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A COMPUTATIONAL APPROACH TO DESIGN CODES FOR TSUNAMI-RESISTING COASTAL STRUCTURES

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ABSTRACT

The recent mega-tsunami event on the 26th December 2004, revealed the importance and the necessity of designing "tsunami-resisting" structures. The present study paves a path towards the organization of "design codes" and engineering practices addressing this important issue. A computational model consisting of the nonlinear shallow water equations in the 2DH space, in the presence of an embedded typical building is synthesized and numerically solved by an explicit second order finite difference scheme on a solution domain discretized by a staggered Arakawa "C" grid. The normalized integral of the hydrodynamic loads distributed on the building surface in the direction of the wave propagation is plotted against the normalized length of the structure for various building walls configurations. The importance of the building orientation with respect to the wave propagation and the importance of the openings in the walls are revealed by means of a number of numerical experiments. An experimental verification is pending. The presented numerical tool, in conjunction with the structural resistance analysis of the building walls, leads to operational results as far as the building design specifications are concerned, and mainly in order to resist the expected "design tsunami" and to operate as an "ad hoc" shelter for the people in the vicinity.

KEYWORDS: Tsunami, Coastal Structures, Wave Models

INTRODUCTION

The recent mega-tsunami event on the 26th December 2004 that shocked a major part of South-East Asia and the unique videotaped scenes of the tsunami evolution over the coastal zone, reminded the international scientific community to recognize and model the tsunami generation, propagation and coastal inundation processes as final operational goals to plan the mitigation of the tsunami effects on the coastal communities, to forecast the behavior of man-made structures, and to design them in such a way as to withstand the catastrophic effects of tsunami to the highest possible degree.

The major observed negative effects refer to the destruction of buildings, the erosion of the coastal structures, like the roads and the bridges, the destruction of harbour works, the voluminous sedimentation of the river outflows, and the dynamic effects on the moored vessels.

A wide range of scientific issues belonging to the domain of coastal engineering appear in relevance to the mitigation of tsunami effects, having as common scope the proper design of a number of coastal structures in areas prone to the appearance of tsunamis (Silva et al., 2000).

The design codes (such as the one developed by Koutitas et al., 1986) imposed by the relevant authorities to the local engineering communities eventually have to be properly modified in order to incorporate the tsunami component in the design specifications. This study follows the previous ones (Koutitas et al., 1986; Tinti, 1991; Demetracopoulos et al., 1994) that already considered the tsunami risk, and proceeds in this direction by extending contemporary numerical models of hydrodynamic loading and of scouring around structures applied in coastal engineering.

The present study aims at the inauguration of such a procedure regarding the resistance of coastal buildings to the hydrodynamic loads applied by the water moving in a quasi-periodic oscillatory motion around the building due to tsunamis. It makes use of a typical numerical model of nonlinear shallow water equations, and its aim is to reveal the importance of the building configuration, at least in the zone of the ground and first floor (usually inundated), to resist the resultant hydrodynamic force in presence of the tsunami water masses.

THE TECHNICAL PROBLEM AND THE SCOPE OF THE STUDY

The technical problem when tsunami waves, like those of the 26th Dec 2004 event, propagate over a low coastal area, is the capacity of the buildings, especially the low ones, to withstand the horizontal hydrodynamic loads exercised by the water on the lateral surface of the building. Other non-negligible factors may be the capacity of the water to erode the foundation of the structure and the addition to the impact loads by the solid debris, like floating cars and heavy objects, colliding with the building.

The hydrodynamic forces, a resultant of drag and inertia forces, act on the horizontal section of the structure and as a first approximation may be considered as 2DH ones. The situation does not differ much from the classical Morisson-type loads on piles under waves, as far as it concerns the relative ratio of the structure length to the wave length, but the section of the building has a very complicated geometry in comparison to a pile geometry.

The relation between the inertia and the drag part of the total hydrodynamic load is dominated by the Keulegan-Carpenter number (N_{KC}) and the Reynolds number (*Re*) of the flow. From their magnitudes it is concluded that the flow is always in the high Reynolds number regime and that the inertia and drag components contribute to the total hydrodynamic loading with the same intensity. For a building with a façade of D = 20 m, a maximum velocity of the order of U = 3 m/sec and a wave period of the order of T = 100 sec, one may take $N_{KC} = 15$ and $Re = 10^7$.

The scope of this study is (i) to adapt properly the mathematical model of the nonlinear shallow water equations for the above physical situation, (ii) to examine the various geometric configurations of a typical building, with all the outside walls active while some of them (like glass walls) broken, in order to conclude on the worst possible scenarios, and (iii) to derive the first conclusions about the procedure to be followed and the relation between the structure geometry and the wave-induced loadings. The hydrodynamic loads are not based on the classical Morisson or Keulegan-Carpenter approaches used in pile design, but they are computed by the integration of the hydrodynamic pressure values computed on the surface of the structure.

THE MATHEMATICAL MODEL AND THE NUMERICAL SOLUTION

Assuming an oscillatory 2DH flow around an obstacle the mathematical model describing the spatio-temporal evolution of the mean horizontal velocities $u(x,y,t)$ and $v(x,y,t)$ (over depth) and of the free water surface elevation $\zeta(x,y,t)$, the mass conservation and the horizontal force equilibrium principles lead to the following equations:

$$\frac{\partial \zeta}{\partial t} + \frac{\partial (hu)}{\partial x} + \frac{\partial (hv)}{\partial y} = 0 \tag{1}$$

$$\frac{\partial u}{\partial t} + \frac{u \partial u}{\partial x} + \frac{v \partial u}{\partial y} = -g \frac{\partial \zeta}{\partial x} - \left(\frac{f_w}{2} \right) u_t \frac{u}{h} + N \left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} \right) \tag{2}$$

$$\frac{\partial v}{\partial t} + \frac{u \partial v}{\partial x} + \frac{v \partial v}{\partial y} = -g \frac{\partial \zeta}{\partial y} - \left(\frac{f_w}{2} \right) u_t \frac{v}{h} + N \left(\frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2} \right) \tag{3}$$

where f_w is the friction factor and u_t is the total wave velocity $\left(u_t = (u^2 + v^2)^{1/2} \right)$. Friction at the bottom is neglected over the length scales of the applications (of the order of some meters), and only the influence of the eddy viscosity is considered.

This set of equations forms a simple operational model readily applicable for the description of the pulsating motion of the water. It is a first approximation for the complicated physics of the unstable flow at high Reynolds numbers around rough large bodies (like the building cross-sections), using as “turbulence closure” the classical second order terms of the right hand side and a “turbulent eddy viscosity coefficient" *N*, expressed according to the physical oceanographic approach as a function of the local vorticity (Love and Leslie, 1979; Madsen et al., 1988):

$$N = C\Delta x^3 \left|\mathrm{grad}\,\omega\right| \tag{4}$$

where ω is the vorticity and C is a dimensionless coefficient with $O(C) = 0.1$. The total depth h is the sum of the initial depth h_0 and the free surface elevation ζ.

The boundary conditions completing the field equations are (a) the upstream incident wave, described by a predefined velocity, which is a function of the incident wave amplitude (a) and period (T), (b) the lateral symmetry conditions, and (c) the downstream free linearised radiation condition:

$$u = \zeta\sqrt{(g/h)} \tag{5}$$

The model can accept as upstream boundary condition any signal, periodic or not, like waveforms suggested by offshore tsunami models incident to the coastal location under study. The flow domain is depicted in Figure 1. The types of models and the general approach used here are not unique, but only indicative of the existing technological possibilities.

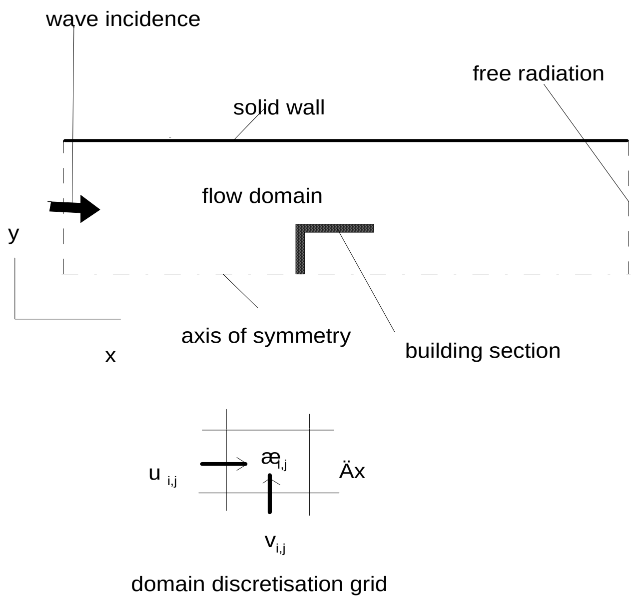


Fig. 1 Computational flow domain

The required hydrodynamic force on the building is, in general, the summation of two components, the pressure and the frictional forces $\left(F_{px}, F_{fx}\right)$.

The first is deduced from the integral of the pressure (actually the pressure head ζ) on the outer immersed surface of the building:

$$F_{px} = \int \rho g \zeta h ds_x \tag{6a}$$

where ds_x is the *x*-component of the differential length ds.

The second is deduced from the integral of the frictional components on the building surfaces parallel to the wave direction. Without going into the boundary layer details, this component is parameterized by a simple quadratic form:

$$F_{px} = \int \rho C_f U^2 h ds_y \tag{6b}$$

where U is the local slip velocity (parallel to the building surface) and ds_y is the component of ds in the y-axis, normal to the wave direction. The friction coefficient on the rough wall surfaces and due to high Reynolds values is of the order of $C_f = 0.05$.

The numerical solution of the system of the three differential equations (Equation (1)-(3)) with the boundary conditions is achieved after the discretization of the flow domain by a staggered Arakawa "C" grid and by the use of an explicit centered FD scheme. The stability of the numerical solution is controlled by the CFL criterion, and the overall behavior of the numerical solution has been proven to be robust and stable. The normalization of the results is done on the grounds of the characteristic magnitudes of the phenomenon like the water depth and the corresponding wave celerity, $c = (gh)^{1/2}$, and the overall lengths of the structure in the wave direction *A* and across the wave direction *B* (i.e., aspect ratio *A*/*B*). The normalized load F_x is estimated from the sum of Equation (6a) and of Equation (6b) divided by $\rho gHhB$ where H is the wave height. The normalized wave length is estimated by the ratio $(gh)^{1/2} T / A = L / A$.

OPERATIONAL APPLICATIONS

The operational applications of the model aim at revealing the importance of the building's typical horizontal section, for various wave periods, in determining the resultant horizontal hydrodynamic force during the wave passage. Five different building configurations are considered, as illustrated in Figure 2. The closed impervious rectangular section is the reference configuration. The rotation of this section by 45^0 and the wave incidence along a building diagonal is compared to the reference configuration. The collapse of some walls on the building perimeter (like glass walls) creates multiply-connected, concave etc. shapes resulting in different hydrodynamic loads.

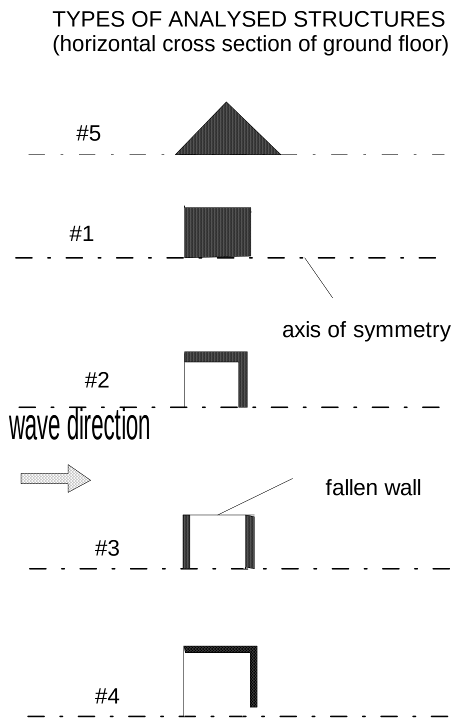


Fig. 2 Building configurations

The oscillating hydrodynamic loads on the various building configurations for 3 different wave periods (60, 120 and 180 sec) are depicted in Figure 3. Although the earthquake generated tsunamis have periods of the order of 10^2-10^3 sec, the applications illustrating the hydrodynamic loading are done with periods of 10^2 sec periods referring to landslide-generated local tsunamis.

The water vector fields for 4 time instants within a wave period for the building configuration # 2 are depicted in Figure 4. The generation of inertial eddies, and their evolution in time and space, is a first indication of the importance of the geometry of the immersed-in-the-flow "body" to the development of the hydrodynamic loading. Figure 5 finally is an integrated presentation of the maximum wave-induced loads for the various building configurations and the three wave periods.

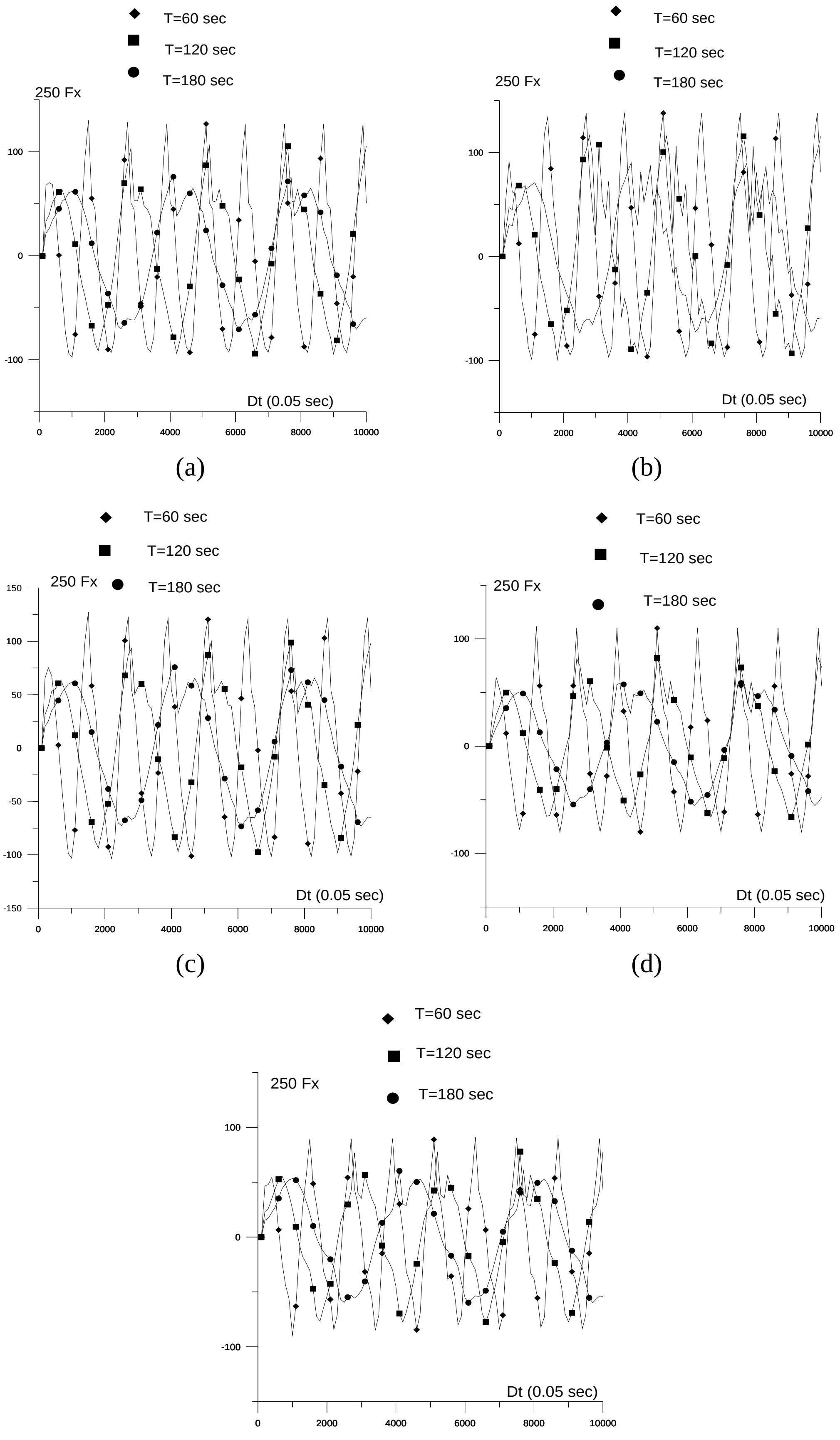


Fig. 3 Oscillating hydrodynamic loads for (a) building configuration # 1, (b) building configuration # 2, (c) building configuration # 3, (d) building configuration # 4, (e) building configuration # 5

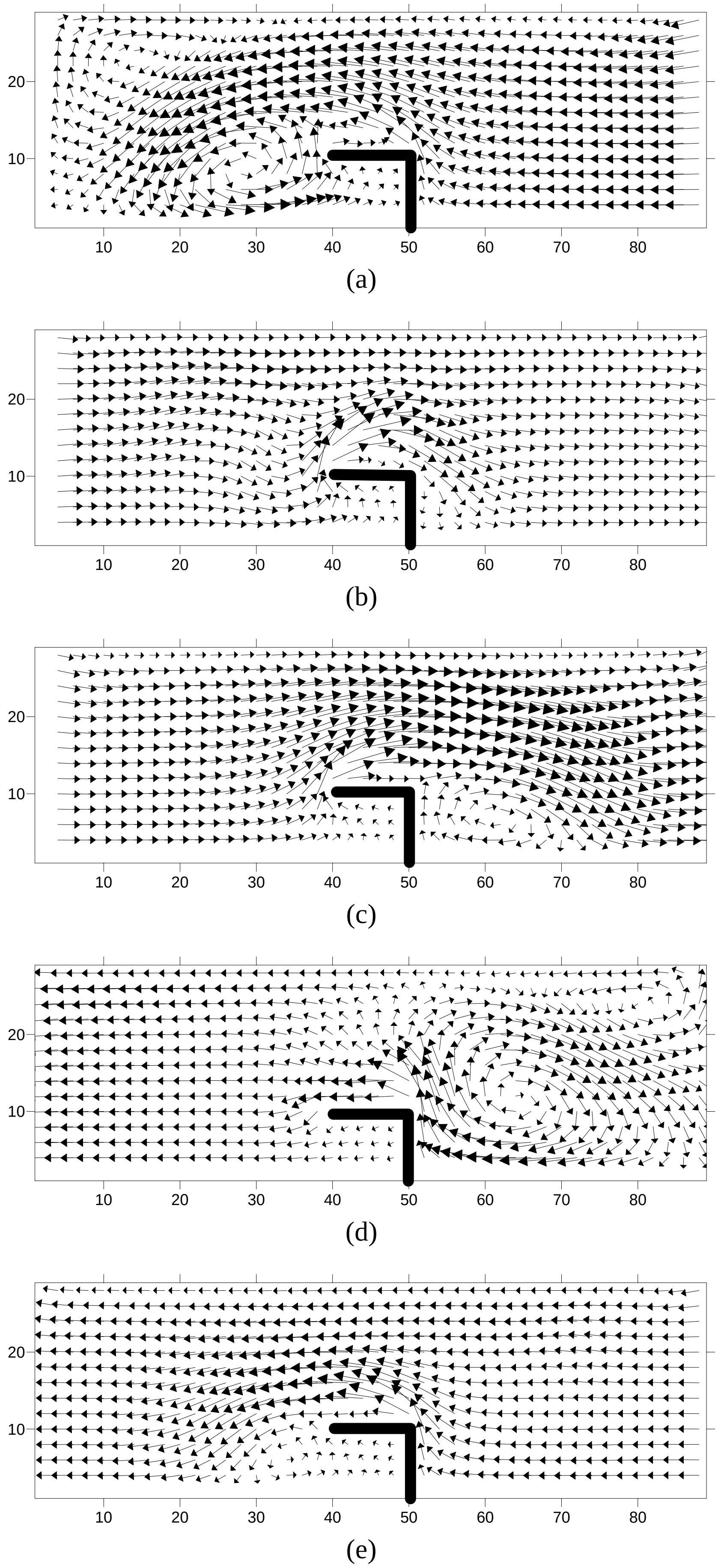
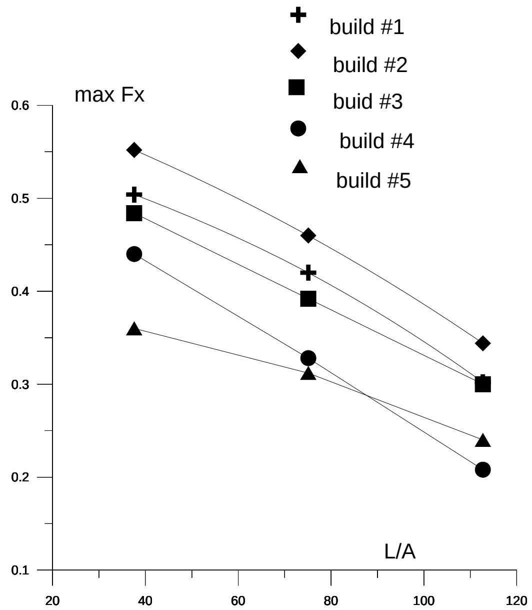


Fig. 4 Vector field for (a) $t = T$, (b) $t = T + 0.2T$, (c) $t = T + 0.4T$, (d) $t = T + 0.6T$, (e) $t = T + 0.8T$



influence of geometry on load

Fig. 5 Maximum wave induced loads for the various building configurations

The empirical coefficients used are taken from general coastal engineering formulae since the exact calibration can only be done experimentally in cases of tsunamis. In spite of this, even the comparative character of the study reveals that there exist important issues, like the orientation of the buildings and the open or closed forms of the ground floors that influence considerably the hydrodynamic loads and the safety of the structures. More specifically it is revealed that

a) the increase of the wave period results in a considerable decrease of the total hydrodynamic load,
b) the proper orientation with respect to direction of the wave propagation of the building is crucial, and
c) the partial collapse of the outside walls may create adverse loading situations that have to be either prevented or forecast.

SCOURING/SEDIMENTATION

When a structure (such as a building, a bridge pile, etc.) is placed in an erodible bed, scour will take place around it due to the action of tsunami waves. This process is of importance in connection with the stability of the structure. Extensive scour around the structure may reduce its stability, thus leading to its failure. The process of scour is mainly due to two effects:

- the structure blocks the flow, leading to increased flow velocity around the structure;
- the presence of the structure creates a local system of turbulent vortices which increases the local transport capacity.

The above nonlinear free-surface flow model is linked with a sediment transport and bed morphology evolution submodel. The submodel predicts the sediment transport (i.e., bed and suspended load) using an energy approach, which is based on the Bagnold's original idea that the sediment transport load is proportional to the time-averaged energy dissipation of the stream. In this approach the submerged weight transport rates, i_{xt} in the x-direction and i_{yt} in the y-direction, are given by Bailard (1981):

$$<i_{xt}>=<\left[\frac{\varepsilon_b}{\tan\phi}\left(\frac{u_o}{u_{ot}}+\frac{d_x}{\tan\phi}\right)\omega_b+\varepsilon_s\frac{u_{ot}}{w}\left(\frac{u_o}{u_{ot}}+\varepsilon_s\,d_x\frac{u_{ot}}{w}\right)\omega_b\right]>$$

$$<i_{yt}>=<\left[\frac{\varepsilon_b}{\tan\varphi}\left(\frac{v_o}{u_{ot}}+\frac{d_y}{\tan\phi}\right)\omega_b+\varepsilon_s\frac{u_{ot}}{w}\left(\frac{v_o}{u_{ot}}+\varepsilon_s\,d_y\frac{u_{ot}}{w}\right)\omega_b\right]> \qquad (7)$$

where u_o and v_o represent the near-bottom instantaneous velocity vector components; the angled brackets represent (numerical) time-averaging; d_x and d_y are the bed slopes; w is the sediment fall velocity; ϕ is the angle of internal friction; ε_b and ε_s are the bed and suspended load efficiency factors respectively; and ω_b is the local rate of energy dissipation,

$$\omega_b = f_w / 2u_{ot}^3 \tag{8}$$

where f_w is the friction factor and u_{ot} is the total near-bottom wave velocity $\left(u_{ot} = (u_o{}^2 + v_o{}^2)^{1/2}\right)$. According to the original Bagnold estimations (from river data) the bed and suspended load efficiency factors ε_b and ε_s take the values $\varepsilon_b = 0.13$ and $\varepsilon_s = 0.01$. The empirical coefficients are taken from general hydraulic engineering formulae.

The principle of concentration of sediment is applied in order to assess the bathymetry changes ζ_b in time:

$$\frac{\partial i_x}{\partial x} + \frac{\partial i_y}{\partial y} = \lambda \frac{\partial \zeta_b}{\partial y}, \ \lambda = (1-\varepsilon)(\rho_s/\rho - 1)\rho g \tag{9}$$

in which ε is the sediment porosity, and ρ and ρ_s are the densities of water and sediments.

In Figure 6 the bed changes (i.e., erosion and accretion) due to tsunami wave action around a building (configuration # 1) are shown. The tsunami wave height is H = 1 m, its period 100 s, while the duration of the event is 500 s. The mean sediment diameter is taken equal to 0.5 mm (d_{50} = 0.5 mm). The maximum scouring depth is of the order of 0.15 m and this occurred in an area of the order of 50 m^2 near the corners of the structure.

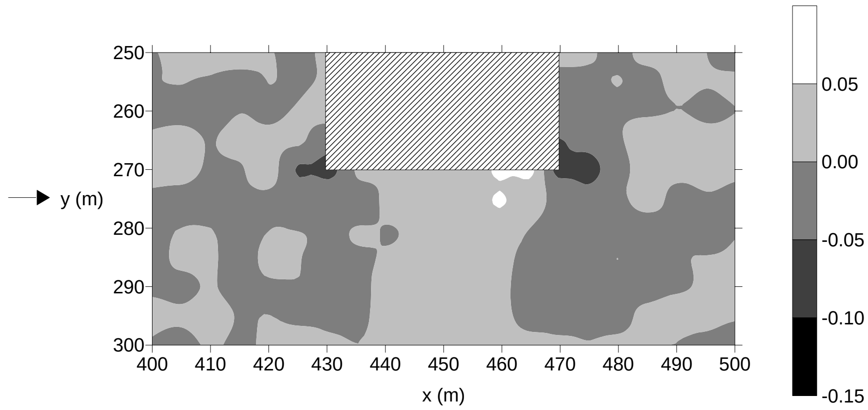


Fig. 6 Bathymetry changes ζ_b (scouring depth) around a building (configuration # 1, L/A = 35)

CONCLUSIONS

Operational models for the analysis and quantitative description of the hydrodynamic loading on buildings with complicated sectional geometry and the scouring or sedimentation around structures, under the attack of a nonlinear shallow wave like tsunami, are presented. The types of models and the general approach used here are not unique, but only indicative of the existing technological possibilities.

The non-calibrated applications aimed at the comparative study of the influence of the building geometry to the total dynamic loading. It is revealed that the building orientation with respect to the wave direction, and the partial or total collapse of the outside walls, may generate loading situations in favor or adverse to the building resistance to the tsunami.

Although the probability of tsunamis, even in the most tsunami-prone areas, is quite low, it is very important, in parallel to the tsunami warning systems, to provide in the design specifications of the coastal structures, like buildings, roads, bridges, harbours, etc., to the largest detail extent, the necessary measures in order to make the structures as much as possible “tsunami-resisting” or “tsunami-functioning”.

In the case of inadequacy of time for settlement evacuation, the consideration for designing “tsunami-functioning” structures becomes more important for the safety of coastal population.

Apart from the issues addressed in this work there are others, like the effectiveness of the building entrance and the staircases leading from the ground-floor to the first and second floor, which belong to the same class of issues and are open to further research. Especially when those provisions do not increase dramatically the cost of the structures, securing at the same time their best functioning in the event of tsunami, there is not justification for neglecting them.

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