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FIGURE 6
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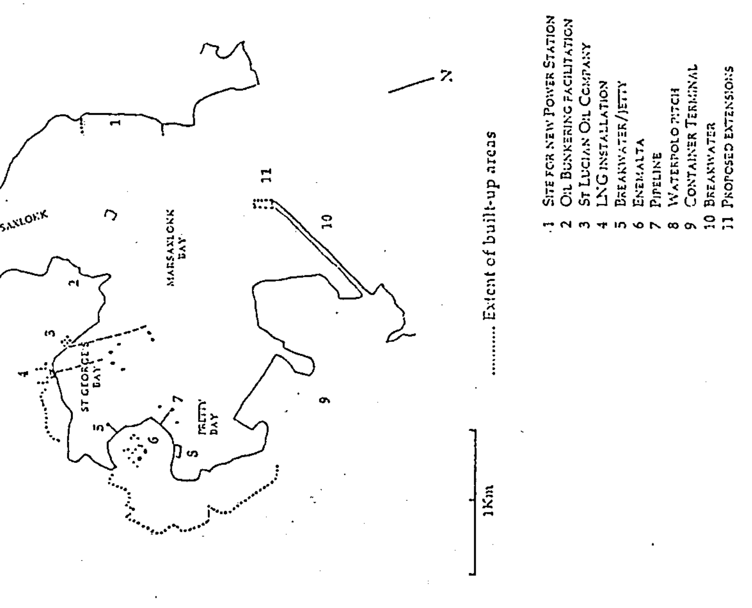

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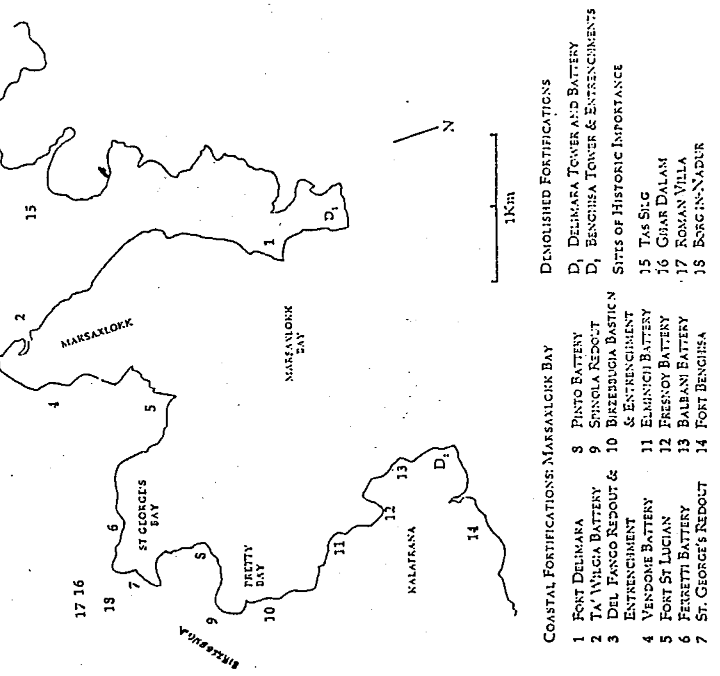

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Wave induced pore water pressure in a rubble-mound sea-wall

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Abstract

This paper deals with the results obtained through an experimental prototype study aiming to evaluate the behaviour of a particular sea-wall under wave attack and to determine the pressure field inside the core of the structure itself. For this study some sections of the "Murazzi" sea-wall, in the extreme south of the Venetian Lagoon shore-line have been instrumented. The data obtained through the gauges have been analysed and the interaction between incident waves and structure, evaluated during a big storm, is reported.

A finite element model was then applied to the described structure in order to estimate the wave induced pore pressure inside the core of the rubble mound sea-wall. Numerical results and in-situ measurements have been compared in order to calibrate the model, estimating the proper values of the geotechnical parameters to be assumed for a correct schematization of the investigated structure.

In particular the effect of parameters, such as the volumetric water content, the pore-water storage capacity and the permeability on the non confined seepage flow were taken into account in the model applications.

Introduction

One of the most usual structures in maritime works, aiming to protect a shore or a coast line against wave attacks, is either a detached breakwater or a sea-wall. These structures consist of a core covered with one or more layers of stones.

Globally these structures act as an inverse filter which prevents the loss of finer material with the primary layer able to withstand incident wave action.

The stability of a rubble-mound structure is mainly based upon four different criteria:

- the overall stability, generally related to the sole primary layer
- the unit stability, referred to a single element of the armour layer
- the stability of the armour layer units, referred to the unit strength
- the geotechnical stability including the overall soil stability of the core.

The first and second items are largely mentioned in literature so that the stability is generally ensured by accurate calculations and physical model tests. The unit strength is usually verified through prototype full-scale tests performed prior to the design. The last stability analysis, however, is generally not taken into account with sufficient interest; some structure failures, in fact, stressed the importance of considering the possible migration of materials out of structure core and the failure risk related to pressure gradients.

Only very simple and conservative principles are generally used in engineering practice for filter design; for example to prevent the penetration of the core material into the external layer the following relation (Graauw et al., 1984) is often considered:

$$\frac{(D_{15})_{layer}}{(D_{85})_{core}} < 5 \qquad (1)$$

Such a relation between the grain sizes of the core and the covering layer appears too simple for a correct consideration of the hydraulic phenomena occurring within the porous media. A more appropriate approach should for example consider the local hydraulic gradients which must be compared with the critical value, over which a significative particle motion can occur (Bezuijen, 1987).

Prototype measurements

Field measurements were performed in an interesting sea-wall (known as "Murazzi") located along the Venetian Lagoon shoreline. In Fig. 1 the studied area is shown: that structure stability for the defence of the lagoon and of the town of Venice itself from the dangerous floods and from the wave attacks that sometimes occur along the Adriatic coast does appear to be important.

In recent years the zone was the site of new maritime works that strongly reinforced this thin littoral zone; during these works some sections of the structure were instrumented in order to measure the internal forces acting in the core of the structure during bigger storms. For such purposes a complete set of pressure gauges was located in three different sections of the "Murazzi" sea-wall (Ruol, 1990). A typical section of the final work is drawn in Fig. 2; it can be seen that

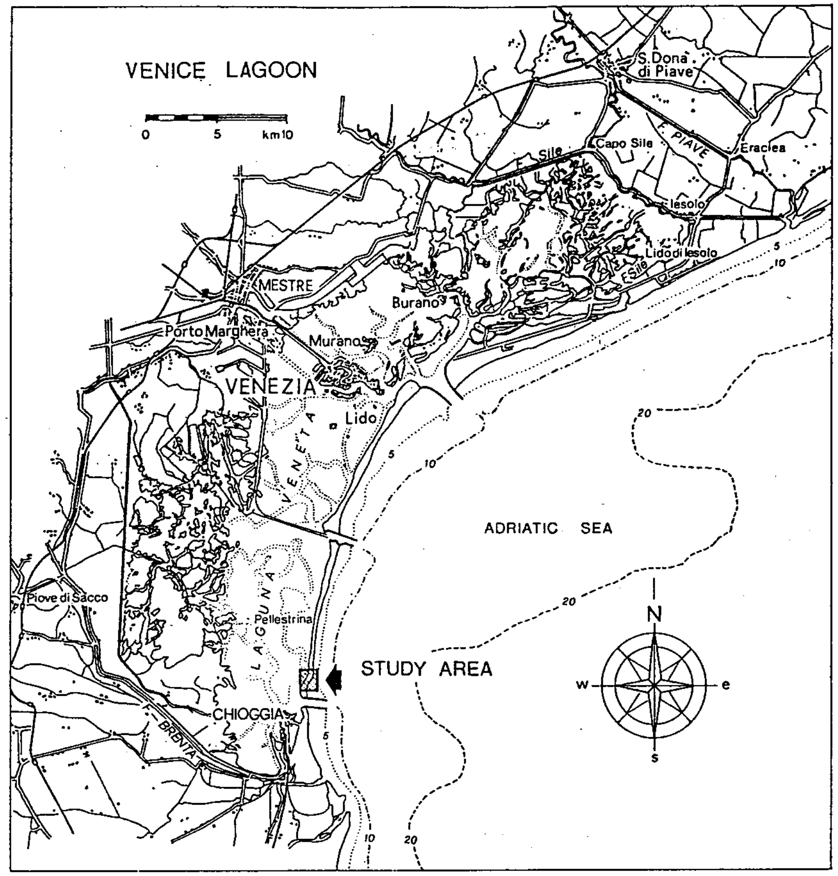

Fig. 1. *Map of Venice Lagoon and study area location.*

two different gauges were located inside the core of the structure, while another gauge was located sea-ward of it for measuring incident wave heights.

The analysed structure is essentially composed of a primary layer of big natural rocks placed over the finer material core. In each section the internal gauges were located preparing a borehole 15cm in diameter, placing the transducers inside these holes and filling them with appropriate grain-size gravel. All instruments were located at a depth which ensured the continuous submergence of the instruments even during a wave attack occurring with lower tide levels. The data were recorded with no interruptions during storm periods.

An example of the data registered during a big autumn storm is reported in Fig. 3. In it the surface elevation, measured through the off-shore gauge, is

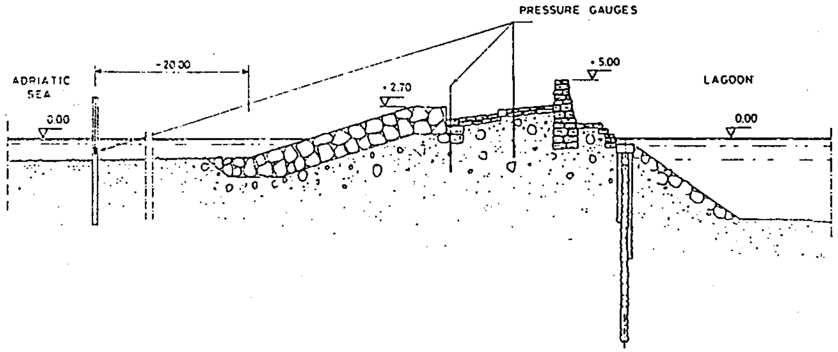

Fig.2. *Section of "Murazzi" sea-wall and location of pressure gauges.*

compared with the inner pressures measured by the internal gauges. A considerable decrease may be observed in the pressure field through the porous structure. During the peak of the analysed storm (characterised by an incident significative wave height $H_{1/3}$ = 140 cm and a significative period T = 4.5 s), for example, the "attenuation factors", defined by the expression:

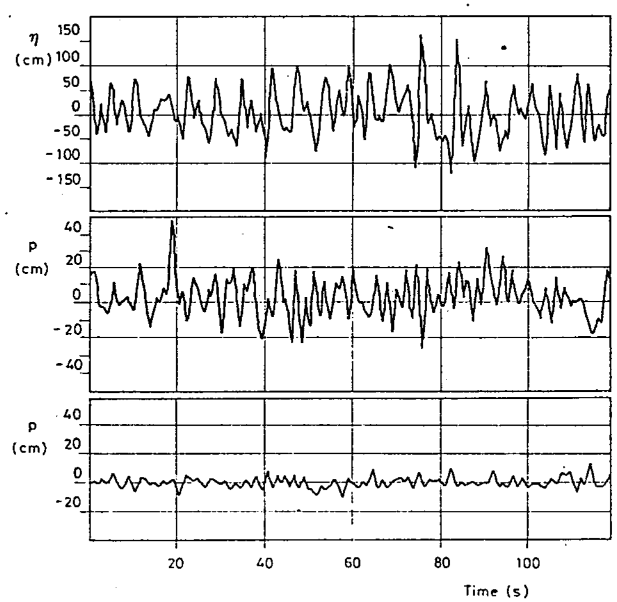

Fig.3. *Example of data recording (elevation η, pressure p) during a wave attack.*

$$K'_c = H_{1/3}/P_{1/3} \qquad (2)$$

in which:
- $H_{1/3}$: significative wave height (average of highest one-third of wave heights)
- $P_{1/3}$: significative pressure (average of highest one-third of pressures),

were calculated. Referring to the described section (Fig. 2) it was evaluated a value of K'_c = 3.3 for the gauge located closer to the sea and K'_c = 10.4 for the gauge more distant from the sea.

Theoretical approach

The flow through unsaturated porous media involves both air and water phases; so two differential equations are necessary for characterizing the flow of air and water. Fortunately for most geotechnical problems a "single-phase" (i.e. water phase) flow approach is sufficient; the air flow is neglected, the air phase is assumed to be continuous and the pore-air pressure u_a is supposed equal to atmospheric pressure. These assumptions are valid for a degree of saturation S lower than 85%, while for values greater than 85% the air pores could be actually occluded (Lam et al., 1987).

In saturated soils Darcy's law is generally accepted and the coefficient of permeability, function of the void ratio, is generally assumed to be constant, even when transient seepage problems are considered.

The water flow through unsaturated soil can also be studied using Darcy's law, but the coefficient of permeability cannot be assumed to be constant any more, depending on water content (or on matric suction) and degree of soil saturation. In fact water flow occurs only through the voids filled with water, therefore soil can be treated as a saturated soil with a reduced water content (Fredlund, 1993). The coefficient k is often described as a singular function of the degree of saturation S, or the volumetric water content θ_w.

As the soil starts to become unsaturated, air then replaces some of water in the largest pores; so water flows through the smaller pores and the tortuosity of the flow paths increases. An increase in the matric suction (u_a - u_w) of soil causes a further decrease in pore-water volume; the air-water interface gets closer and closer to the soil particles, and the k-coefficient rapidly decreases.

For engineering purposes the stress state of an unsaturated isotropic soil can be expressed by two independent stress state variables, (σ - u_a) and (u_a - u_w), and the constitutive equation for the water phase becomes (Fredlund and Morgenstern 1976, 1977):

$$d\theta_w = m_1^w \cdot d(\sigma - u_a) + m_2^w \cdot d(u_a - u_w) \qquad (3)$$

where:

- θ_w: volumetric water content equal to the ratio between the pore-water volume V_w and the total volume V
- m_1^w: slope of the $(\sigma - u_a)$ vs. (θ_w) plot when $d(u_a - u_w) = 0$
- m_2^w: slope of the $(u_a - u_w)$ vs. (θ_w) plot when $d(\sigma - u_a) = 0$
- σ : total stress
- u_a, u_w: pore air and pore water pressures.

Keeping the above combination of the stress state variables, the effects of a change in total normal stress can be separated from those caused by pore-water variation. If the pore-air pressure is taken to be equal to the atmospheric pressure and the total stress constant, during a transient process, the volumetric water content depends only on the pore-water pressure u_w:

$$\theta_w = V_w / V = m_2^w \cdot (u_a - u_w) \qquad (4)$$

The pore-water storage parameter m_w^2 represents the rate of change in the amount of water taken in or retained by the soil, as a consequence of pore-water pressure variation. The storage curve (u_w vs. θ_w) may be nearly linear, as for clayey soils (this means a low rate of desaturation), or non linear, as for silty and sandy soils (fast rate of desaturation). In the latter case the greatest changes in volumetric water content occur at low negative pore-water pressure; beyond a critical point relatively small changes in θ_w occur. Volumetric water content variation can also occur for positive pore-water pressure changes: the slope of positive portion of the storage curve is equivalent to the well-known coefficient of volume change m_w.

According to the Darcy's law, within the range of laminar flow, a linear proportionality between the hydraulic gradient and the flow velocity is generally assumed, while in turbulent regimes the hydraulic gradient i can be expressed by the equation:

$$i = av + bv^n \qquad (5)$$

where a and b are constants and n ranges between 1 and 2. Experiments have shown that for any Reynolds number lower than 12 the flow can be treated as laminar (Leps, 1973).

Model Description

The analytical approach previously described can be easily related to the study of a rubble structure in which the role played by the porosity and permeability of the different layers of the structure does appear to be important. These

parameters in fact are fundamental to evaluating the intensity of in and out flows, as well as in determining the location of the water surface level within the structure.

The aim of the study was to model the wave motion within the core of the sea-wall and to calculate the instantaneous pore water pressure under waves action. Such a numerical model should take into account the following basic factors:
- external motion of waves and uprush onto the breakwater face
- energy dissipation produced by wave breaking and friction at the breakwater surface
- evaluation of boundary conditions under the primary layer
- unsteady water flow through the porous core
- in non completely saturated zones flows of water and of air must be considered.

No model appears nowadays to be able to incorporate all of these factors and therefore some simplification must be acknowledged.

For example it is generally accepted that the kinetic energy related to the dynamic wave transmission in the porous structure can not be taken into account in dense and fine pores.

The calculations were based on a finite element software, named PC-SEEP (Geo-Slope International, 1987), that allows to study one and two-dimensional water flows through porous materials, considering either saturated steady state or transient saturated-unsaturated flows. Furthermore the model allows the flow and the hydraulic heads in the unsaturated zone to be determined.

The basic assumptions of the model are:
- constant total stress, i.e., no loading or unloading can occur during the transient process
- pore air pressures are constant and equal to atmospheric pressure during transient process
- the difference between the flow entering and leaving a soil volume, in a certain period of time, is equal to the change in the volumetric water content.

The last condition can be expressed by:

$$\frac{\partial}{\partial x}\left(k_x \cdot \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_y \cdot \frac{\partial h}{\partial y}\right) = \frac{\partial \theta_w}{\partial t} = \frac{\partial (V_w / V)}{\partial t} \qquad (6)$$

where:
- h: the total head
- k_x, k_y: the coefficients of permeability on horizontal and vertical directions
- t: the elapsed time.

By differentiating eq.(2) and substituting it in (6) it results:

$$\frac{\partial}{\partial x}\left(k_x \cdot \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_y \cdot \frac{\partial h}{\partial y}\right) = m_2^w \cdot \frac{\partial (u_a - u_w)}{\partial t} = m_2^w \cdot \rho \cdot g \cdot \frac{\partial h}{\partial t} \qquad (7)$$

where ρ is the density of water and g the gravitational constant.

Furthermore the model is based on the validity of Darcy's law in both saturated and unsaturated soil; under unsaturated conditions the coefficient of permeability k varies, depending on soil volumetric water content θ_w and negative pore water pressure u_w. When u_w decreases, the volumetric water content θ_w and the coefficient of permeability k of soil also decrease. The k-coefficient assumes its highest values when the soil is completely saturated.

For modelling the water flow through unsaturated soil it is necessary to estimate the relationships between k , θ_w and u_w.

Changes in θ_w depend on changes in stress states and on soil properties: therefore the volumetric water content θ_w can be only related to matric suction (u_a - u_w) and in particular to pore water pressure u_w.

Numerical simulation and results

In order to define the nature and the geotechnical characteristics of soils to be introduced as input data in the numerical model, an accurate in-situ and in-laboratory soil investigation (boreholes, piezometric measurements, disturbed and undisturbed samplings, classification tests) was carried out.

Three different zones were defined:

- primary layer, composed of natural stones
- core, composed of coarse granular soil (cobbles, gravel and sand)
- natural soil, forming the base of the rubble mound, mainly composed of clayey and sandy silts.

As far as waves are concerned the results of the described prototype measurements were considered.

After having determined the geometric and geotechnical parameters and boundary conditions, the problem of modelling the turbulent water flow inside the primary layer was studied. The analysis of the water flow through the primary layer in fact involves heterogeneity and turbulent phenomena, that cannot be studied by Darcy's law and a different law proper of the turbulent flow should be taken into consideration. Due to the model features, which assume only laminar water flows, it was necessary to neglect the phenomena within the primary layer and to carry out the numerical simulation only within the core. The wave boundary conditions were directly imposed on the primary layer-core interface.

The relations of the permeability coefficient and volumetric water content against the pore-water pressure are plotted in Fig. 4 both for the core and the natural base. Such relations have been estimated on the base of soil nature and on its grain size distribution (Elzeftawy A., Cartwright K. 1981), (Fredlund D.G., Morgenstern N.R. 1976, 1977).

The finite element mesh of the model was composed of 319 triangular elements (Fig. 5).

According to prototype recording, the wave period T was assumed equal to 4.5 s. The numerical analysis was carried out considering a time step of 0.25 T.

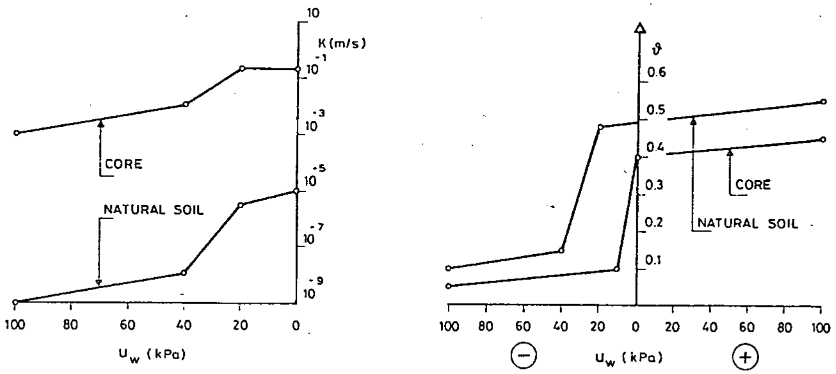

Fig.4. *Unsaturated permeability function (a) and moisture retention curve (b) of granular core soil.*

The calculation was stopped after 20 wave periods, as soon as results appeared to be stable with time.

During each time step average total heads, seepage velocities and gradients along horizontal (X) and vertical (Y) directions were evaluated. Some results referred to the 8 elements located on the sloping interface between primary layer and core are summarized in Tab. 1 in which the lower the element number, the deeper its location. The hydraulic gradients appeared to range from 0 to 0.22 and, being these values very far from the critical ones, the analysed structure stability was confirmed.

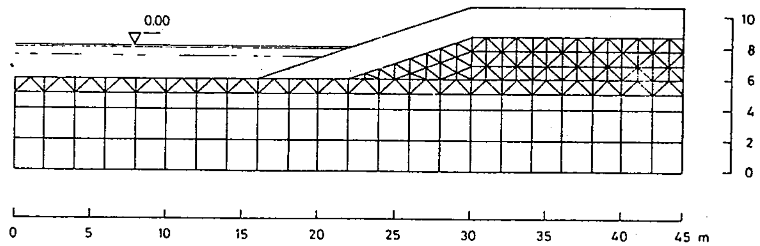

Fig.5. *Finite element mesh layout.*

Element Number	Average Total Head (m)	X - Direction Velocity (m/sec)	Y - Direction Velocity (m/sec)	i_x	i_y
TIME STEP : 1	(0.25 T)				
77	8.091	-1.7E-02	-2.1E-02	-.086	-.103
79	8.102	-1.7E-02	-2.6E-02	-.085	-.132
81	8.111	-1.1E-02	-3.3E-02	-.057	-.166
84	8.120	-3.5E-03	-4.0E-02	-.017	-.201
88	8.123	1.0E-02	-4.4E-02	.051	-.218
92	8.118	3.4E-02	-3.2E-02	.170	-.158
97	8.103	4.5E-02	-1.5E-02	.227	-.076
103	8.084	4.4E-02	-1.0E-03	.218	-.005
TIME STEP : 2	(0.50 T)				
77	7.996	-1.5E-02	8.5E-03	-.073	.042
79	8.002	-1.3E-02	9.9E-03	-.063	.050
81	8.007	-1.3E-02	1.1E-02	-.062	.055
84	8.012	-1.4E-02	1.1E-02	-.068	.057
88	8.018	-1.7E-02	8.5E-03	-.084	.043
92	8.025	-1.9E-02	2.6E-03	-.094	.013
93	8.033	-8.7E-03	-1.3E-04	-.043	-.001
97	8.033	-1.6E-02	-1.9E-03	-.080	-.010
103	8.040	-1.0E-02	-3.0E-05	-.052	.000
TIME STEP : 3	(0.75 T)				
77	7.933	-9.2E-03	1.8E-02	-.046	.088
79	7.937	-1.1E-02	1.8E-02	-.057	.088
81	7.943	-1.8E-02	1.6E-02	-.088	.082
84	7.949	-2.1E-02	1.6E-02	-.105	.079
88	7.959	-2.5E-02	1.4E-02	-.127	.071
92	7.971	-3.2E-02	8.9E-03	-.158	.044
97	7.986	-3.3E-02	9.5E-03	-.165	.048
103	8.000	-3.1E-02	7.4E-03	-.154	.037
TIME STEP : 4	(T)				
77	8.005	1.1E-02	-4.4E-04	.054	-.002
79	8.001	7.4E-03	-4.4E-04	.037	-.002
81	7.998	4.7E-03	3.4E-04	.024	.002
84	7.997	2.9E-03	1.7E-03	.015	.009
88	7.996	1.3E-03	4.4E-03	.007	.022
92	7.996	-3.8E-03	4.7E-03	-.019	.023
97	7.998	-7.9E-03	3.9E-03	-.040	.019
103	8.003	-1.1E-02	1.8E-03	-.057	.009

Tab.1. *Numerical results obtained along the primary layer-core interface*

The wave motion causes an increase and decrease in pore water pressure with time. These fluctuations were calculated through the model which defined an unsaturated zone inside the core in which the phreatic lines were varying throughout the wave period (Fig. 6). Furthermore, according to prototype measurements, a set-up in mean water level was calculated within the breakwater.

Opposite to the usual confined water flows, in the analysed case, the phreatic line ($u_w = 0$) was noticed not to be a flow line, being it crossed by velocity vectors during the different phases of incident waves.

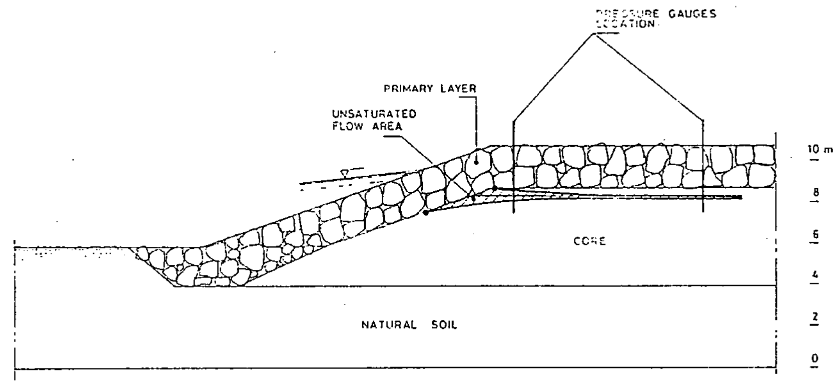

Fig.6. *Unsaturated flow area location within the structure core.*

Conclusions

The prototype measurements performed in some sections of the "Murazzi" sea-wall showed a considerable decrease in the pressure transmission through the structure core: the pressure reduced to about one-third at about 10m within the structure and to about one-tenth at about 15m.

Based on such measurements it was possible to calibrate a numerical model and to determine the correct relations between the permeability coefficient as well as volumetric water content and pore-water pressure to be assigned to the structure core.

After the calibration, the model allowed for the evaluation of some parameters as the pore-water pressure distribution and velocity field within the sea-wall under wave attacks. The results confirmed the analysed structure stability.

It did appear the numerical model to be very useful also for engineering purposes for an accurate geotechnical stability analysis. In fact the model allows for the evaluation of hydraulic gradients (that can be compared to the critical values) as well as pore water pressure distribution and led to a design more correct than the usual ones, simply based on relations between the grain-size distribution of the different layers of the structure.

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Modelling beach management schemes

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Abstract

The coastline is subject to numerous competing interests including the requirement for flood protection, recreation and amenity uses together with environmental pressures. Coastal managers must balance these sometimes conflicting concerns to arrive at a solution acceptable to all interested parties. There has been increasing recognition of the benefits of basing coastal defences around beaches, which also provide many amenity, aesthetic and environmental benefits. Beach management and replenishment schemes must be actively designed and, being part of a dynamic environment, the future response must be modelled as a necessary step in the design optimisation process. This paper sets out the design philosophy developed for beach replenishment schemes in the UK and examines its adoption in other study areas including the Mediterranean.

Background

The UK coastline is long and varied in character. Beaches of cobbles, shingle, gravel, coarse and fine sand are all found, often with these materials occurring as mixtures. Wave climates range from very mild, eg within estuaries, to exceptionally severe on the edge of the North Atlantic. Tidal ranges and currents can also be significant factors in shaping the coast.