \qquad (1)$$

in order to ensure the formation of a stationary wave.

Later on, a few researchers suggested to increase the threshold values for the ratios d/H and d'/H. LARRAS (1937) proposed:

$$2.0\text{–}2.5\,H \le d\,;\quad 2.5\text{–}3.0\,H \le d' \qquad (2)$$

These ratios were also recommended at the PIANC Conference of Rome in 1953. Later on NAGAI (1973) carried out extensive model tests to study the influence of various factors, including berm width at the toe of the wall and the slope and roughness of the seabed. He obtained a complete series of conditions guaranteeing the total reflection of the waves and then suggested:

$$0.75\,d' < d\,;\quad 1.8\,H \le d' \quad (\text{with } H = H_{1/10}) \qquad (3)$$

Despite the warning from these failures and the application of the above recommendations, no preventive rehabilitation works were carried out to upgrade other old vertical breakwaters designed before the 1930's in a similar way as the one at Catania.

Therefore after the war other disasters occurred to the breakwaters of Genoa (1955), Ventotene (1966), Palermo (1973), Bari (1974) and Naples (1987). In all cases the collapse was due to high wave impact loading, only partly induced by the limited toe depth, which exceeded the often underestimated design conditions.

The failure at **Genoa** is described by D'ARRIGO (1955). With reference to fig. 3 it may be worth noting that only the uppermost block and the mass-concrete superstructure slided inwards for a length of 450m, while the lower blocks remained in place almost undisturbed. Artificial 40 t and 60 t armour parallelepiped blocks were later dumped against the renovated seaward face of the wall.

The case of the collapse of the depth-limited breakwater in the island of **Ventotene** was a particular one, since this was in fact small a Japanese-type composite structure (Fig.3) with clear deficiencies in the design. During a storm with an estimated peak Hs just exceeding 5.0 m the low rock protection (up to +1.0 m) was eroded to a depth of -3.5 m and the caissons slided inwards due the high direct breaking wave loads, which had not been considered in the design, since only the absorbing capacity of the weak armour protection was relied upon. Shallow water effects had also been neglected. For the rehabilitation of the composite breakwater a 15 t tetrapod protection was then used.

A more catastrophic damage occurred on 25 October 1973 to the old offshore (36 m depth) breakwater of **Palermo** harbour, built between 1922 and 1938. This failure has been reported by MALLANDRINO (1974). The whole 700 m long breakwater (except the roundhead!) disappeared underwater. The old and new cross section is shown in Fig.3. The actual size was slightly reduced respect to the original design due to economic constraints. The vertical structure was constituted by four cyclopean concrete blocks (9.0x2.6x2.5 m weighting 130 t) founded on a rock mound at -10 m and with two apron slabs (5.0x2.5x2.0 m) placed at the toe of the outer face for scour protection, reducing the local water depth to only 6 m. The two upper block layers and the concrete superstructure (reaching +9.0 m) slided towards the harbour basin. The peak significant wave height at the breakwater was estimated by simple hindcasting and refraction methods as 6.1 m. Subsequent calculations showed that by using the traditional Sainflou pressure diagram for standing wave the vertical wall would have been just marginally stable. The application of the well known formulae by Minikin and Nagai for breaking wave conditions, even neglecting the presence of the toe blocks, demonstrated that the stability criteria against sliding and overturning were not fulfilled, particularly at the interface between the two upper blocks at -2.2 m depth . The breakwater was later transformed into a rubble mound armoured with 40 t cubes.

Severe damages were also observed at the old vertical breakwater of **Bari** harbour made with 10 m wide 400 t concrete blocks sitting on a narrow rubble berm at -10.5 m. Again the design wave conditions had been underestimated due to the lack of knowledge on wave hindcasting before the war and the occurrence of waves breaking in front of the structure was neglected. The rehabilitation works included an armour protection with 30 t tetrapods.

Probably the only European breakwater failure case involving caissons instead of a block-wall occurred at the offshore breakwater Duca D'Aosta in **Naples** harbour during a storm on 11 January 1987, which reached a hindcasted peak $H_s$ of 5.1 m ($H_{max}$=9.3 m) in front of the structure at water depths of 17-20 m. A detailed analysis of this failure is described by FRANCO and PASSONI (1992). The 2.5 km long breakwater is composed of different sections built over the years, offering a good picture of the evolution of maritime constructions in the last century (Fig.4). While the old cellular block wall was mainly eroded due to concrete degradation under wave impact loading, the worst damage occurred to a few small caissons (section D) built in the 50's (one of the first applications of this technique), which slided landward by 4 to 9 m, tilting down the rear slope of the rubble foundation, probably overloaded (fig.5). In fact their plan size was only 11.5x6.7 m with a toe depth of just 10 m. The adjacent new caissons (type E) just installed in 1981, having a width of 14 m and toe depth gradually increasing to -12-13 m, only slided inwards by 1.2 m and 0.5 m respectively. The application of the well known Goda's wave pressure formula gives a good description of both the incipient sliding motion and of the total caisson displacements induced by partially breaking waves, probably enhanced by 3-D effects. The tilted caissons have been demolished and substituted by larger ones, and additional concrete was casted on to the seaward face of the displaced type E caissons to achieve a uniform alignment.

As far as more modern vertical breakwaters are concerned, no major collapse has been experienced so far. The main problems typically observed in caisson breakwaters are the differential settlements due to poor foundation soils and/or to scour effects at the toe. Wave overtopping is also a usual source of damage (more functional than structural).

Localized damage to the superstructure and to the foundation in fact characterized the two most recent failure cases occurred in late 1991 at the new caisson breakwaters of Gela and Bagnara (see sections in fig. 6).

The offshore breakwater of **Gela** built in the early 60's in just 12 m depth is one of the first applications of the concave parapet wall on the caisson crown. It was hit by a severe storm on 24 Nov 91 with an estimated peak $H_s$ around 6 m, resulting in local breachings of the crownwall and damages to the pipelines running on the superstructure caused by heavy wave overtopping. The caissons remained globally stable.

At **Bagnara** (built in 1985) the damages observed after the storm of 20 Dec 91 were also mainly concentrated on the rear side and at the toe, where a tetrapod protection was eroded.

The main lessons from the above described failures resulted in the increase of both dimensions of the vertical structure and its monolithic solidarity with independent portions of the superstructure. The reduction of wave forces and overtopping has also been pursued in the new designs by means of various structural changes to the front geometry (cylindrical, perforated), to the crownwall (sloped and curved parapets), and to the foundation (wider front rubble berm, larger flat perforated toe apron slabs or "guardian blocks"), as illustrated in the following paragraphs.

A much greater confidence in the design of vertical breakwaters has been achieved by the systematic use of laboratory model tests. Generally the new advances in maritime hydraulics have led to an increase of the design wave height and to a deepening of the caisson toe to avoid seabed-induced wave breaking.

However,in most even recent deepwater failure cases some "partial breaking" has been observed in front of the wall, which can be attributed to the 3-D irregularity of waves, to the phase interference of incident and reflected waves as well as to the wind effect on the steeper highest waves. The consequent impact load results in an increase of the horizontal force derived from the Sainflou theory, although not so large and localized.

Wave concentration and damage often occurs at singular weak points of the breakwater like heads, bounds and junctions between two different types of structure.

Wave overtopping and toe scour/liquefaction are also typical causes of failure modes such as landward or seaward tilting of the caisson.

Therefore, the total wave breaking load history and a full probabilistic dynamic approach (instead of the usual static calculations) are needed to evaluate the wave-caisson-foundation interaction and perform a proper design in 3-D wave conditions.

## 3. RECENT DEVELOPMENTS OF VERTICAL BREAKWATERS IN ITALY

Following a previous review by ROMITI, NOLI and FRANCO (1985), it is believed worthwhile to produce an updated "inventory" of the most recent applications of the vertical-breakwater concept in Italy.

A unique example of pure vertical breakwater in soft soils (silt-clay) and in a waterdepth of 11 m is represented by the concrete screen driven to -14 m to MSL, supported by steel piles and capped by a solid r.c. slab for the protection of the "island-harbour" at Manfredonia in the southern Adriatic Sea (Fig.7).

The large majority of modern vertical breakwater structures are in fact cellular r.c. caissons. Figs.8.a-b shows the cross sections of nearly all the latest applications and brief technical notes are given below, while the new perforated caisson breakwater at Porto Torres is described in more detail in the following paragraph.

1) PALERMO, main breakwater (1980)
The water depth is around 35 m and the bottom of the 16 m wide caissons sits at -16.8 m to MSL.

2) NAPOLI, "Martello mole" (1982)
Small antireflective caissons (13 m wide) were built for a new breakwater in a partly sheltered area of Naples harbour. The caissons with only two cells are founded at a depth of 12 m on a rubble footing upon a seabed at -16 m. Both the seaward wall and the sloping parapet are perforated.

3) SORRENTO, ferry and craft harbour main breakwater (1985)
The water depth can reach -23 m in a relatively sheltered location. Depth and width of the caisson are both 14.0 m. Both the outer two chambers and the sloping concrete crownwall have circular holes to dissipate part of the wave energy by friction and turbulence.

4) GENOVA-VOLTRI, main breakwater (1986)
The traditional vertical face caissons (18.5 x 30.1 $m^2$) sit at -20 m on a rubble foundation reaching a depth around -30 m on sady-silty soils, partially upgraded with fill material. The design wave height is 8.0 m. Settlements between 1.0 m and 1.5 m were measured during construction.

5) VADO LIGURE, extension of main breakwater (1988)
The cross section is very similar to the one at neighboring Voltri harbour (probably the same equipment was used). The 19 m wide caissons are based at -19.5 m in depths of 30 m.

6) NAPOLI, West Breakwater "Duca degli Abruzzi" extension (1988)
In this case the traditional cellular square caissons (16.5 x 22 m) exhibit a semicylindrical seaward front and a sloping parapet wall like the one at Civitavecchia. Full model tests were conducted at DH. The sandy seabed ranges between -25 m and -40 m and the caisson toe sits at -19.50 m. The design wave height is 6.0 m with a period of 9.0 s. Seismic loading was also considered. The maximum observed settlement was 0.6 m. The two perforated absorbing chambers on the harbour side are due to reduce the diffracted wave disturbance.

7) BRINDISI, Punta Riso breakwater (completed 1989)
A detailed description of the design and construction features (including a risk analysis) is given in FRANCO et al.,1986 and CHIUMARULO et al.,1990. The caisson-type structure develops for about 1 km on water depths between -24 m and -32 m to MSL on sandy-silty soils, with the crown of the rubble foundation being at -18 m. The horizontal section is 18 x 21.4 m. A design wave with a height of 8.0 m and length of 139 m was assumed. Both 2-D and 3-D model tests were performed at Delft Hydraulics. Settlements of 0.5 m within 6 months after caisson installation were recorded.

8) CIVITAVECCHIA, main breakwater extension (1990)
The extension of the long breakwater at Civitavecchia harbour is located in water depths of 25-30 m on sandy soils and the design waveheight was 8.0 m. The caisson size is 20 x 21.5 m with a height of 19 m. The crownwall has a sloping-face parapet set back 8 m from the seaward wall. This parapet type reduces horizontal forces and overturning moments compared to a semi-vertical one with similar set-back, according to the results from model tests at DH. It was also investigated the performance of alternative cylindrical caissons, which underwent maximum impact forces 1.5 times less than the equivalent square caisson. However the selection of caisson type was left open in the tender and the contractors eventually preferred the traditional structure with rectangular section.

Moreover a few more vertical breakwaters have been built in the last fifteen years, but they have been placed in sheltered bays or were soon included into or protected by a rubble mound structure (Taranto, Sibari, Castellammare di Stabia, Ravenna) (fig. 9).

Application of the caisson technology can even be found in the design of breakwaters for marinas in shallow waters as shown in figs. 10,11 (FRANCO and NOLI,1989, FRANCO and MARCONI, 1992).

## 4. THE NEW CAISSON BREAKWATER OF PORTO TORRES INDUSTRIAL HARBOUR

It is worth giving a more detailed description of this new original vertical breakwater, since it has been and will be subjected to interesting laboratory and prototype investigations.

The existing offshore caisson breakwater (completed in 1976) has been connected into land 1992 via a 2 km long breakwater to provide a better protection from northwesterly waves and create a new terminal for 150,000 DWT ore-carriers. It is made with a rubble mound section on water depths up to 17 m, with perforated caissons for depths between 17 m and 19 m and plain-walled caissons (similar to the existing ones) for depths between 19 m and 21 m, thereby allowing a gradual transition between differently reflecting structures. Due to the shelter of the Asinara island the design significant wave height is 4.7 m with a peak period of 11.0 s.

All caissons have plan dimensions of 20.5x13.9 m (with two lateral bottom expansions of 1.5 m). Some are founded at a level of -14.0 m, some at -15.0 m. The crownwall reaches an elevation of + 8.0 m. The toe is protected by one row of apron slabs (3.2x1.0x6.0 m) with pressure relieving holes of 300 mm diameter . The rubble footing is armoured with 1-3 t rock.

The perforated caissons are subdivided in 4 rows of 6 cells, the outer three rows being connected with the sea through three perforated walls, forming a sequence of wave absorbing chambers with a total length of about 10 m. The seaward outer wall of each caisson presents 48 holes on 4 rows, the first internal wall has 36 holes on 3 rows and the second internal wall has just 12 along one line. The two external r.c. walls have a thickness of 0.4 m, while the two internal perforated walls are 0.25 m thick and the only plain internal wall 0.18 m. The novel feature is that these holes are infact rectangular windows of 1.9x0.9 m. In the original design the external wall had 84 traditional circular holes of 1.0 m diameter along 7 rows to reach the same usual optimum 30% porosity. This change was basically due to construction reasons allowing a reduction of time for casting and placing reinforcement. However the longer wet perimeter of the squared holes seems to induce a larger wave energy dissipation by friction and turbulence, particularly under oblique attack.

In order to protect the perimeter of the window from the tangential action and cavitation phenomena due to the ingoing and outgoing flow, the holes have been framed by properly designed fiber-reinforced concrete elements. The addition of the polypropylene fibers (with a content of 4 kg/m$^3$) can in fact increase the material impermeability and resistance to chemical aggression, impact forces, cracking and abrasion. A number of laboratory tests on small samples and full-scale elements was carried out to verify the ultimate characteristics of various composites under compression, bending and impact loads. The efficiency of various fiber contents has been also investigated by means of advanced numerical models (FRANCO et al. 1989,1992).

As far as construction is concerned it is worth to say that two different floating platforms were simultaneously used for a quicker prefabrication of caissons. The traditional platform with rigidly connected bottom slabs and lateral walls was used in limited depths requiring a temporary launching of the uncompleted caisson and casting within sliding moulds lifted by a pontoon crane. The modern platform with a catamaran shape could support the whole formwork allowing a single-phase rapid construction. Shifted reinforcement positions were assumed in the internal and external walls to save placement time. A special machine supported by a jack-up platform was used to level the rubble foundation underwater with remote video control. An asymmetric seawater ballast was required to compensate a small eccentricity of the centre of gravity of the perforated caisson during the towing phase, the most critical for stability.

## 5. MODEL TESTS AND MODERN STRUCTURAL FEATURES

From the previous illustration of the modern caisson breakwaters it appears that the main geometrical parameters (caisson width and toe depth) usually have very similar values. The geotechnical characteristics of the seabed can vary from very hard compact soils to sands even mixed with silt. Especially in the latter case a careful design of the rubble foundation was necessary (often including the some seabed dredging) and enough time was left before the placement of the caisson to allow settlements to take place.

Most of the caissons are prismatic (square) with plain vertical walls (repetitive shapes are a consequence of the re-use of the same prefabrication platform), but modern solutions include variable geometries of the front face and parapet wall, and some have perforated absorbing chambers. The introduction of new structural features to reduce wave forces and overtopping has been supported by many hydraulic model tests conducted in recent years.

The basic shapes of the solid wall are schematically represented in fig. 12. Cases b,c,d, have a better sliding resistance due to a favourable downward component of the wave force. In a semi-circular caisson (type d) the resultant force even acts towards the centre and makes no rotational moment, resulting in a quasi-uniform distribution of bottom reactions, which is advantageous on soft grounds: the scheme was in fact proposed in Japan , also for its soft appearance (TANIMOTO and GODA,1991).

The sloping face (type b) is actually feasible only for the superstructure (b1) as shown in fig.10 and applied at Hanstholm caisson breakwater. The obvious advantage of the sloped wall in reducing horizontal wave forces (by 30-50%) is particularly effective when tidal variations are small, but it is balanced by a worse overtopping performance if compared to the scheme a1 with concave wall crest having opposite characteristics.

An efficient combination of the two concepts is represented by a sloping face parapet wall set back a few metres from the caisson vertical wall. The overall stability is thus increased due to a reduction of the maximum horizontal and vertical force caused by the delay in the wave action on the two surfaces and due to the prevention of setting up impulsive breaking wave pressures caused by the face discontinuity (fig. 13) (GONZALES et al.,1992). The Spanish researchers found that this parapet shape produces a greater stress reduction if compared with a vertical wall protected by an offshore submerged barrier. This alternative solution seems to be justified only when wave overtopping is of greater concern.

As mentioned earlier the concept of a sloping curved set-back parapet was used for the Civitavecchia caisson breakwater and the shape of the curved superstructure was optimized with model tests at DH . Some results are shown in fig.15 : the more vertical forward sloping parapet II showed larger horizontal forces (about 8%) and overturning moments (above 19%) than the preferred parapet I.

Vertical forces should also be considered when the wall crest is jutting seaward. The curved solution gives the best force reduction as shown by JUHL, 1992 (fig.14).

Cylindrical and perforated fronts are also hydraulically effective. The perforated caissons are typically used for harbour quaywalls, but they have now been successfully used also for breakwaters to reduce wave reflection affecting coastal navigation and toe scour and to reduce wave overtopping. These structures also seem to be less sensitive to impact loads.

The reduction of the wave overtopping due to the perforated wall and to adequate shaping of the crownwall can be particularly beneficial for marina breakwaters where yachts are moored against the rear side,as unfortunately often occurs in Italy. It may also improve the aesthetical impact by lowering the parapet crest.

In particular the hydraulic performance of the perforated caissons to be built for the new breakwater of **Ponza** harbour in water depths of 14 m and 9 m has been model tested at DHI and partially reported by JUHL (1992). The horizontal and vertical forces and the overturning moment were measured by a dynamometer which the caisson were suspended to. Tests with different wave steepness (important factor) and water levels showed a limit of stability at Hs = 2.8-3.5 m for the small caisson and at 4.1-4.6 m for the deeper one. The low "aesthetical" sloping parapet wall was raised by 0.5 m to reduce wave overtopping to an acceptable level (see final design sections in fig.11). Compared to a traditional caisson the perforated one showed smaller horizontal forces, overtopping discharges and reflection coefficients (especially for the shorter design waves), but a higher total vertical force due to the uplift action on the slab induced by the waves penetrating through the holes.

It is also worth reporting here on the new model investigations on wave overtopping of caisson breakwaters carried out in the random wave flume of ENEL-CRIS hydraulic laboratory in Milan (FRANCO et al.1992).

Various caisson configurations and crownwall geometries, such as perforated and sloping walls or composite structures were tested at a scale of 1:20. The most innovative features of this research are the measurement of individual wave overtopping volumes collected in a tray suspended to a load cell and the analysis of their effects on target models of persons and vehicles placed behind the crownwall, therefore assessing new safety design criteria based on the statistical analysis of failures. The modelling of the targets has been calibrated by comparison of the full scale behaviour of both a "volunteer" and a ballasted plastic dummy subjected to variable water jet volumes, quickly dropped from a height of 5 m without notice.

Despite the complex non-linearities and scatter, interesting results were obtained: the maximum overtopping volume, which is a better indicator of damages on the rear side, is not uniquely correlated to the mean discharge; larger holes and perforated chambers can halve the reflection coefficient and reduce overtopping by two orders of magnitude compared to a plain wall, if combined with a suitable concave shaping of the vertical parapet wall (fig.16); the wave period plays an important role; a rock protection in front of the caisson produces a larger overtopping, unless it emerges far out of the sea level; a pure vertical face typically gives a smaller percentage of overtopping waves, but with larger max volumes and a greater probability of damage with the same overtopping compared to armoured or sloping superstructures.

Very interesting findings have recently been obtained by BOCCOTTI (1992) with an original small-scale experiment in the real sea. In the perfect "natural laboratory" of the Messina Straits he installed a small vertical metal wall (12x2.1m in a waterdepth of 1.5 m) supported by a steel truss (fig. 17). Thirty pressure transducers were placed along the wall and in front of it up to a distance of 7.5 m. The small breakwater can be considered as a 1:30 reproduction of the Genoa caisson breakwater weighting 570 t/m, but it would be remarkably more hazardous with a waterdepth of 45 m against 18 m and a crest elevation of 18 m against 6 m to M.S.L. The model withstood waves with $H_s$ of 0.42 m and $T_p$ of 2.56 s corresponding to exceptional prototype values of 12.5 m and 14 s. This amazing performance is mainly attributed to three design modifications of the traditional vertical structures:

a) a larger longitudinal extension of the wall, which would simulate a caisson with a length of 360 m instead of 20 m. A long caisson receives a reduced force per unit length under oblique wave attack and a smaller total force from the highest waves having a limited front extension;

b) a discontinuous bottom support along the two external faces, which nearly eliminates the uplift pressures typically acting on the flat bottom slab of caissons;

c) hinged heels at the structure edge, which penetrate into the rubble foundation increasing the friction against sliding. The heel is effective if the foundation is vertically loaded and a hinge support on the rear face would also induce a more uniform load transmission to the foundation, avoiding soil breakage and improving the overall sliding resistance.

Of course for real caissons practical construction and installation problems need to be solved. However the longest caisson just placed for a breakwater in Japan in 1992 is 100 m long and it was towed to the site for a distance of 370 km.

Further solutions for improving the caisson stability can be experimented, such as vertical bonding joints between adjacent caisson units (to increase the longitudinal solidarity) or a higher rubble cover on the rear side (to improve the sliding and geotechnical resistance).

A overview of modern and future applications is illustated by FRANCO (1992).

## 6. PROTOTYPE MEASUREMENTS

It has always been felt that a substantial improvement of the knowledge on the complex wave-caisson-foundation interaction could be gained by means of full-scale measurements on real breakwaters, although the expected high costs for long regular recording periods coupled with the required sophisticated instrumentation have discouraged researchers so far.

Early prototype measurements were taken even prior to World War II at the breakwaters of Genoa and Naples, which were fitted with pressure transducers. Then, in the mid 70's, a measurement station was set up in front of one caisson of the Genoa breakwater at 25 m water depth. The experimental facility is shown in fig. 18. It consists of a laboratory room 3.2m x 3.0m x 22.0m with 10 windows facing seawards. The windows are closed by bronze flanges. A tube crosses each flange and connects the sea and the laboratory room, allowing measurements of the pressure at various depths in nearly ideal laboratory conditions.

Investigations have also been carried out on the effective values of the important uplift forces acting undernearth the caissons. Measurements showed that the traditional triangular distribution of the maximum uplift pressures tends to become close to uniform when the harbour side of the rubble mound is obstructed by the deposition of fine sediments (MARCHI,1977).

Unfortunately a number of practical and financial problems inhibited a regular and efficient activity of the station. Later on it was renovated by BOCCOTTI (1984) who introduced new water-mercury piezometers tranferring data to tranducers in the upper room digitized at a rate of 0.5 Hz. Records were taken for a few months and were basically used as an indirect measurement of wave heights in front of the structure. The analysis of the random wave records were then used by Boccotti to validate his theory of quasi-determinism of the highest sea waves. The zero upcrossing wave pressure measurements were also used to verify the reliability of the $3_{rd}$ order standing wave theory for regular waves. The computations were carried out assuming two different water depths: once the 17 m above the apron slab, once the 25 m at the rubble toe. The best agreement, particularly for the lowest measuring position, was found when referring to the smaller depth at the toe of the caisson.

No further recordings and analyses were reported from the Genoa measurement station, despite the installation of an offshore waverider buoy.

In recent years various detailed monitoring programs have been proposed for the new caisson breakwaters at Voltri, Vado Ligure, Brindisi, Civitavecchia and Porto Torres. So far only in the latter harbour the West Breakwater is being eventually instrumented and a brief description is given here below.

Two caissons, one with plain wall, the other one with perforated walls and absorbing chambers, at a distance of about 80 m have been fully equipped with pressure cells, electric piezometers and accelerometers as shown in figs. 19. Both caissons are also equipped with an inverted echosounder for measuring surface elevations in front of the external wall. A directional wave recorder is already functioning at a depth of 20 m, 1 km off the breakwater. The full instrumentation set (48 sensors) should be operational in October 1992.

Measurements will be taken during 10 minutes every hour, storing only the maximum record each day for a planned period of 2 years. The data sampling frequency will be 20 Hz for the sensors along the vertical wall and 2 hz for those along the bottom slab, the accelerometers and the wave recorder. Most sensors even inside the inner perforated walls can be easily substituted since they are installed in an extractable flanged tube. All the data are teletransmitted ashore through a small station shed placed on the caisson (nearly 300.000 data points for each record).

## 7. CONCLUDING REMARKS

It can be concluded that vertical breakwaters are still very popular structures in Italy, despite the dramatic failures occurred to a few old breakwaters in the last 60 years.

The main reasons for this "success" can be attributed to:

- the progress in construction technology of prefabricated monolithic concrete caissons which ensures reduced costs, shorter installation times and better quality and durability of the structure (with low maintenance);
- a favourable environmental impact in relation to spatial and visual obstruction, potential removability of infilled caissons and smaller air/water/acoustic pollution during construction compared to a rubble mound ;
- the greater confidence in the design which takes advantage of the recent advances of knowledge in maritime hydraulics and of the extensive use of laboratory model testing;
- the introduction of new alternative caisson geometries (e.g. cylindrical fronts, perforated absorbing chambers, sloping parapet walls) which can reduce the wave forces, wave reflection, overtopping discharge and toe scour effects.

The safety against wave overtopping in particular is gaining importance for the increased recreational use of breakwaters which should be easily accessible to the public (fishermen..).

Further improvement of knowledge of the complex wave-caisson-foundation interaction are being achieved from new research activity (particularly within the present European MAST G6-S project) and increased practical engineering experience. Useful information is also expected from new prototype measurements, just about to start in two instrumented caissons of the West Breakwater at Porto Torres industrial harbour.

A better insight of the effective dynamic response of vertical structures under high impact forces due to breaking waves will undoubtely promote a wider application of the caisson technology even in shallow waters and will lead to safer and more economic breakwaters.

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FIG. 1: Location map of vertical breakwaters in Italy (1992)

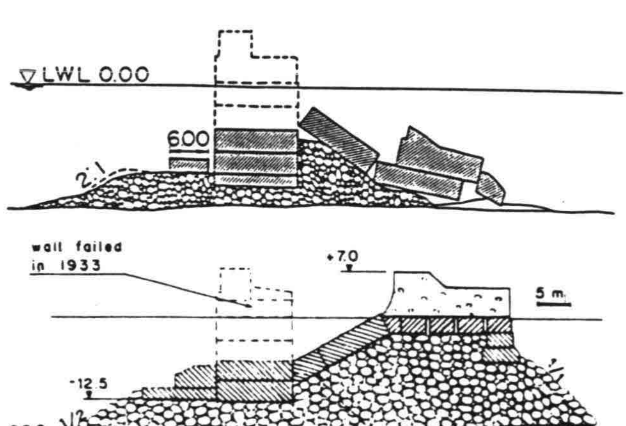



FIG. 2: Failure and rehabilitation of Catania breakwater

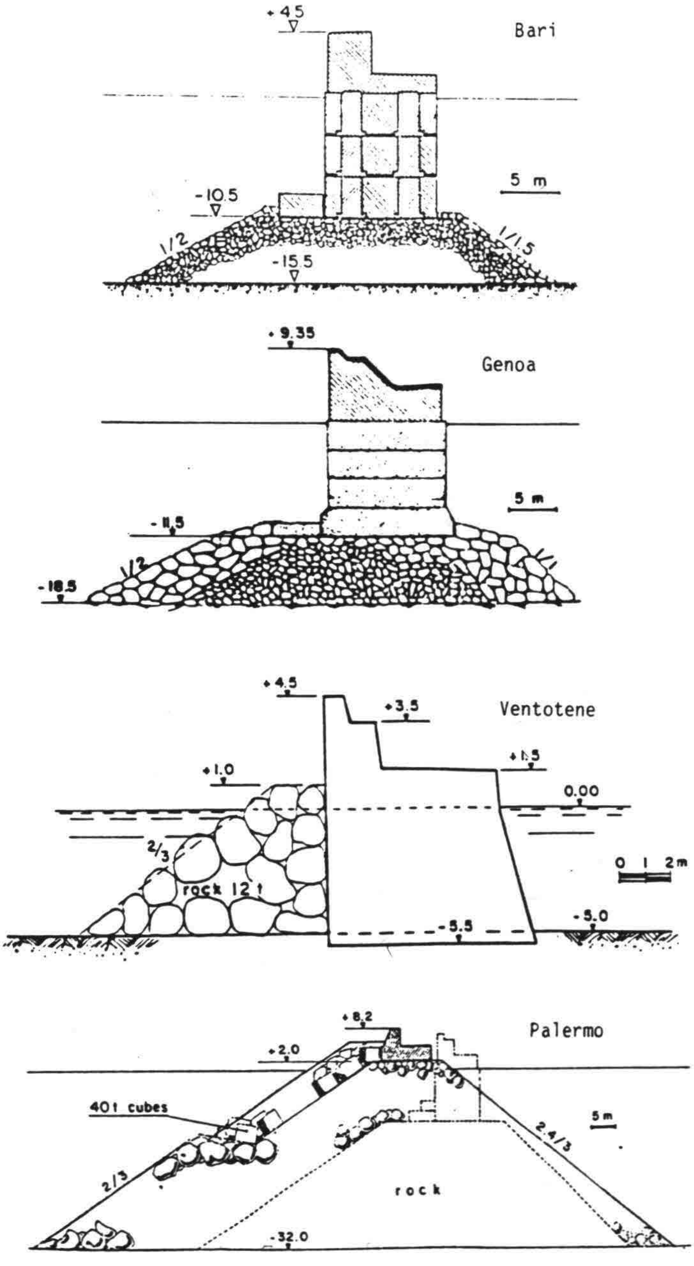



FIG. 3: Section of failed Italian vertical breakwaters

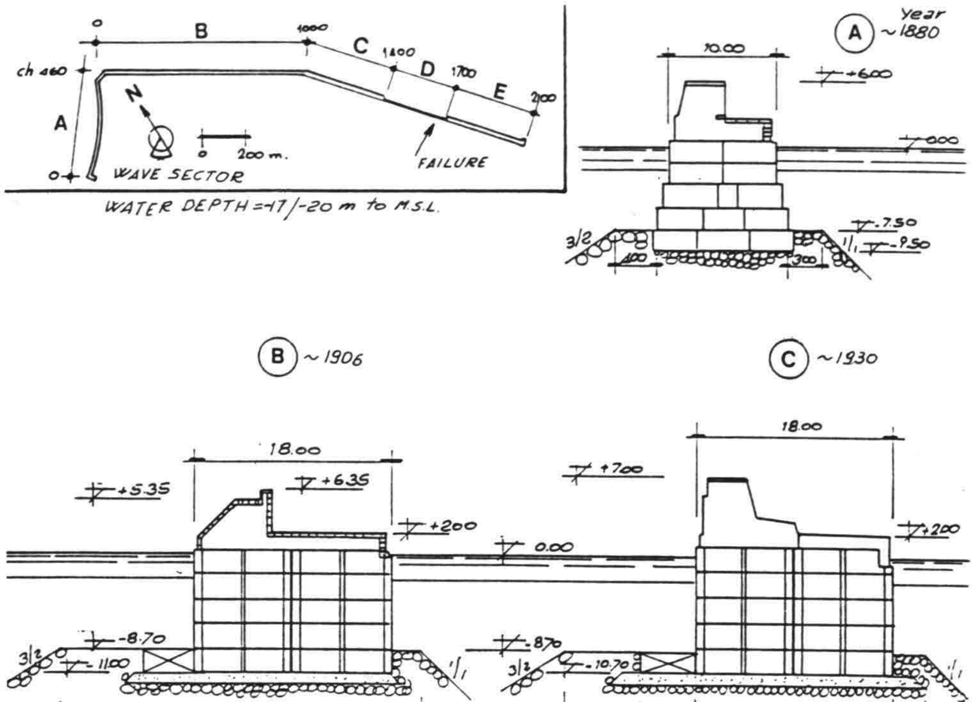



FIG. 4: Plan and sections of Duca d'Aosta breakwater in Naples

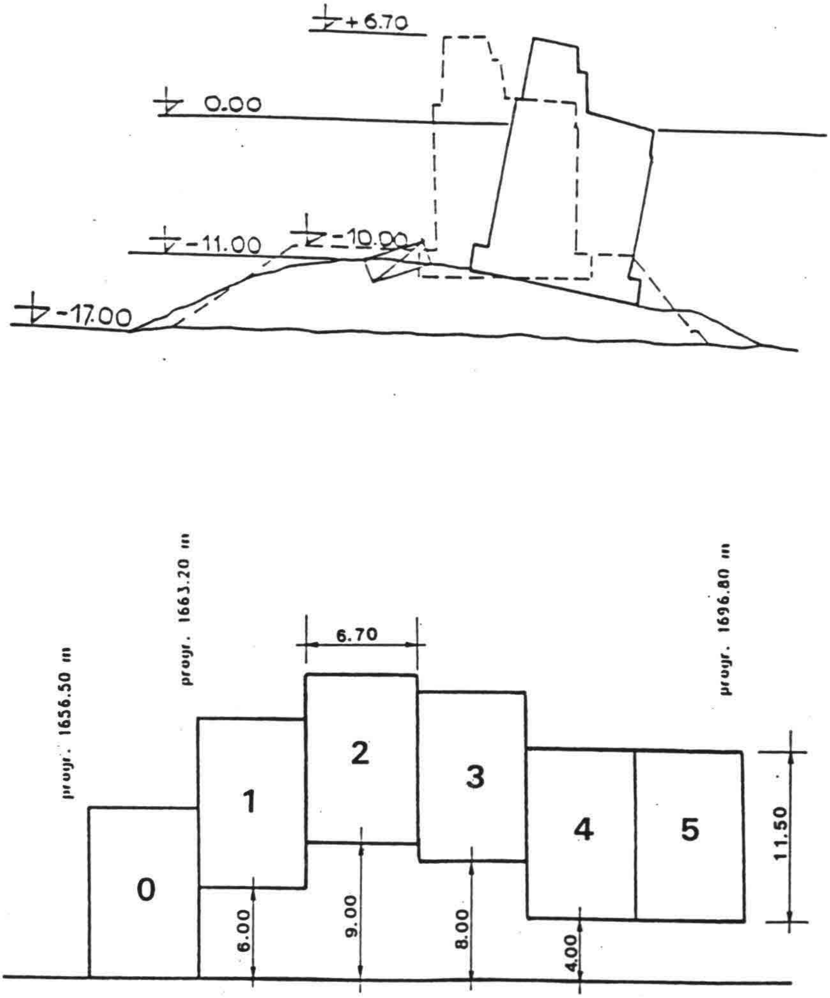



FIG. 5: Plan and section of the type D caisson displaced during the storm of Jan. 1987 Naples harbour

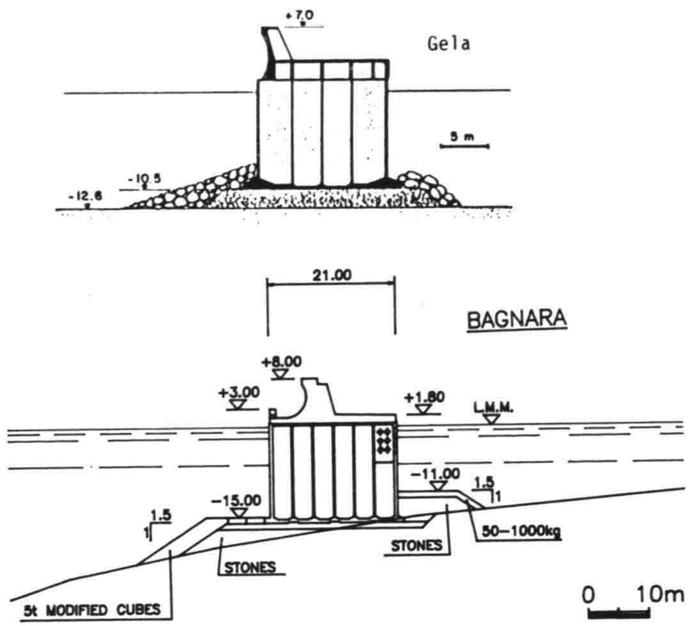



FIG. 6: Sections of two modern caisson breakwaters damaged in 1991

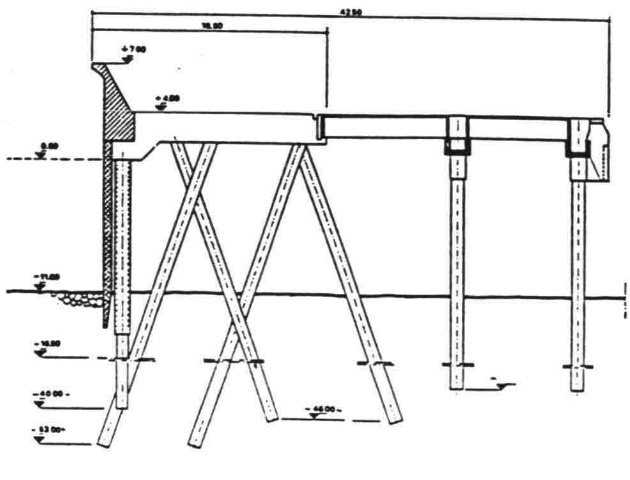


FIG. 7: Sections of the vertical breakwater at Manfredonia

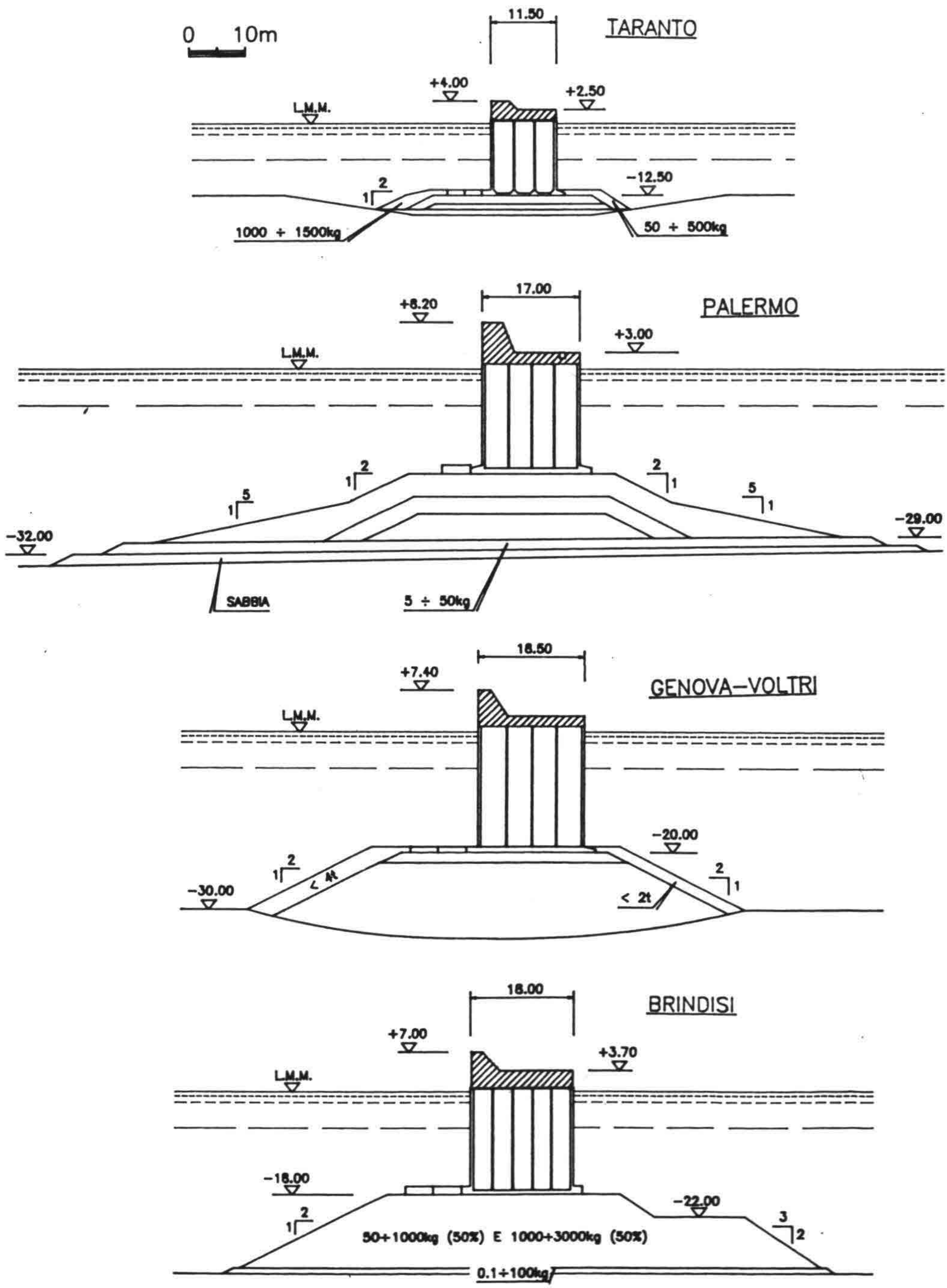



FIG. 8a: Sections of new italian caisson breakwater

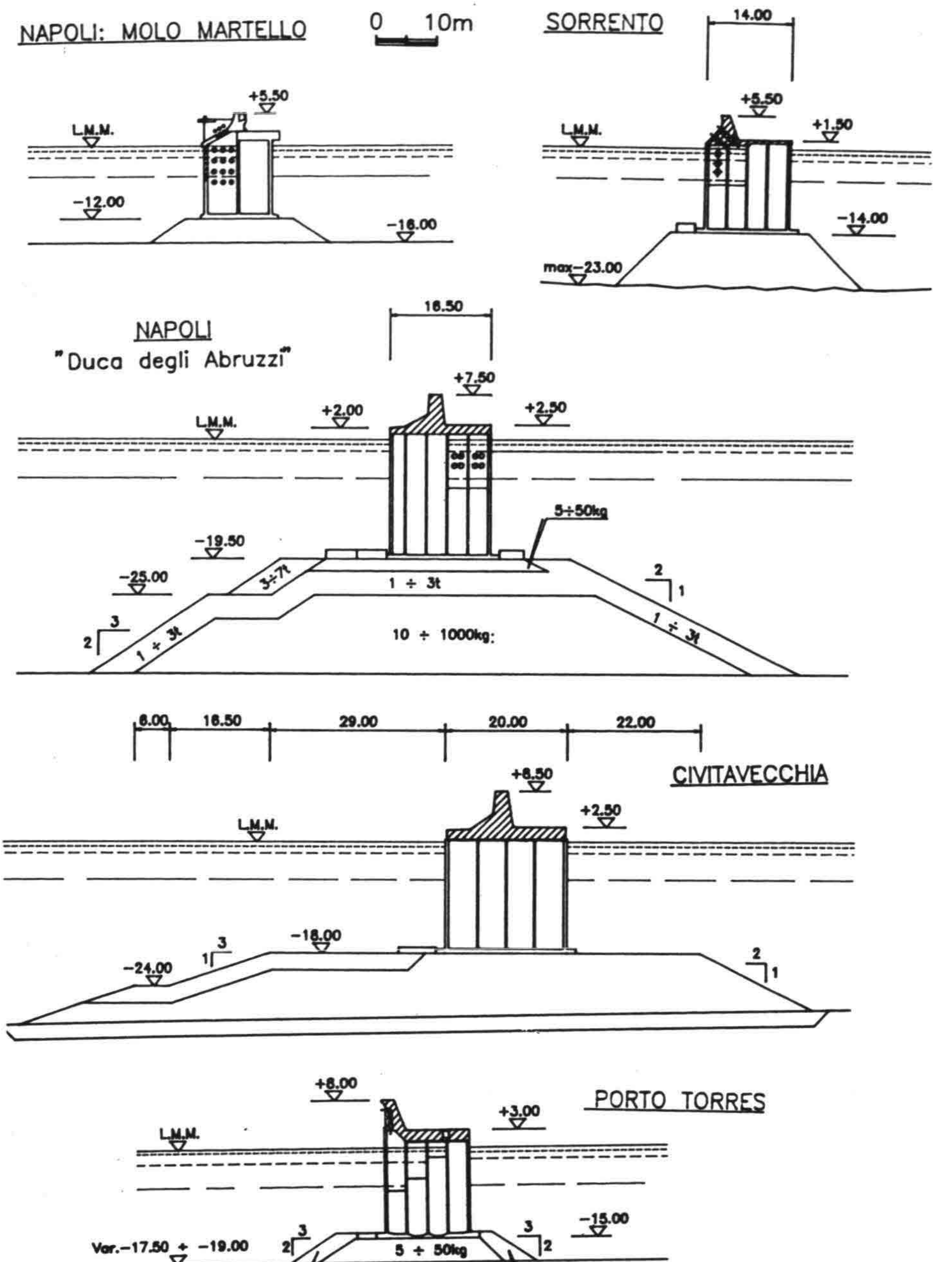



FIG. 8b: Sections of new italian caisson breakwater

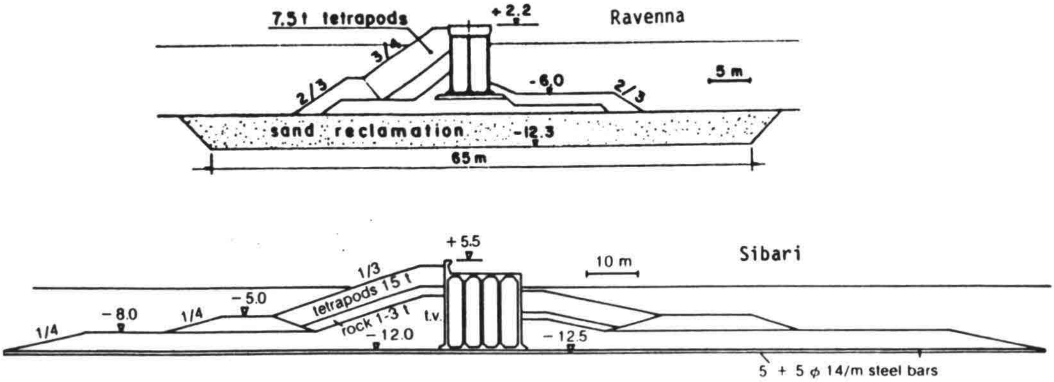



FIG. 9: Examples of composite breakwaters

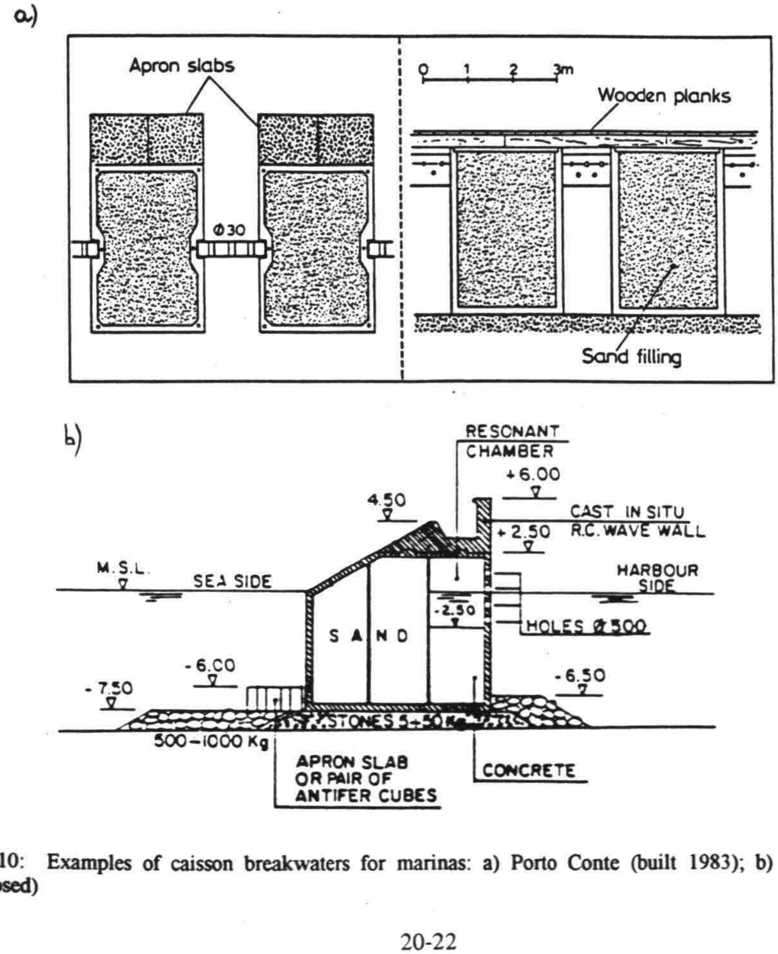



FIG. 10: Examples of caisson breakwaters for marinas: a) Porto Conte (built 1983); b) Bova Marina (proposed)

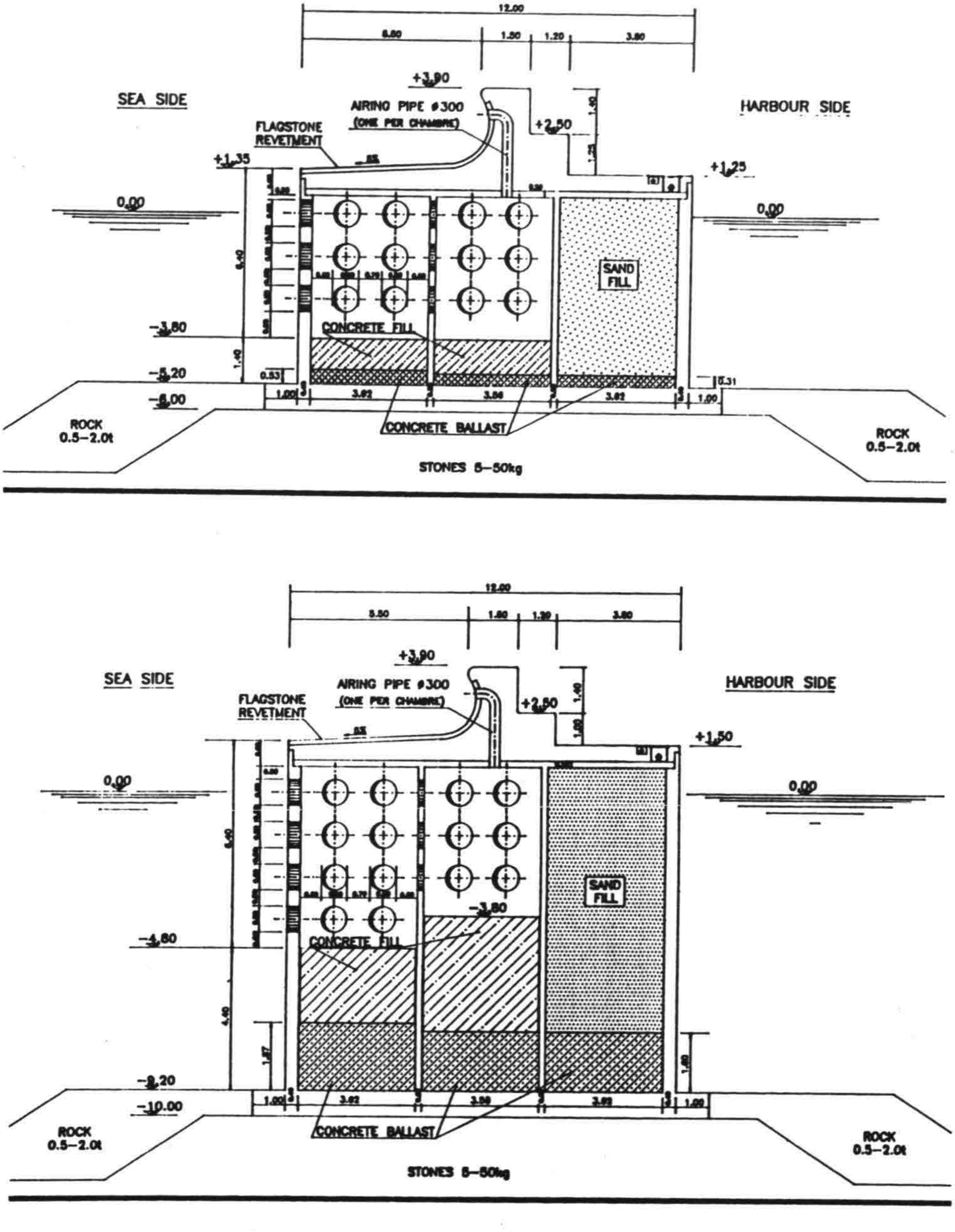



**FIG. 11: Design sections of the new caisson breakwater at Ponza (1992)**

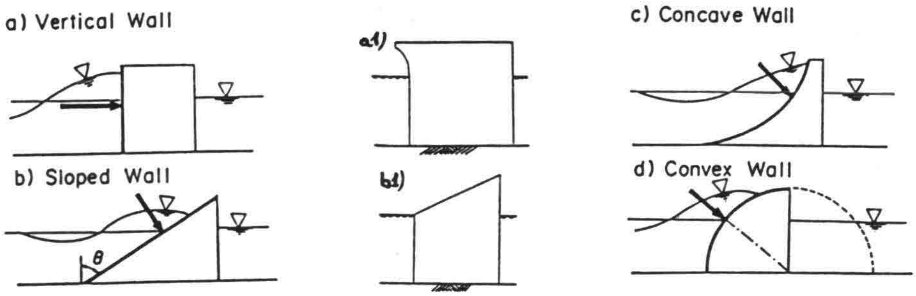



FIG. 12: Two schematic superstructure geometries for reducing wave overtopping (a) and wave forces (b)

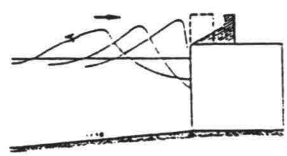


FIG. 13: Wave impact on variable parapet geometries (Gonzales et al., 1992)

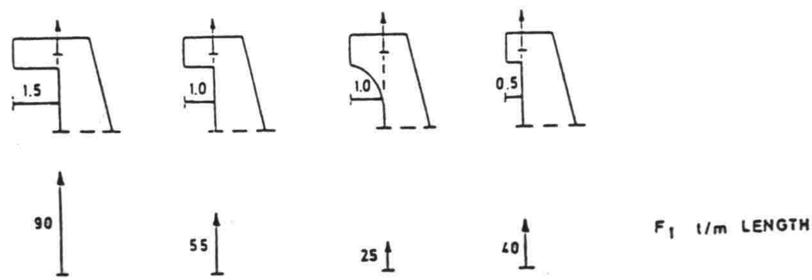
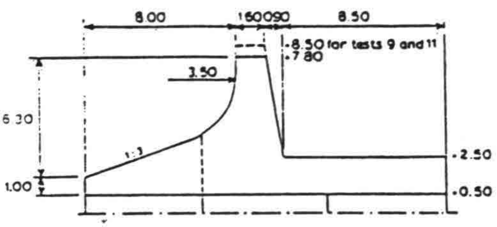
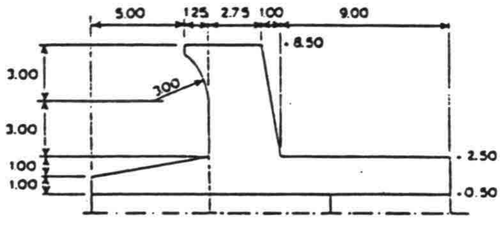


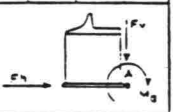

FIG. 14: Shear forces in crown heads (model test by DHI, Juhl, 1992)




PARAPET I

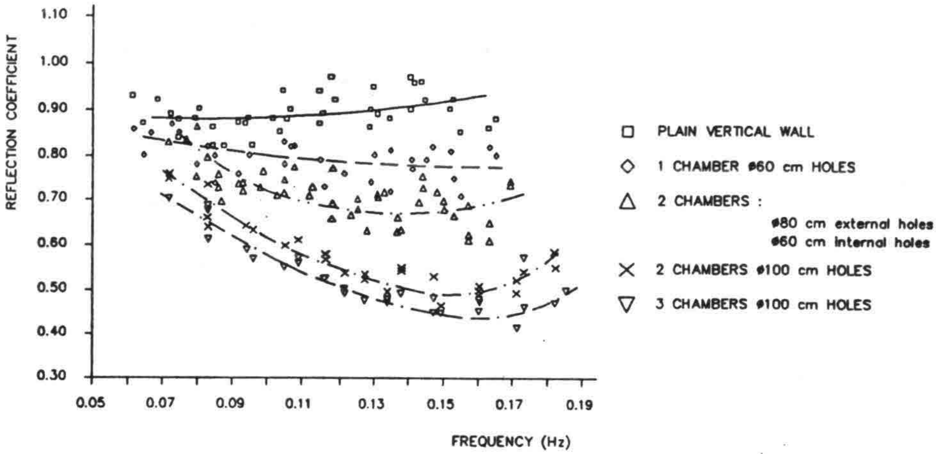



PARAPET II

| test | $H_{si}$ m | $\bar{T}_z$ sec | Fv $10^6$ N/caisson 1I | Fv $10^6$ N/caisson 2I | Fh $10^6$ N/caisson 1I | Fh $10^6$ N/caisson 2I | Ma $10^6$ Nm/caisson max | Ma $10^6$ Nm/caisson 1I | Ma $10^6$ Nm/caisson 2I |
|---|---|---|---|---|---|---|---|---|---|
| 9A | 8.4 | 9.9 | 17.4 | 16.1 | 42.7 | 41.3 | 464 | 421 | 410 |
| 9B | 9.6 | 10.8 | 21.0 | 19.3 | 46.1 | 43.2 | 519 | 449 | 424 |
| 10A | 8.4 | 10.0 | 17.9 | 17.1 | 46.4 | 43.6 | 609 | 488 | 459 |
| 10B | 9.6 | 10.9 | 20.7 | 18.9 | 48.3 | 47.4 | 640 | 524 | 497 |
| 11A | 8.0 | 10.4 | 15.7 | 14.5 | 41.5 | 39.0 | 487 | 431 | 398 |
| 11B | 9.4 | 10.7 | 17.6 | 16.3 | 45.1 | 42.9 | 566 | 473 | 438 |
| 12A | 8.1 | 10.3 | 15.0 | 13.7 | 45.1 | 40.5 | 564 | 514 | 470 |
| 12B | 9.7 | 11.3 | 18.6 | 16.5 | 50.0 | 47.4 | 722 | 589 | 554 |
| Forces and moments for a wave crest<br>Forces and moments for a wave trough | | | | | | | |  Fv Fh A $M_a$ | |
| 9A | 8.4 | 9.9 | -11.9 | -11.0 | -34.7 | -33.6 | -183 | -158 | -156 |
| 9B | 9.6 | 10.3 | -13.5 | -12.4 | -36.3 | -34.9 | -197 | -162 | -158 |
| 10A | 8.4 | 10.0 | -11.0 | -10.4 | -34.3 | -33.6 | -188 | -158 | -154 |
| 10B | 9.6 | 10.9 | -12.3 | -11.5 | -36.5 | -35.4 | -183 | -164 | -158 |
| 11A | 8.0 | 10.4 | - 7.5 | - 7.0 | -33.6 | -32.4 | -273 | -263 | -257 |
| 11B | 9.4 | 10.7 | - 8.4 | - 7.5 | -35.1 | -34.6 | -294 | -270 | -266 |
| 12A | 8.1 | 10.3 | - 8.3 | - 7.7 | -32.5 | -31.3 | -287 | -272 | -266 |
| 12B | 9.7 | 11.3 | - 9.5 | - 8.0 | -36.0 | -34.3 | -281 | -271 | -266 |

test 9 : parapet I - circular caisson
test 10: parapet II - circular caisson
test 11: parapet I - square caisson
test 12: parapet II - square caisson

FIG. 15: Model tests results for two differents parapets of Civitavecchia breakwater




FIGG.4a and 4b — RESULTS OF HYDRAULIC MODEL TESTS

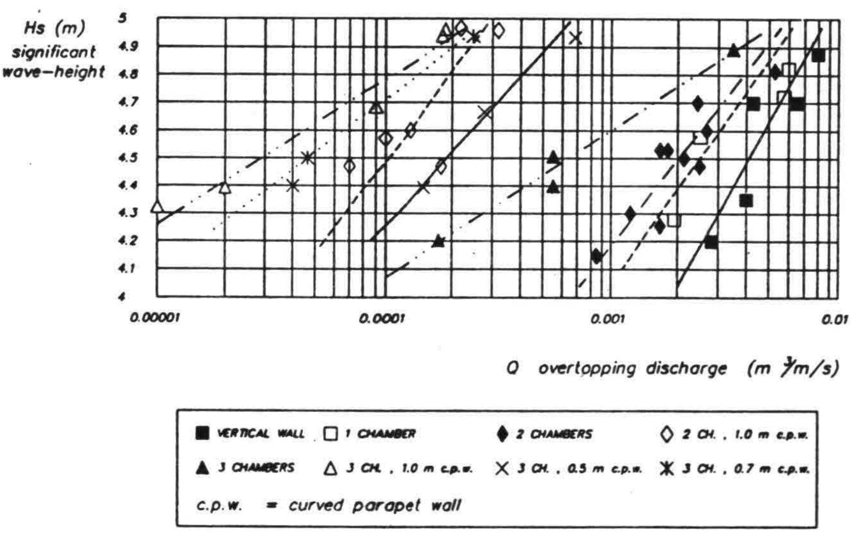



**FIG. 16: Reflection coeff. and overtopping discharge for various perforated model caissons**

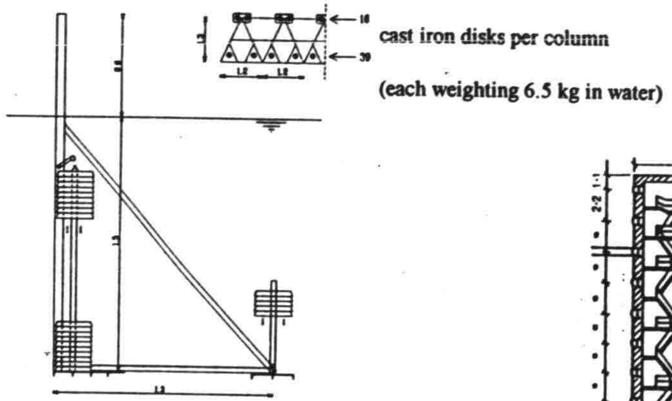



FIG. 17: The experimental small vertical breakwater for prototype measurements installed by Boccotti (1992) at Reggio Calabria

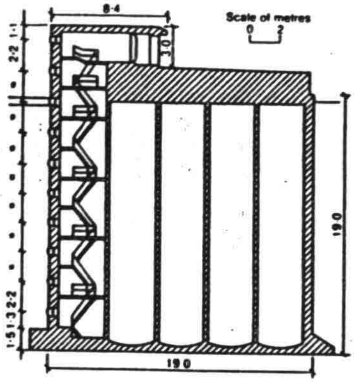
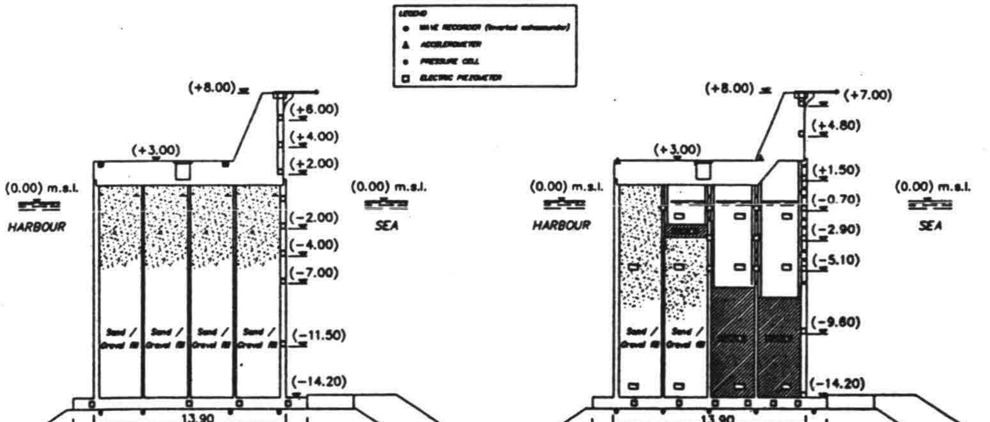



FIG. 18: Measurements station at Genoa caisson breakwater




FIG, 19: The instrumented caisson breakwater of Porto Torres industrial harbour (1992)