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Liquefaction-induced large displacements of acaisson quay wall: Barcelona Harbor, Spain

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ABSTRACT

Large deformations induced by liquefaction are an important cause of instability in structures that interact with saturated granular soils. In this work the failure of a caisson type quay wall, occurred on January the 1st 2007 in Barcelona Harbor, is considered as case study. Due to the static liquefaction of the backfill soil, 16 of 37 caissons failed and were subjected to large displacements. The Material Point Method (MPM) is presented as a numerical tool capable of capturing the main aspects of this failure.

KEY WORDS: MPM; static liquefaction; caisson quay wall; material point method

INTRODUCTION

Liquefaction is a phenomenon through which a saturated or partially saturated soil loses strength and stiffness, changing from a solid to a nearly liquid state. That is due to the effective stress drop caused by an increase of pore water pressure. Over the years, several experimental and numerical methods have been developed for investigating the instability of supported structures, such as caisson type quay walls, when liquefaction of backfill and foundation soil is expected to occur. Most of them focused on the dynamic behavior of liquefiable soils during earthquake events (Inagaki et al., 1996; Kohama et al., 1998; Yang et al., 2001;Manzari et al., 2011).On the contrary, the numerical simulation of real cases of instability induced by static liquefaction is still challenging.

The caisson failure investigated in this study occurred in the Barcelona Harbor during the construction of a new container terminal. It involved the construction of a quay parallel to the coast, about 1000 m long, which was part of the terminal enclosure. The quay was made of reinforced concrete caissons, which were founded on a granular embankment. The enclosure filling was made with hydraulic fill dredged from areas adjacent to the quay, deposited through a rainbowing technique. This process made that material a highly heterogeneous mixture away from a normal sedimentary structure, therefore, difficult to characterize. However, it can be classified as very thin silts and sand with a very soft consistency. On January 1st 2007, during the filling operation, 16 of 37 caissons failed and experienced large displacements. Field observations and results of geotechnical and structural investigations, required by the Barcelona Port Authority, support the hypothesis that the failure was mainly caused by the static liquefaction of the backfill soil. Although the causes of triggering are still unclear, the dynamic force generated by the liquefied filling exceeded the resistance offered by the caissons weight and the friction at the contact with the underlying granular embankment, making the caissons slide on it. The failure mechanism did not involve the soil foundation nor the embankment on which caissons were founded which remained at the original location after the failure.

The main objective of this work is to ascertain if the MPM can capture the general characteristics of this case study and to provide a constitutive model able to reproduce the static liquefaction process.

METHODOLOGY

The Material Point Method (MPM) was developed to represent fluid dynamics by Harlow et al. (1964) and extended to soil mechanics problems by Sulsky et al. (1994, 1995). The method is intermediate between particle-based methods and finite element methods. The continuum media is described by a set of lagrangian materials points that can move with the material and a computational mesh that remains fixed through the calculation and covers the whole domain. Each point represents a portion of the domain and carry all the information of the material while the governing equations are solved at the nodes of the computational background. This double discretization can simulate large deformations without the problems associated with the distortion of the mesh elements, which are typical in conventional finite element methods.

In this study, plane strain analyses with the MPM 2-phases 1-point formulation (Zabala and Alonso, 2011;Jassim et al., 2013)are conducted on a representative section by using the *Anura3D* software(http://www.anura3d.com), developed by the MPM Research Community. First, a simple simulation in which the backfill soil is described with the Tresca model and the static liquefaction is imposed by reducing instantaneously it sun drained shear strength, is performed. Then, in order to simulate the loss of strength due to the increase of pore water pressure in the filling material, an undrained effective stress analysis is conducted by using the Ta-Ger constitutive model (Tasiopoulou and Gerolymos, 2016a, 2016b).

NUMERICAL MODEL

The geometry of the problem and the computational mesh, are given in Figure 1. Plane strain conditions are imposed by means of the boundary conditions restricting out-of-plane deformation and horizontal displacements along the vertical contours. The computational mesh is generated by using a thin 3D mesh of linear tetrahedral elements. The thickness of the model is considered to be the same as the element size of the mesh and has a value of 4 m. Initially, four material points are distributed within each element.

In order to calculate the reaction forces of the caisson structure at the contact surface with the backfill soil, it is necessary to track that surface during the simulation. To do that, the portion of mesh including the caisson is defined as “moving mesh” that translates in horizontal direction with the same average horizontal displacement of the material points discretizing the caisson, which is assumed rigid. The moving mesh feature discretizes the domain in a portion of the grid whose elements do not deform along the simulation but moves rigidly (moving mesh) and a portion that consequently modifies its dimensions with time (deforming mesh). In Figure 1, compressing, extending and moving meshes are indicated.

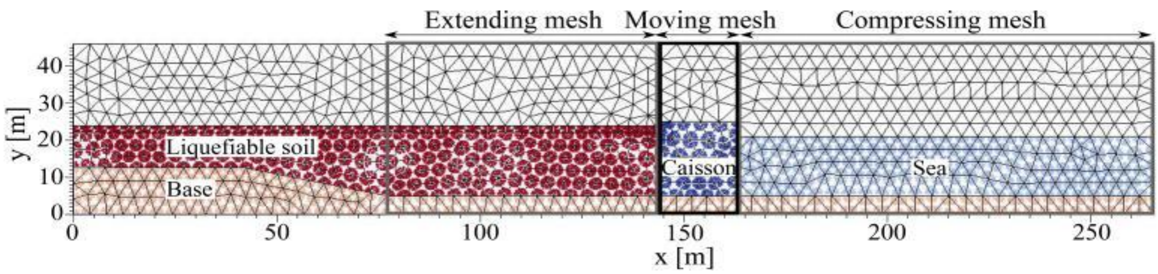

Figure1 MPM model of caisson quay wall failure including moving mesh.

The caisson structure is modeled as linear-elastic with a high Young’s Modulus. Since the failure mechanism did not affect the soil foundation nor the granular embankment on which caissons were located, they are both described as a unique linear-elastic material. The contact between the concrete caisson and the underlying base is defined with a friction coefficient$\mu = \tan\delta = 0.6$, where δ is the friction angleat the interface between the two materials. Regarding the characterization of the filling material, the only information available is related with the index properties and the undrained shear strength of the soil. Based on these data an initial porosity of $n = 0.45$ and a solid unit weight of $\gamma_s = 17\ \mathrm{kN/m^3}$ are considered for the simulations carried out with both constitutive models.

RESULTS

Undrained total stress analysis with Tresca model

An undrained total stress analysis by using the 2-phase 1-point MPM formulation is performed. The backfill soil is described by using the Tresca model with a value of undrained shear strength $c_u = 50$ kPa.The simulation is carried out in two phases. Stresses are initialized by applying a quasi-static gravity loading. A local damping factor of 0.75 is applied to reach a quasi-static equilibrium state in a faster way allowing a considerable reduction in the computational time. In the second phase, the static liquefaction of the backfill soil is imposed by instantaneously reducing to 0 its undrained shear strength (c_u). In this phase, the full dynamic behavior of the soil is analyzed and a homogeneous local damping factor of 0.05 is used. Moreover, in order to take into account the pore water pressure at the base of the caisson, which is not considered in the model, its weight is reduced to the submerged value (Equation 1).

$$W_{\text{submerged}} = \gamma_{\text{submerged}} \cdot A_{\text{caisson}} = \gamma_s \cdot A_{\text{caisson}} - \frac{(\gamma_{\text{sat}} \cdot h_{\text{backfill}} + \gamma_{\text{w}} \cdot h_{\text{sea}}) b_{\text{caisson}}}{2} \quad [\text{kN/m}] \tag{1}$$

Where A_{caisson} and b_{caisson} are respectively the area and the base of the caisson, h_{backfill} and h_{sea} are the height of the filling material and the sea at the caisson sides.

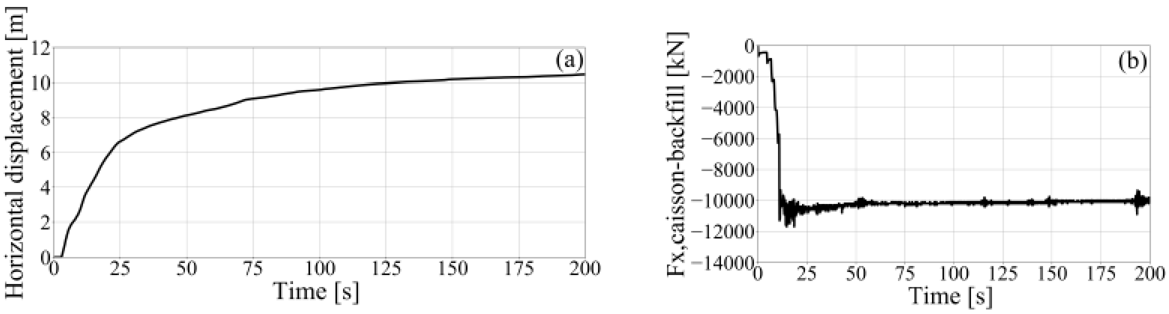

Figure2 (a) Horizontal displacement of caisson; (b) horizontal reaction forces of caisson at the contact with backfill soil

Figure 2(a) shows the horizontal displacement curve with time for the material point at the barycenter of the caisson structure. Figure 2(b) shows the sum of the horizontal reaction forces of the caisson at the contact nodes with the backfill soil. The caisson reaches a new stable position when the thrust of the backfill converge to a value of 9800kPa for which the equilibrium of the acting horizontal forces is satisfied (Equation 2). This is due to the progressive decrease of the backfill height behind the caisson from 19 m to 15.4 m, during the motion. In Figure 3 the final distribution of horizontal displacements is shown.

$$\left(W_{submerged} \cdot t_{model}\right) \cdot \mu + F_{x,sea\,side} = F_{x,backfill\,side} \qquad [\text{kN}] \tag{2}$$

Where t_{model} is the thickness of the numerical model, μ is the friction coefficient at the contact between the caisson and the granular embankment, $F_{x,sea\,side}$ and $F_{x,backfill\,side}$ are the sums of the caisson reaction forces at the sea side and at the backfill side.

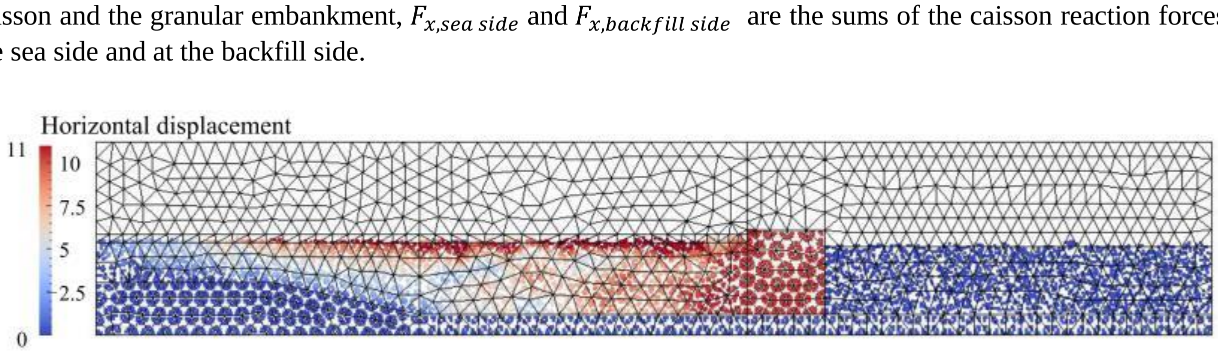

Figure 3 Horizontal displacements results at the end of the simulation

Undrained effective stress analysis with Ta-Ger

The Ta-Ger constitutive model was developed by Tasiopoulou and Gerolymos (2016a, 2016b). It is an elastoplastic model based on the critical state theory and developed with the aim of reproducing the behavior of a soil under different types of loading (monotonous, cyclical), drainage conditions and initial stresses, without the need to recalibrate its parameters. A material that exhibits a contractive behavior in undrained conditions is considered to model the liquefiable backfill soil. In order to characterize the material used in the analysis, undrained triaxial compression tests are simulated using the MPM Anura3D code for samples consolidated to25, 50 and 100 kPa (Figure 4). The model parameters are summarized in Table1.

Table 1TaGer model parameters.

Parameter	Symbol	Unit	Value
Shear modulus constant	G_0	-	1500
Shear modulus exponent	m	-	0.5
Poisson ratio	ν	-	0.2
Hardening exponent	n	-	0.6
Friction angle at critical state	φ_{cs}	°	30
Bounding surface	M_{s0}	-	1
	M_{sp}	-	1
Phase transformation surface	M_{pt0}	-	1.25
	c	-	10

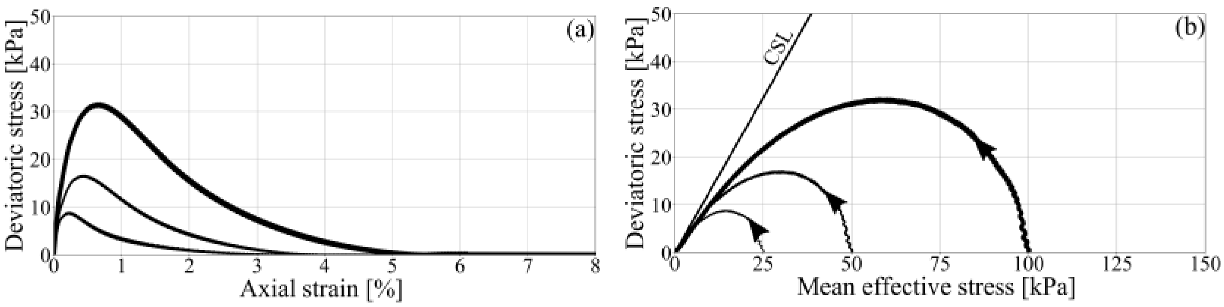

Figure4 Results of undrained triaxial tests at different confining pressures simulated with Anura3D. (a) Stress-axial strain relationship; (b) stress path in triaxial plane

One of the hypotheses that could explain the liquefaction triggering is that the hydraulic discharge of the last two meters of filling induced an overloading under undrained conditions on the underlying material. It was subjected to a sudden increasing of pore water pressure that induced a decrease of effective stresses. In order to reproduce this condition with Ta-Ger the solid unit weight of the upper two meters layer is increased during 10 seconds until the final value of 17 kN/m^3 for which liquefaction occurred.

Thus, the simulation is carried out in three phases: 1) stresses are initialized by quasi-static gravity loading; 2) the density of the upper backfill layer is increased until the backfill soil is liquefied and 3) the caisson weight is updated to its submerged value. During the dynamic calculation, a homogeneous local damping factor of 0.05 is applied.

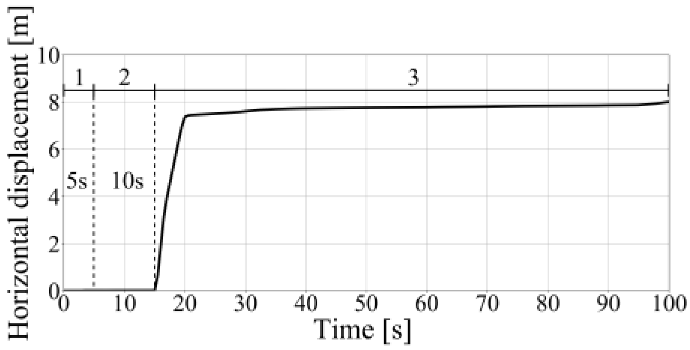

Figure 5 Horizontal displacement of a MP belonging to the caisson at the different phase of simulation

Figure 5 shows the horizontal displacement of the caisson with increasing time. The increase of the unit weight until fill liquefaction lasts 10 s. When the fill liquefies at this time, the caisson accelerates and moves forward 7.5 m in 5 seconds. Then the caisson velocity decreases to a small value until the end of the simulation period. The maximum horizontal displacement is lower compared with results of the previous analysis, probably because the liquefaction does not occur in the whole material at the same time. Moreover, during the propagation of the liquefaction process the fill material recover some strength until reaching a new stable configuration. In order to better visualize the liquefied areas at the end of the upper layer filling, the material points in which the mean effective stresses decreased more than 70% with respect to the initial value, is plotted in Figure 6.

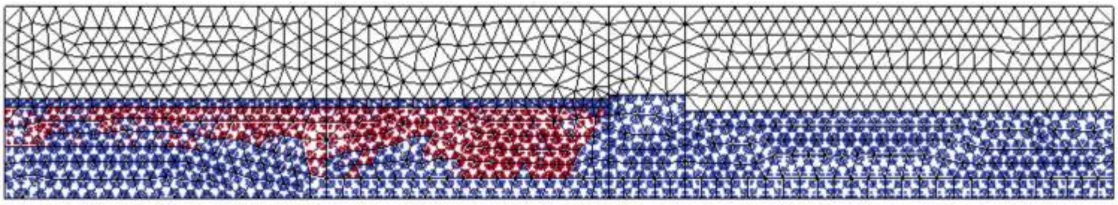

Figure 6 MPs where liquefaction occurred (in red colour)

CONCLUSIONS

Although the real sequence of events that led to failure are still unclear, the MPM is able to capture the general characteristic of the problem and the large displacements reached by the quay caisson. The hypothesis of the instantaneous liquefaction of the entire filling material is unrealistic but shows the capabilities of MPM to reproduce this instability problem. A better characterization of the backfill soil is necessary to deeper understand the evolution of the failure but Ta-Ger constitutive model shows a potential for the simulation of the liquefaction process.

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