# VERTICAL CIRCULATION INDUCED BY A SUBMERGED BREAKWATER

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## Introduction: The concept of submerged breakwaters

Submerged breakwaters are structures whose crest height is below the main water level. From a hydrodynamic standpoint this type of structures show significant differences in behaviour with respect to the emerged ones. As an illustration (figure 1), it is well known that the wave height field in the lee of a structure is significantly different for emerged or submerged breakwaters or even the way it varies with the wave height to free-board, H to F, ratio. Also from a hydrodynamic standpoint, the enhanced mass flux over the submerged breakwater may lead to erosion on the lee-side instead of the conventional accretion and resulting tombolo formation.

From a morphodynamic standpoint, submerged breakwaters induce lower profile mobilities in addition to helping building a perched beach. In general terms, since a submerged structure exerts a smaller barrier effect, the morphodynamic impact should also be smaller (see for some additional discussion the papers by Van de Graaf and Sánchez-Arcilla and others dealing with the results of the DYNBEACH research project as presented in the Coastal Dynamics 95 and 97 conferences).

Finally, this type of submerged structures are normally associated to smaller impacts (e.g. smaller visual impacts, smaller ecological impacts associated to an enhanced water renovation which is essential for a tideless sea as the Mediterranean, etc.)

To understand and model the hydrodynamic behaviour of submerged breakwaters a 3D nearshore circulation model (see e.g. (Svendsen and Putrevu, 1994), (Sánchez-Arcilla et al, 1992),etc) is required in combination with a wave

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propagation model able to deal with depth induced breaking and the geometry and porosity of the submerged breakwater. There is nowadays a number of nearshore circulation models, featuring relatively sophisticated "effects" such as the transition zone (Nairn et al, 1990) (Dally and Brown, 1995) the full matrix of wave induced shear stresses $\langle \tilde{u}_i \tilde{u}_j \rangle$ (Rivero and Sánchez-Arcilla, 1993, 1996) or the mixing associated to vertical dispersion of the current -UV-terms (see e.g. Svendsen et Putrevu, 1994)-. However, these models cannot fully reproduce the experimentally observed hydrodynamic behaviour over a submerged breakwater. Some of the observational (Okayasu, 1989) (Rivero et al, 1997) features which these models cannot reproduce are: i) The mean water level gradient above the submerged breakwater, ii) The recirculation cell over the front slope of the submerged breakwater, and iii) The wave current interaction in front and above the submerged breakwater.

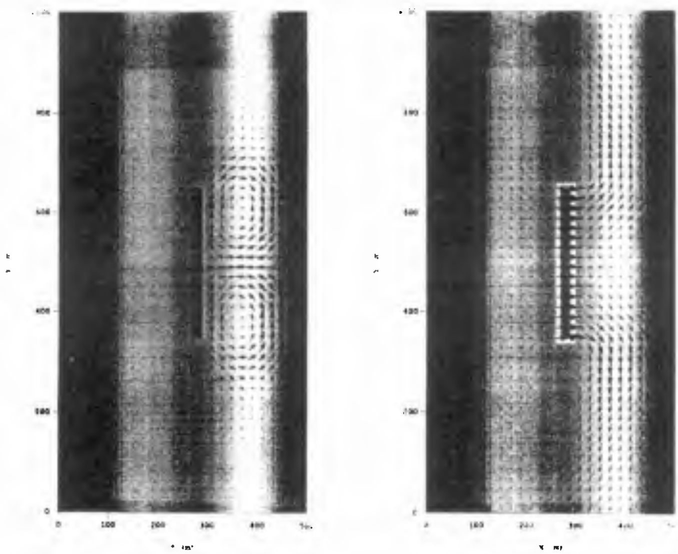

Figure 1. The differences in the vertically averaged circulation field induced by breaking waves in the presence of a shore parallel breakwater depending on whether it is an emerged or a submerged structure

The aim of this paper is to present the on-going development of a 2DV circulation model and a phase-averaged wave model to better understand the hydrodynamic behaviour of a submerged structure. The experimental information derived from previous research projects and the plans for a new set of mobile-bed tests with a submerged breakwater will also be presented.

Numerical modelling: the waves and currents around a submerged breakwater

i) THE WAVES

The wave model developed for the submerged breakwater case is an energetics one, based on the following equation:

$$\nabla\left[(C_g+U)\frac{E}{\sigma}\right]+\frac{D_b}{\sigma}+\frac{D_f}{\sigma}+\frac{D_p}{\sigma}=0 \qquad (1)$$

which states the conservation of wave action (due to the existence of strong currents above and in front of the submerged breakwater) considering the rates of "action loss" due to wave breaking, $D_b$ (occurring on the up-slope of the submerged breakwater) bottom friction, $D_f$ (because of the drag exerted by the submerged breakwater main armour units) and porous flow, $D_p$ (due to the permeable flow inside the submerged breakwater).

The $D_b$ term is evaluated using the (Battjes and Janssen, 1978) formulation for irregular wave tests and the (Dally et al, 1985) formulation for regular waves. Even though these formulae are intended for purely depth-induced breaking, all parameters are set to standard values. The maximum wave-height, Hm, in (Battjes and Janssen '78) is calculated using (Tomé, 1997):

$$Hm=\frac{2\pi\gamma_d}{k}tanh\frac{\gamma_s}{\gamma_d}\frac{kh_b}{2\pi} \qquad (2)$$

where $\gamma_d,\gamma_s$, are the deep and shallow water breaker indexes, to account for breaking waves of different frequencies (i.e. different $h/L$ ratios).

The $D_f$ term is evaluated according to (see e.g. Tolman, 1992)

$$D_f=\frac{2\rho}{3\pi}f_w U_{orb}^3 \qquad (3)$$

This term, although expected to be important above the submerged breakwater section, has not been analysed in any detail in this work. The friction coefficient used is a standard wave friction factor (see e.g. (Nielsen, 1992)).

The loss of energy (action) due to the existence of a permeable submerged breakwater is evaluated using (Sawaragi et Deguchi, 1992):

$$D_p=\frac{\rho}{4}\beta_p C_g H^2 \qquad (4)$$

where $\beta_p$ corresponds to the imaginary part of the wave number and is a function (see (Sawaragi and Deguchi, 1992)) of:

$$\beta_p = \mathrm{f}\left(\frac{d_{porous}}{h}, \frac{K_p\sigma}{\nu}, Cg\right) \tag{5}$$

with $d_{porous}$ the thickness of the porous layer (the lab tests described in next section use an impermeable core) and $K_p$ the permeability of such a layer.

The introduction of reflection in the proposed phased-averaged model -given essentially by equation (1) plus the corresponding dispersion relationship, including the current Doppler-shift- is accomplished using "external" linear superposition.

$$\eta = \mathrm{Re}\left(a \cdot e^{i(\theta-\Delta\theta)} + R \cdot a \cdot e^{i\theta}\right) \tag{6}$$

where Re means "real part", $ae^{i(\theta-\Delta\theta)}$is the incident (regular) wave train, and $\Delta\theta$ is the phase lag between incident and reflected waves due to their travel above a bottom with variable slope (figure 2):

$$\Delta\theta = 2\int_{x}^{xl} K dx \tag{7}$$

This expression coincides, for a plane sloping bottom, with the ones proposed by other authors (see e.g. (Méndez, 1997), (Sutherland and O'Donoghue, 1998)).

The reflection coefficient R is written in terms of a modulus and a phase (the former expressing the ratio between reflected and incident wave heights and the latter the uncertainty about the exact point where reflection starts.)

Within this framework it is relatively straight forward, although cumbersome, to derive other wave properties such as (Méndez, 1997):

$$U = \mathrm{Re}\left[-\frac{g}{w} a \cdot k \cdot f(z)\left[e^{i(\theta-\Delta\theta)} + \mathrm{Re}^{i\theta}\right]\right] \tag{8}$$

$$Mf = E\frac{K}{w}(1 - R \quad R^*) \tag{9}$$

$$Sxx = E\left[\left(1 + R_R^2 + R_I^2\right) \cdot \left(2 \cdot n - \frac{1}{2}\right) - R_R \cdot \cos\Delta\theta + R_I \cdot \mathrm{sen}\,\Delta\theta\right] \tag{10}$$

$$\text{where } n = \frac{1}{2}\left(1 + \frac{2 \cdot k \cdot h}{\mathrm{senh}(2 \cdot k \cdot h)}\right)$$

and where $M_f$ is the wave-induced mass flux and R* is the complex conjugate of R (all other variables having their usual meaning in this context).

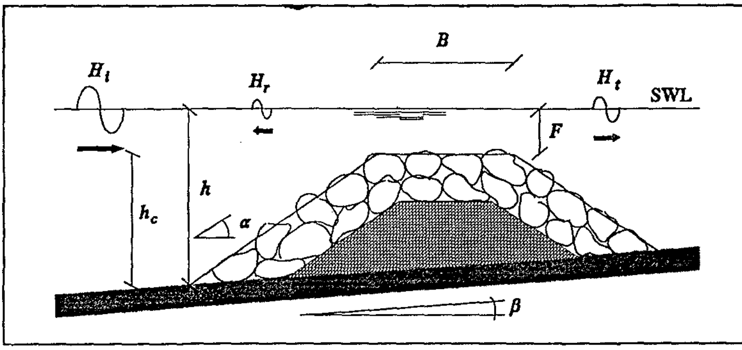


Figure 2. Schematization of the geometry and coordinates for a submerged breakwater under waves

## ii) THE DEPTH-AVERAGED CURRENTS

The mean water level variations, $\langle\eta(x)\rangle$ are derived from the vertically-averaged x-momentum equation:

$$\frac{\partial}{\partial x}\langle\int_{-h}^{\eta_{cr}} u^2 dz\rangle + \frac{\partial S_{xx}}{\partial x} + \rho g\left(h + \langle\eta\rangle\right)\frac{\partial\langle\eta\rangle}{\partial x} = \langle\tau_s\rangle - \langle\tau_b\rangle \qquad (11)$$

Where $U = I + \tilde{u}$ , $\eta_{cr}$ is the crest level and the rest of variables are as usual in surf zone analyses. The $S_{xx}$ term has two contributions: $S_{xx}$=$S_{xx}$, waves +$S_{xx}$, roller (see e.g. (Tomé, 1997)).

The conventional approach to evaluate $\langle\eta\rangle$ in beach surf-zones is to neglect the convective term $\left(u^2\right)$ and the $\tau_s$ and $\tau_b$ so that the x-momentum equation becomes:

$$\frac{\partial S_{xx}}{\partial x} + \rho g\left(h + \langle\eta\rangle\right)\frac{\partial\langle\eta\rangle}{\partial x} = 0 \qquad (12)$$

However, in front and above the submerged breakwater, according to the (laboratory) experimental evidence the $u^2$ terms cannot be neglected, since $U$ is large near the submerged breakwater and the $\frac{\partial}{\partial x}$ of $u$ is also large. This term is thus retained, assuming a uniform undertow distribution below $\langle\eta\rangle$ and a 1DV local mass balance (i.e. the mass flux above $\langle\eta\rangle$ cancels the undetow). With this, the convective term can be evaluated as:

$$\langle\int_{-h}^{\eta_{cr}} \rho u^2 dz\rangle = \int_{-h}^{\langle\eta\rangle} \rho U^2 dz + \int_{\langle\eta\rangle}^{\eta_{cr}} \rho u^2 dz = \frac{Mf^2}{\rho(h+\langle\eta\rangle)} + aMf^2 \quad (13)$$

with $a = \dfrac{64}{3\pi\rho H}$

iii) SOME RESULTS

A sample of the wave and mean water level results so obtained is shown in figure 3 for regular waves. The wave-height variation appearing in figure 3a) shows the oscillatory behaviour typically associated to the superposition of an incident and a reflected wave. The $\langle\eta\rangle$ variation with $x$ follows reasonably the experimental results although under-estimating the set-up in the downward slope of the submerged breakwater.

The results "without" reflection appear in figure 4 and show that the model without reflected waves is not able to predict the spatial oscillations of H in front of the submerged breakwater.

The mean water level predictions, although comparable, also tend to improve "with" reflection in the sense that an enhanced set-up is predicted (by the "full" model) in the downward slope.

The effects of the convective term (figure 5) affect basically the $\langle\eta\rangle$ predictions. The introduction of the $u^2$ term allows a deeper "trough" in $\langle\eta\rangle$ right at the beginning of the submerged breakwater crest just as it appears in the observations. The increased set-up right after the submerged breakwater is however not fully reproduced.

The same trends in results are obtained for irregular waves although in this case the H and $\langle\eta\rangle$ predictions are far better than the $Q_b$ results. It should be, however, remarked that the fraction of breaking waves (determined visually from video recording of the experiments) is being used far beyond its intended original applicability. The overall agreement between observations and predictions for H and $\langle\eta\rangle$ is however satisfactory.

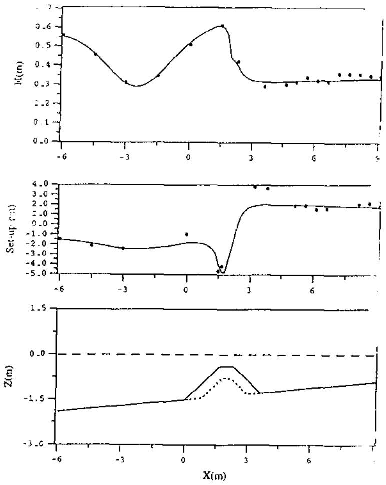


Figure 3. Sample results of the wave-height field and associated MWL for the "full" model presented in section 2 and the geometry depicted in the figure

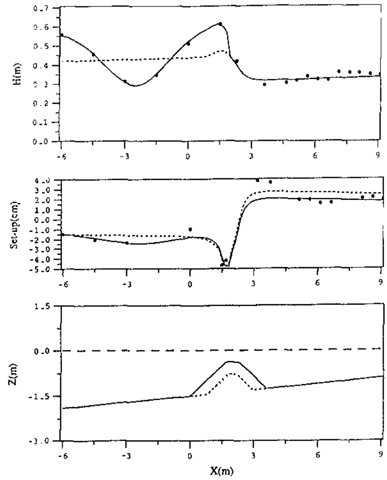


Figure 4. Same results as in figure 3 but without the reflection mechanism for the wave model (dashed line)

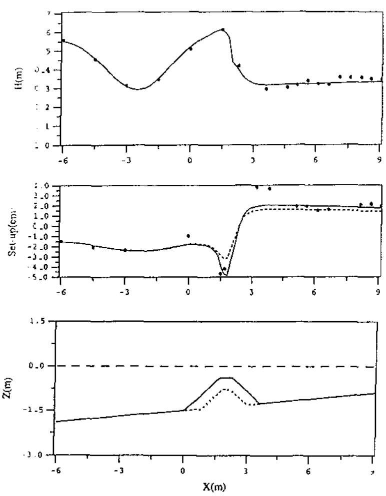


Figure 5. Same results as in figure 3 but without the convection terms in the $\langle \eta \rangle$ equation (dashed line)

iv) SOME REMARKS ON THE 2DV CURRENT FIELD

The 2DV current problem is being solved with the corresponding mass and momentum equations, using $\sigma$-co-ordinates so as to have the same vertical resolution for all water depths (i.e. well in front of the submerged breakwater or right above it).

The preliminary obtained results are based on the following x-momentum equation:

$$u\frac{\partial u}{\partial x}+w\frac{\partial w}{\partial z}+\frac{1}{\rho}\frac{\partial P}{\partial x}=-\frac{\partial}{\partial x}\langle\tilde{u}^2\rangle-\frac{\partial}{\partial z}\langle\tilde{u}\tilde{w}\rangle+\frac{\partial}{\partial z}\left(\upsilon_T\frac{\partial u}{\partial z}\right) \tag{14}$$

where the pressure is evaluated by

$$\mathrm{P}=\rho g(\eta-z)-\rho\langle\tilde{w}^2\rangle \tag{15}$$

and the x-derivative mixing term has been neglected (due to the expected greater vertical variations in flow properties).

The obtained results show a relatively high sensitivity to the bottom and mean water level boundary conditions, given by the corresponding shear stresses at $\langle\eta\rangle$ (associated essentially to the wave decay due to breaking, see e.g. (Sánchez-Arcilla et al., 1992), (Sánchez-Arcilla et al., 1994) ) and at $z_b$ (associated to the experienced bed friction due to the drag exerted by the submerged breakwater plus the flow inside of it). These preliminary results appear also to be quite sensitive to the $\langle\tilde{u}_i\tilde{u}_j\rangle$ wave shear stresses, in the sense that the resulting circulation field could feature or not a recirculation cell in front of the structure.

Because of this, the focus here will be exclusively on the evaluation of the wave stresses for the case of a sloping bottom with a submerged structure.

In general, the wave shear stresses, for the x-momentum equation, WSS$_x$, are given by:

$$WSS_x=-\frac{\partial}{\partial x}\langle\tilde{u}^2\rangle-\frac{\partial}{\partial z}\langle\tilde{u}\tilde{w}\rangle \tag{16}$$

There will be a non-zero contribution to $WSS_x$ whenever there is a spatial variation of the wave-height field . Far the case of a sloping-bottom with a submerged structure there are two physical origins for these $H$ variations:

a) The shoaling/breaking processes, for which

$$\frac{\partial\langle\tilde{u}\tilde{w}\rangle}{\partial z}=\langle\tilde{u}\tilde{\omega}\rangle-\frac{1}{2}\left[\frac{\partial}{\partial x}\left(\langle\tilde{u}^2\rangle-\langle\tilde{w}^2\rangle\right)\right]\approx-\frac{1}{2}\left[\frac{\partial}{\partial x}\left(\langle\tilde{u}^2\rangle-\langle\tilde{w}^2\rangle\right)\right] \tag{17}$$

according to (Rivero and Sánchez-Arcilla, 1995) and assuming that the oscillating motion has no vorticity, $\tilde{\omega}$ (even for the breaking wave case).

With this, the resulting shear stresses, $WSS_{S-B}$, are given by:

$$WSS_{S-B} = -\frac{1}{2}\left[\frac{\partial}{\partial x}\left(\frac{H}{2}\frac{gT}{L}\right)^2 \frac{ch(2kz)}{ch^2(kh)}\right] \tag{18}$$

The order of magnitude of $WSS_{S-B}$ is $-0.1$ m/sec$^2$ for shoaling waves and +0.1 m/s$^2$ or greater for breaking waves (governed by a saturation law of the type $H_b = \gamma h$). For more details see (Vergés and Sánchez-Arcilla, 1998).

b) The reflection processes for which (assuming a horizontal bottom), the corresponding shear stresses, $WSS_{I-R}$, are given by:

$$WSS_{I-R} = -\frac{1}{4}kH^2w^2\frac{\sin 2kx}{sh^2(kh)} \tag{19}$$

This term turns out to be, thus, a function of 2kx which represents the phase-lag between reflected and incident waves (Vergés and Sánchez-Arcilla, 1998). It is due to the fact that the correlations between incident and reflected wave velocities need not be necessarily 0:

$$\begin{aligned} \langle \tilde{w}_i \tilde{u}_r \rangle &\neq 0 \neq \langle \tilde{u}_i \tilde{u}_r \rangle \\ \langle \tilde{w}_i \tilde{w}_r \rangle &\neq 0 \neq \langle \tilde{u}_i \tilde{w}_r \rangle \end{aligned} \tag{20}$$

The order of magnitude of the modulus of this term is around 0.5 m/s$^2$.

The $WSS_x$ term for the general case of sloping bottom with submerged breakwater can therefore be written as (Vergés and Sánchez-Arcilla, 1998):

$$WSS_x = \left(1 + K_R^2\right)WSS_{S-B} + K_R WSS_{I-R} \tag{21}$$

where $K_R = \dfrac{H_R}{H_I}$ is the reflection coefficient.

Model testing: The fixed and mobile bed experiments.

i) RIGID-BED TESTS

A set of hydrodynamic tests to analyse the submerged breakwater hydrodynamic performance and "impact" have been performed in a number of recent research projects carried out at the LIM/UPC. The tests, which are now beginning to be fully processed, cover a range of $H/L_0$ (wave steepnesses) and F/H (relative freeboard) values (figure 6). For most of the tests twelve "verticals" were obtained (figure 7). Each vertical was instrumented with 5 electromagnetic currentmeters and 6 pressure sensors.

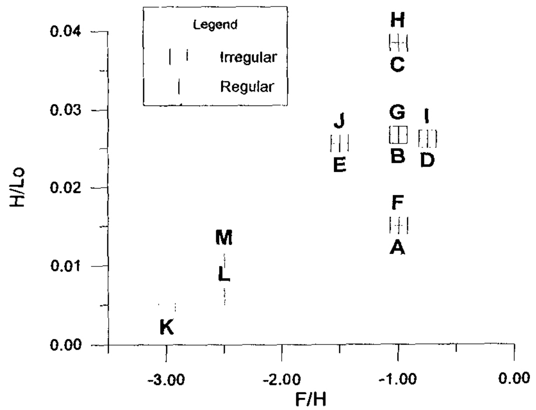


Figure 6. Schematization of the range of wave steepnesses ($H/L_0$) and free-boards (F/H) covered in the "hydrodynamic" experiments

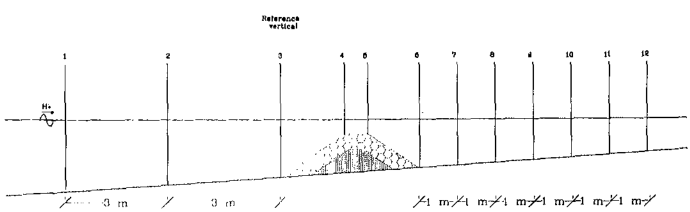

Figure 7. Distribution of "instrumented" verticals (with current-meters and pressure sensors) in the experiments

The preliminary results show transmission coefficients, $K_T$, in close agreement with state of art predictions (see e.g. (Van der Meer, 1990)). The dependence of $K_T$ on $F/L_0$ (freeboard over the deep-water wavelength) rather than on $F/H_{m0}$ (as is usual in the state of art) allows improved predictions and a "clearer" variation with the wave steepness $H/L_0$ (figure 8). From this type of analysis it is possible to derive engineering formulae for the design of submerged breakwaters such as:

$$K_T = K_T(I_r)\big|_{F=0} + f(I_r)F\big/L_0 \tag{22}$$

This expression shows that $K_T$ starts -for F=0- with a value (a function of Iribarren's parameter) for submerged breakwaters reaching the mean water level, and then $K_T$ increases linearly with $F/L_0$, the slope being also a function of

Irribarren's parameter, Ir. The same type of analysis can be applied to e.g. the reflection coefficient or, in general, the hydrodynamic behaviour of the structure.

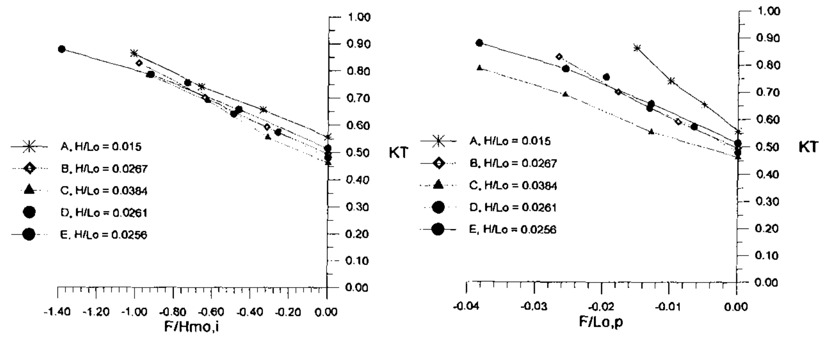


Figure 8. Free-board, F, dependence on wave-steepness for two possible ways to make F dimensionless

The observed circulation fields also display the expected features (e.g. a strong return flow over the crest of the structure, enhanced with respect to a real submerged breakwater since in the flume there must be 1DV mass flux compensation) plus some less "obvious" characteristics, such as the vortex in the up-ward slope of the structure.

## ii) MOBILE BED TESTS

The mobile bed tests will be carried out as part of the MAST-III SCARCOST project whose main aim is, in this context, to analyse the hydrodynamic behaviour of the submerged breakwater and, in particular, the scouring in front of it.

The tests, scheduled for the 2nd term of 1999, will "repeat" the geometry, wave and "free-board" conditions tested for a rigid bed. This will allow to concentrate on the hydrodynamics in front of the structure and the associated bed evolution (figure 9). The observations include, as shown in the figure, free-surface elevations in front and over the structure, the current field in front and on the up-ward slope and pore pressures in the sand in front of the submerged breakwater. A PIV system, recently implemented in the CIEM flume of the LIM/UPC, will also be used to record the spatial velocity field in front of the structure and, in particular, in the vicinity of the scouring section.

Each of the mobile bed tests will have a minimum of 15,000 waves so as to allow for a "reasonable" development of the scouring in front of the structure (see e.g. (Herbich et al, 1984)).

The expected results will allow a characterisation of scouring as a function of hydrodynamic conditions and also the derivation of practical recommendations for the design of submerged breakwaters.

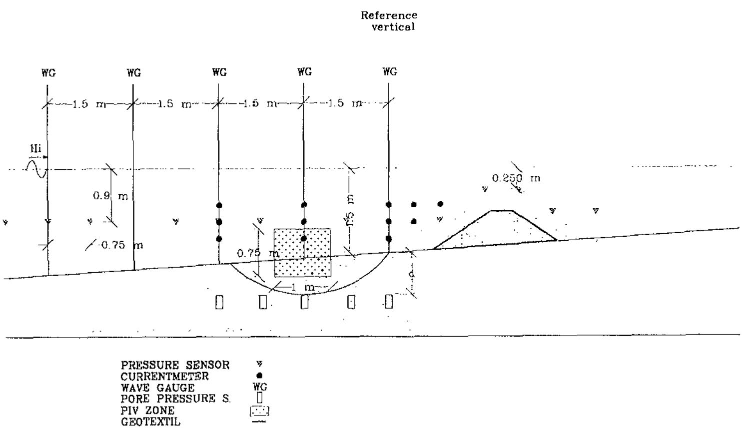


Figure 9. Mobile bed test arrangement, showing the hydrodynamic measurements, the PIV area and the pore pressure sensors

## Conclusions: what has been done and remains to be done.

The following conclusions must be considered as on-going remarks since a large part of the experimental work is still to be processed and parts of it (mobile bed tests) even to be done. The numerical simulations are in a similar position since the adaptation of existing models for the submerged breakwater case is still very much under development.

Within this frame, the main on-going conclusions are the following:

i) The hydrodynamic performance of submerged breakwaters is quite sensitive to the design parameters. This requires careful selection and analysis since otherwise the submerged breakwater may not behave as expected (e.g. it could hardly affect the incoming waves) or even have contrary effects (e.g. enhanced erosion in the lee-area due to enhanced mass flux over the structure).

ii) The wave modelling with the submerged breakwater geometry appears to require a reflection coefficient "modulus" (to account for the decrease in $H_r$) and a reflection coefficient "phase" (to account for the exact origin of reflection). The same (i.e. real and imaginary parts) happens with the wave number, whose imaginary part is required to account for the energy loss in the porous (granular) layer.

iii) The current modelling appears to require the convective terms for the accurate predictions of set-up variations and the full "wave" stresses (due to the sloping bottom and the co-existence of incident and reflected wave fields) for the correct simulation of the current pattern.

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