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Seismic densification in Antofagasta Harbour, Chile

Densification due aux seismes au port D'Antofagasta, Chili

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ABSTRACT: Berth 7 in the Antofagasta harbour was built using caissons filled with crushed stones poured underwater without densification. This type of soil avoid liquefaction due to a very high permeability, but it was easy to densify by seismic strong motions. Due to the big magnitude of the predicted seismic settlements, a flexible pavement, easy to remove, was placed on the yards. The system behaved quite well during the Antofagasta earthquake of July 1995 with Ms = 7.3. Settlements between 65 ÷ 85 cm were measured, which were in accordance with the predicted settlement range.

RÉSUMÉ: Le quai 7 du port d'Antofagasta a été construit en utilisant des caissons remplis sous eaux avec des granulats concassés sans compactage. Ce type de sol ne présente pas de risques de liquéfaction à cause de sa haute perméabilité. Par contre, il est trés déformable face à de fortes sollicitations sismiques. C'est pour cette raison que des chaussées flexibles, de facile réparation, ont été prévues á cause des importants tassements sismiques calculés. Ce système a montré un comportement satisfaisant lors du séisme d'Antofagasta en juillet 1995 (Ms=7.3). Des tassements entre 65-85 cm ont été mesurés, en accord avec l'enveloppe de tassements calculée.

1 INTRODUCTION

The project to renew berth 7 at the Antofagasta port, Chile, included caissons filled and founded on a platform using a crushed andesitic rock 2" - 4" (angular rockfill) with a Los Angeles index of 11% to 13%. In order to reduce costs, the rockfill was placed underwater without compaction, excepting the rockfill over the water level which was compacted using vibratory rollers. Figure 1 shows the caissons and the rockfill geometry defined by the project.

Due to the very high permeability of the rockfill liquefaction was diregarded, but the lack of a compaction procedure impelled to estimate settlements due to seismic densification.

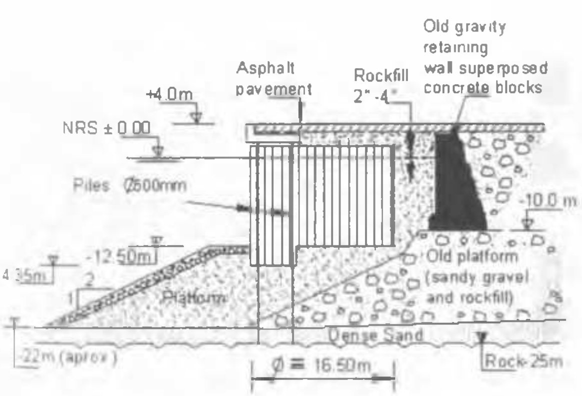

Figure 1. Caisson geometry according to the project.

2 ROCKFILL COMPACTNESS

Before construction, the 2" - 4" rockfill was placed underwater filling a reinforced container 1.9x3.6x1.1 m in height resting on the seabed. This test was performed three times by means of a flexible tube and helped by divers in order to obtain a uniform filling of the container. Once the container was lifted and placed carefully on the yard surface, the rockfill was levelled and weighted to obtain its dry weight. Besides, the maximum dry density, γ_{dmax}, and the minimum dry density, γ_{dmin}, γ_{dmin}, were obtained, taking into account the uniform rockfill gradation which needed carefull measurements.

For γ_{dmax} ten tests were performed by placing the rockfill in five layers into the reinforced container and compacting each layer by dropping the container 5 times from a height equal to 1.2 m, while for γ_{dmin} thirteen tests were performed by placing the rockfill very carefully into a 1 m^3 container. With these tests it was obtained $\gamma_{dmax} = 1.674 \pm 0.046$ t/m^3, $\gamma_{dmin} = 1.453 \pm 0.035$ t/m^3 and an average relative density DR = 30% for the rockfill placed underwater.

3 MECHANICAL PROPERTIES OF THE ROCKFILL

3.1 Shear modulus for seismic loads

The shear modulus for seismic (cyclic) loads, G_c, was obtained with cyclic triaxial tests applying a constant amplitud cyclic shear strain on samples 15 cm in diameter by 30 cm in height. The tests were performed using an angular soil ½" - 1" by crushing the original 2" - 4" rockfill. The cyclic shear modulus is expressed in (t/m^2) using the following relations:

$$G_c = 70\, K_2 \sqrt{\sigma'_c} \qquad (1)$$

$$\sigma'_c = \sigma'_{vo} \left[\frac{1+2Ko}{3} \right] \qquad (2)$$

where K_2 = cyclic shear coefficient; σ'_c = average confining stress (t/m^2) ; Ko = 1-sen ϕ_r is the at rest coefficient com-

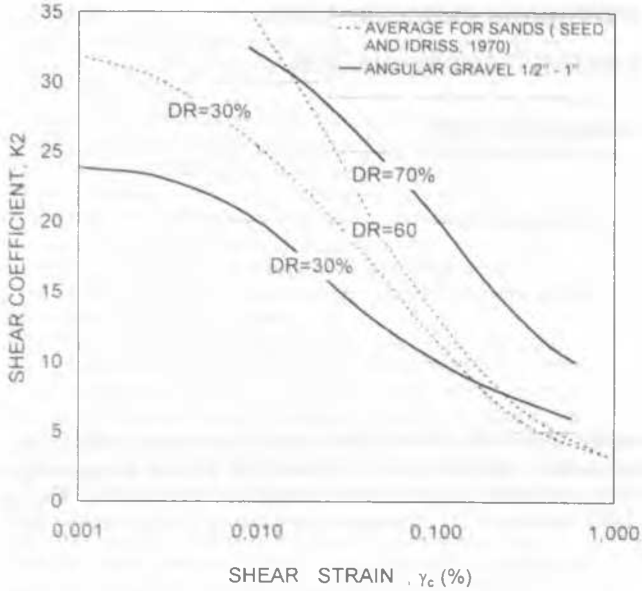

Figure 2. Cyclic shear coefficient as a function of the cyclic shear strain.

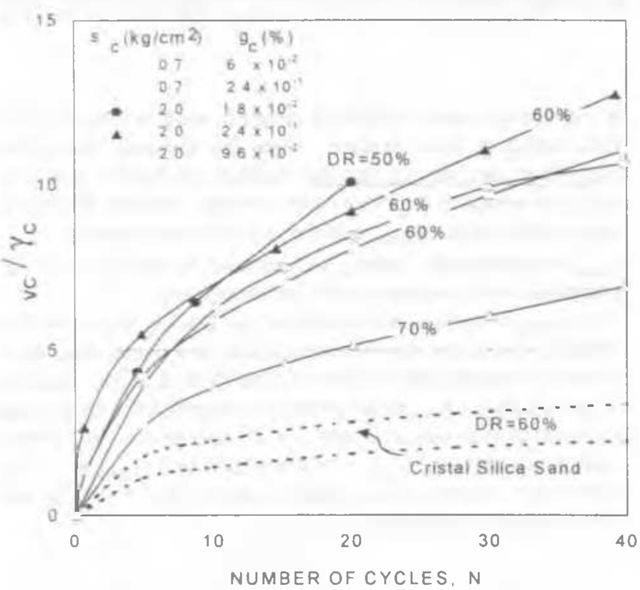

Figure 3. Densification function for the angular gravel 1/ 2" – 1"

puted using the angle of the natural slope, ϕ_r, equal to 36.5° for the rockfill (this angle was used becouse the low relative density of the rockfill placed underwater) ; and σ'_{vo} = vertical effective stress computed using a bouyant unit weight γ_b = 0.96 t/m^3 for the rockfill placed underwater and a total unit weight γ = 1.70 t/m^3 for the compacted rockfill above the water level. Figure 2 shows the variation K_2 vs. γ_c obtained with the cyclic triaxial tests using confining effective pressures in the range 0.5 kg/cm^2 to 2.0 kg/cm^2 to cover the in-situ confining stresses.

3.2 Densification

The rockfill densification was evaluated using cyclic triaxial tests applying a constant amplitude cyclic shear strain , γ_c , on saturated samples. The sample densification corresponds to the volume strain $\varepsilon_{vc} = \Delta V_c / V_o$, where ΔV_c is the sample volume reduction after N cycles, measured by the expelled water, and V_o is the initial volume of the sample, while the densification function is expressed as $\varepsilon_{vc}/\gamma_c$ (Silver and Seed, 1971). Figure 3 illustrates the relation $\varepsilon_{vc}/\gamma_c$ vs. N for the angular rockfill ½" - 1", showing a main dependence on the relative density and a minor dependence on the shear strain and the confining pressure.

The densification function $\varepsilon_{vc}/\gamma_c$ is the main parameter when computing seismic settlements, so it was necessary to test the 2" - 4" rockfill to obtain that function. The lack of a cyclic triaxial apparatus big enough to test the 2" - 4" rockfill, was replaced by testing angular soils by crushing the original andesitic rock ending with finer scaled gradations. Those tests were complemented with the densification function for the Cristal Silica sand reported by Silver and Seed (1971). Figure 4 shows the densification function after N = 20 cycles as a

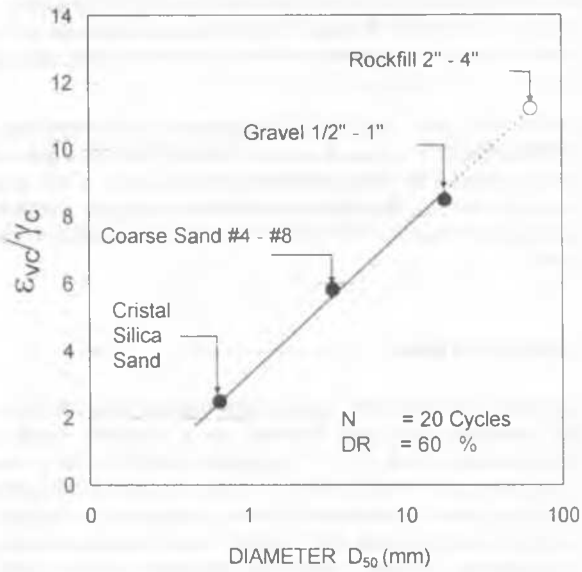

Figure 4. Densification function vs diameter D_{50}.

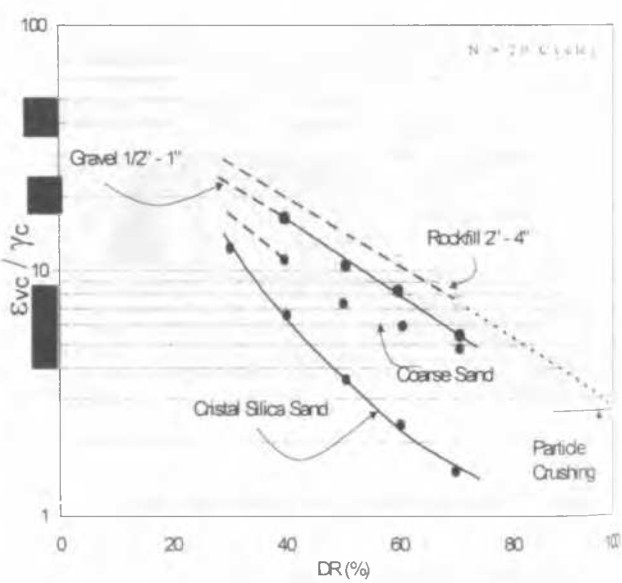

Figure 5. Densification function for different uniform soils with angular particles.

function of the main soil diameter, D_{50}. This figure enabled to obtain by extrapolation the densification function for the 2" - 4" rockfill.

Figure 5 shows the variation of the densification function with the relative density for the different tested soils. The dotted line in that figure corresponds to the 2" - 4" rockfill which was obtained by extrapolation.

4 SEISMIC SETTLEMENT

The seismic settlement, ρ_s, due to the soil compaction between depths z_1 and z_2 measured from the yard surface was obtained using the following equations:

$$\rho_S = C_1\ C_2 \int_{z_1}^{z_2} \left[\frac{\varepsilon_{vc}}{\gamma_c} \right] \gamma_c\, dz \qquad (3)$$

$$C_2 = 2\sqrt{(1+p^2)} \qquad (4)$$

According to Seed and Tokimatsu (1984), C_1 = 0.65 to transform the induced earthquake maximum shear strain amplitude, γ_c, into an equivalent constant amplitude acting in N significative cycles, where N depends on the earthquake magnitud, but normally varies between 20 to 30 cycles.

Equation (4) was obtained from tests reported by Pyke et al (1974). It was introduced to take into account the simoultaneous action of the earthquake horizontal components, where p is the maximum acceleration of the minor horizontal component divided by the maximum acceleration of the main horizontal component of the earthquake record.

The maximum shear strain induced by the earthquake, γ_c, was computed using a simplified method reported by Ortigosa and Musante (1991):

$$\gamma_c = \frac{r_d \cdot a_{max} \cdot \gamma \cdot z}{G_c} \qquad (5)$$

$$r_d = 1 - 0.167\ z \qquad (6)$$

$$a_{max} = A \cdot a_r \qquad (7)$$

where z = depth measured from the yard surface; G_c = cyclic shear modulus of the soil; r_d = reduction coefficient given by Seed and Idriss (1971); γ = rockfill total unit weight below the water level computed with 25% of the water mass moving in phase with the rockfill; a_{max} = maximum horizontal acceleration in the seabed; a_r = maximum horizontal acceleration in the bedrock which was taken equal to 0.25 g; and A = acceleration amplification factor between the bedrock and the seabed. An iterative computation was needed becouse γ_c depends on G_c and G_c depends on γ_c through the cyclic shear coefficient K_2.

The amplification factor, A, was obtained from Figure 6 which includes data from the Valparaiso port, located 1400 km south of the Antofagasta port, for the earthquake of march 1985 with magnitude M_s = 7.8 (Ortigosa et al 1993). As a reference, the amplification factors for the Antofagasta earthquake of July 1995 (M_s = 7.3) are included. However, these amplification factors were not used for the seismic settlement prediction at berth 7, because that prediction was performed before the Antofagasta earthquake of July 1995.

In both cases, Valparaiso and Antofagasta ports, the maximum acceleration at the seabed was obtained matching the permanent seismic horizontal displacements due to sliding between concrete blocks of the gravity retaining walls.

5 RESULTS

Seismic settlements were computed through eqs. (3) to (7) using $\varepsilon_{vc} / \gamma_c$ = 28 obtained from Figure 5 for DR = 30% and N = 20 cycles, a factor p = 0.60 based on earthquake records at the central area of Chile and an amplification factor A ≈ 1.75 obtained from Figure 6 for a rock depth equal to 29 m. Values of K_2 vs γ_c to compute the cyclic shear modulus, G_c, were obtained from Figure 2 using results for the ½" - 1" rockfill with DR = 30%.

The analysis was applied to four sections of berth 7, where the seismic settlements were computed between the caissons and the existing gravity retaining wall. An average settlement of 80 cm was obtained with a variation of ± 3 cm due to different rockfill thickness, while a caisson tilting of 2.2° ± 0.9° was computed due to the different rockfill thickness in the platform. Those results forced to modify the original project driving the caissons front sheetpiles down to the bed rock in order to minimize tilting. Besides, a yard pavement with precast concrete blocks, easy to repair, was built.

During the Antofagasta earthquake of July 1995 seismic settlements of 75 ± 10 cm were measured which validated the project modifications.

6 CONCLUSIONS

1. For a given compactness, the seismic settlement for soils with uniform gradation and angular particles increases linearly with the log D_{50} diameter, which is attributed to particle crushing.
2. Supporting the previous conclusion, recent tests performed on ½" - 1" soils with rounded particles exhibits a densification of the order of 25% of the densification measured for the same soil with angular particles.

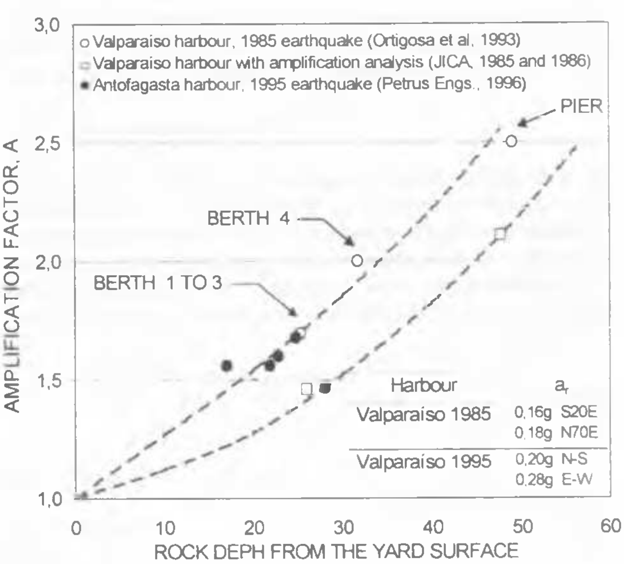

Figure 6. Amplification factors for maximum acceleration in the seabed.

3. The densification function obtained with cyclic triaxial tests applying a shear strain of constant amplitude and the equations for predicting seismic settlements, proved to be adequate at berth 7 of the Antofagasta port.

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