Jet Grouting applications for the soil reinforcement under caisson quay walls and mitigation of seismic effects in Gibraltar.

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Abstract. A case history of Jet Grouting technique is presented for the development of port facilities in Gibraltar. The main objective was to strengthen the soil below the foundations of a series of port caissons to improve their general stability under seismic event. Also, it is described the execution of a watertight enclosure and a bottom sealing with Jet Grouting to carry out the excavation below the water table in an area of facilities located in the back of the caissons. The case presented highlights the effectiveness of jet grouting and its control systems for complex port works.

Keywords: Jet Grouting, liquefaction, sealing slab, impermeability, excavation pit support.

1. Introduction

A new power plant is foreseen on the North Mole in the Port of Gibraltar. In order to supply this plant it is necessary the development of a LNG offloading and storage facilities. An existing 118.5 m quay structure at the end of this Mole will be modified into a LNG offloading berth for LNG vessels. The LNG storage and regasification units will be located on the immediate area behind the quay structure and alongside the proposed power station.

In order to comply with the construction, operating and seismic requirements of these facilities, it is necessary to reinforce the foundations of the existing port caissons and part of the new projected installations. In addition, for one of these installations, an impounding basin must be executed under the water level so it is mandatory to guarantee the water tightness of the excavated area.

Jet Grouting is a very versatile method that allows to become the nature soil into a soil-cement type material, usually denominated as Soilcrete®, which is also able to reach such a reduced permeability that enable excavations below water table.

In addition, the Jetgrouting elements can be applied in localized areas through the formation of different shapes and at different depths. All of this made this technique the optimal option to perform the work described above.

Figure 1 shows a general overview of the project and the location of the relevant facilities.

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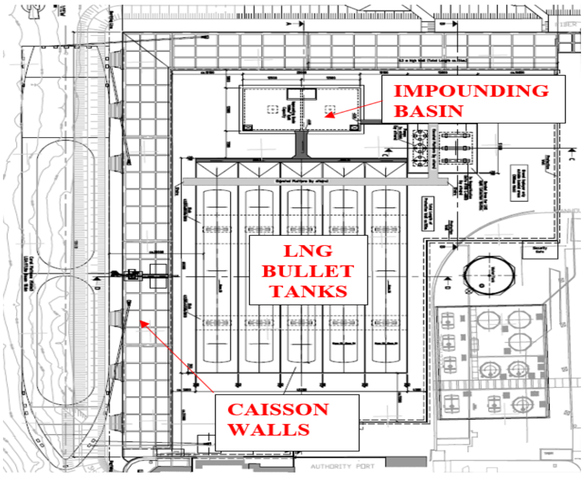


Figure 1. General overview of the project and location of the relevant facilities.

2. Geotechnical characterization and liquefaction potential

2.1. Geotechnical profiles

Two models of soil profiles were considered so as to correctly define the stratigraphy in both seaside and landside areas as shown in Figure 2.

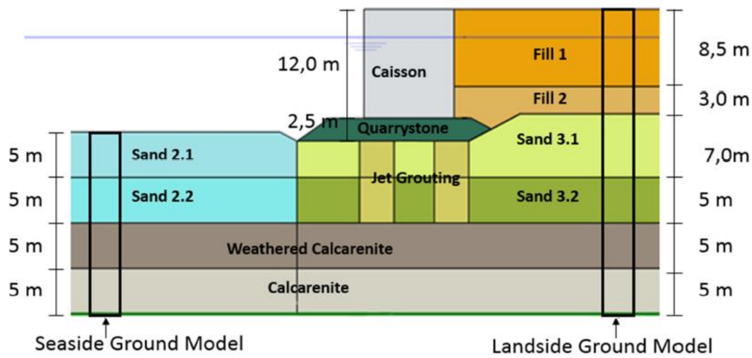


Figure 2. Ground stratigraphies.

The landside model is representative of the reclaimed area behind the caisson wall and consists of a sequence of fill, natural sand layers and calcarenite. On the other hand, the seaside model depicts the fully submerged natural soils consisting of natural sand layers over the calcarenite stratum. In Table 1 all the representative parameters of the soil layers are gathered.

Table 1. Compilation of the constitutive parameters of the soil layers

Layer	FC %	USCS	N_{spt}	$(N_1)_{60\_cs}$	V_s m/s	ϕ' °	c' kPa	γ kN/m^3	γ_{sat} kN/m^3	D_r %
Fill 1	12	SC-SM	37	42	175	35	0	20	21	0.96
Fill 2	10	SC-SM	35	39	225	35	0	20	21	0.92
Sand 1 seaside	10	SP	12	16	150	30	0	19	20	0.59
Sand 2 seaside	12	SC	25-R	31	215	32	0	19	20	0.82
Sand 1 landside	5	SP-SM	35	40	250	30	0.6	19	20	0.93
Sand 2 landside	10	SC	30	36	275	29	0.25	19	20	0.88
Weathered calcarenite	10	-	30	36	-	40	0	21	22	0.88
Calcarenite	-	-	-	-	-	40	450	21	22	-

2.2. Analysis of liquefaction

Free-field liquefaction potential for the LNG facilities site was evaluated considering a return periods of 475, 2475 and 4975 years. The liquefaction assessment was based on the methodology of Boulanger and Idriss (2014), a stress-based approach pioneered by H. B. Seed and I. M. Idriss in the late 1960s. [1]

According to the seismic hazard assessment provided, the characteristics of the earthquake for the return period of 4975 years are a Magnitude of 6.5 and a PGA of 0.28g. The results of the liquefaction analysis for both seaside and landside areas are shown in Figure 3.

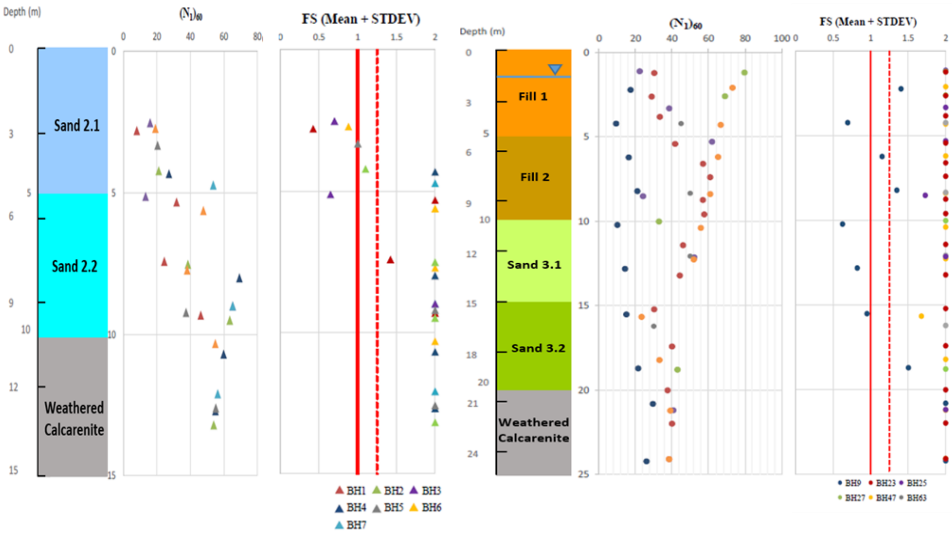


Figure 3. Seaside and landside liquefaction analysis.

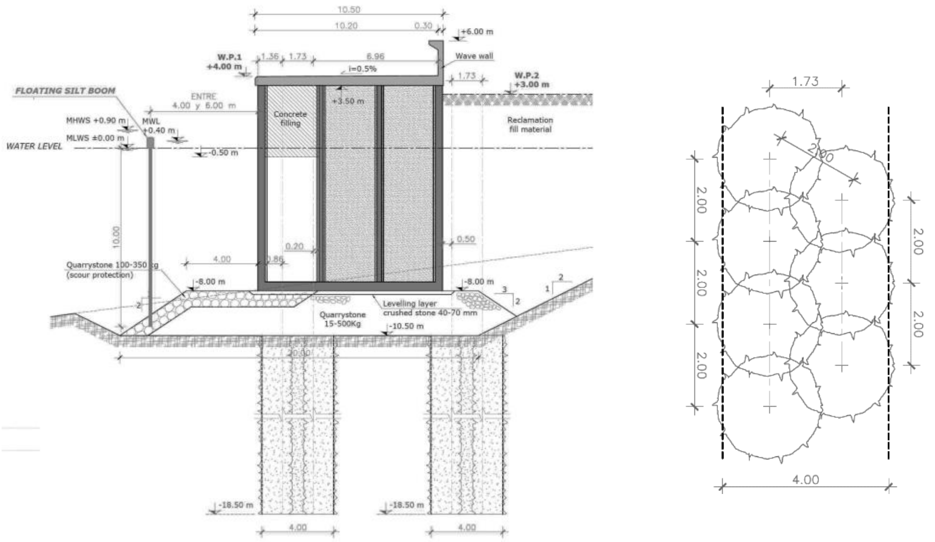
As can be seen in Figure 3, there are some isolated pockets of potentially liquefiable fill material (Fill 1 and Fill 2) and the natural sand layers (Sand 3.1 and Sand 3.2) in the landside area. Regarding the seaside area, the results indicate that the natural sand stratum along the vicinity of the caisson are liquefiable.

3. Description of the Jet Grouting Solution

3.1. Reinforcement under the caisson walls

The purpose of the geotechnical works to be done is to reinforce the soil under the caisson wall in order to minimize the lateral displacements due to soil liquefaction produced during the ground shaking. The fact that the caisson wall is already built limits the possible ground improvement technics to be applied in the jobsite, especially under the rockfill, thus in this case jet grouting is recommended as a less intrusive procedure.

In order to mitigate the liquefaction effects that may occur in the sandy layers beneath the caisson wall and thus provide stability to the structure and the reclaimed fill, two jet grouting walls under the toe and heel of the caisson have been proposed as seen in Figure 4.



(a) (b)

Figure 4. (a) Cross section of the jet grouting improvement solution. (b) Proposed Jet Grouting column diameter (2,50 m) and distribution to guaranty effective wall thickness of 4,0 m.

The reinforcement works under the caisson wall includes the execution of 200 jet columns, in two parallel rows, along 100 linear meters of structure. As can be seen in Figures 4a and 4b, each row of jet grouting wall has an effective thickness of 4.00 m and consists of two lines of overlapped jet grouting columns with a diameter of 2.50 m per column in a triangular arrangement. The depth of the columns reaches the calcarenite layer at 22.5 m and their effective length is 8.5 m. To reach the starting point of the jet columns it is necessary to pre-drill the 12.0 m of the body of the caisson and quarry stone.

This solution comprises an improvement of soil conditions in the treated area. The high modulus of the jet grouting columns has a significant impact on the mitigation of liquefaction as the jet grout columns and the surrounding soil jointly withstand the shear forces after an earthquake keeping the structure stable and achieving a local and global stability. A general description of jet grouting method can be seen in [4].

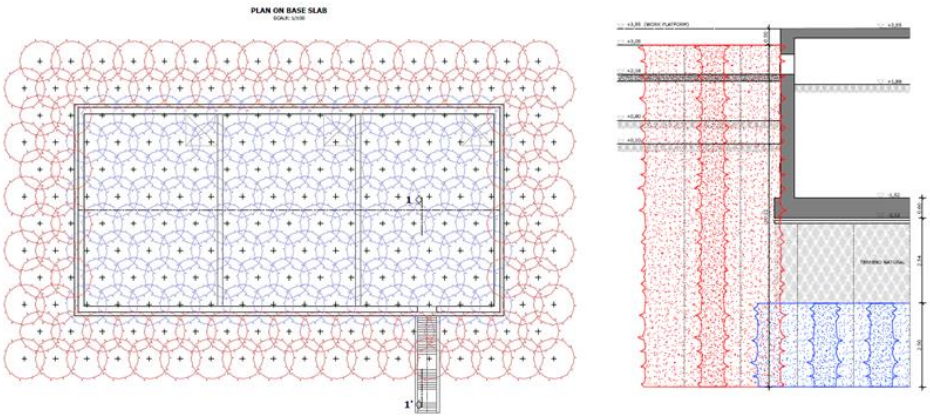
3.2. Excavation for the impounding basin emplacement

The required excavation to build this structure is carried out under the water table. Given this situation, it becomes necessary to ensure the watertightness of the enclosed area to carry out the works in a safe environment.

As shown in Figures 5a and 5b, the adopted solution consists of a perimeter wall of secant columns of jet grouting that provides a temporary retain system as well as a barrier against the phreatic water. This perimeter wall consists of 50 jet columns of 3.0 m diameter. The length of these columns extends from surface level to about 10.0 m depth.

Subsequently, a sealing slab is built at the bottom in order to prevent water from entering the excavation. This solution prevents the flooding of the work area during the construction and the lifting of the ground and basin due to the water subpressure once it is built. This solution comprises a total of 112 columns with a diameter of 2.5 m. Their length is defined by the uplift resistance of the sealing slab body. As can be seen in Figure 5b, after the appropriate verifications, it is not necessary to take the jet column up to the foundation level of the basin. Instead, the weight of the ground below is used to compensate the uplift and reduce their length.

The overlap between the columns of the sealing slab and the perimeter columns must be appropriate to guarantee dry work conditions in the excavated area. It is therefore essential to have a correct control of the verticality during these columns execution.



(a) (b)

Figure 5. (a) Top view and (b) cross section of the impounding basin solution for water-tightness.

3.3. Reinforcement under the LNG Bullet Tanks

In order to improve the foundation of the LNG Tanks for these facilities a local design of jet grouting column is designed under each one.

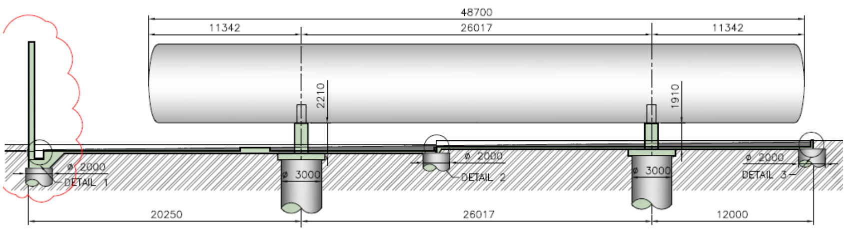


Figure 6. Cross section of the jet grouting reinforcement for the LNG bullet tanks.

For a total of 5 tanks, 10 jet columns with a diameter of 3.0 m have been proposed as the reinforcement under the foundation and 22 columns with a diameter of 2.0 m under the slab. The total length of the columns reaches 22.5 m depth.

4. Evaluation of the solution with FEM

In order to simulate the behavior of the Caisson Wall considering the liquefaction of the soil during the seismic event a Plaxis finite element model was carried out. Figure 2 shows the model considered. The dynamic effects of the earthquake were modelled by the insertion of the accelerograms in the dynamic module of the software.

The constitutive soil models used for the calculation were Mohr-Coulomb and Hardening Soil Model for the soils and Linear Elastic Model for the Caisson Wall. The damping model used was the one proposed by Rayleigh [2],[3].

In Table 2 the soil models and its constitutive parameters are shown:

Table 2. Soil parameters for the FEM model.

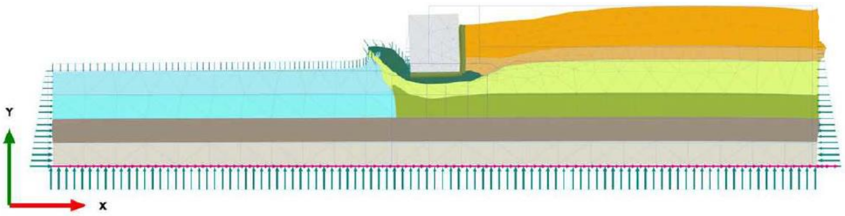
Layer	Material Model	E_{50} MPa	E_{oed} MPa	E_{ur} MPa	E MPa	ν	ϕ' °	c' kPa	γ_{sat} kN/m^3
Fill 1	HS	35	35	105	-	-	35	0	21
Fill 2	HS	35	35	105	-	-	35	0	21
Sand 1 seaside	HS	15	15	45	-	-	30	0	20
Sand 2 seaside	HS	25	25	75	-	-	32	0	20
Sand 1 landside	HS	35	35	105	-	-	30	0.6	20
Sand 2 landside	HS	30	30	90	-	-	29	0.25	20
Weathered calcarenite	HS	90	90	270	-	-	40	0	22
Quarrystone	HS	80	80	240	-		41	0	21
Calcarenite	MC	-	-	-	-	0.3	40	450	22
Jet Grouting	MC	-	-	-	150000	0.25	35	900	19
Caisson	LE	-	-	-	250000	0.2	-	-	22

Liquefaction conditions was modelled by adopting the strength and stiffness residual parameters of the affected materials.

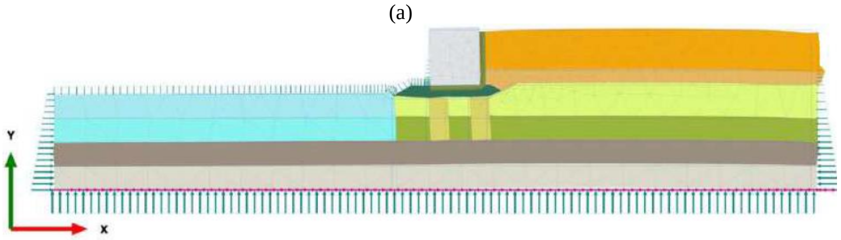
Figure 7a shows the deformed mesh of FEM model with the displacements suffered by the caisson wall both by the dynamic effects of the earthquake and by the effects of liquefaction in the event of not carrying out any improvement in the foundation. On the other hand, Figure 7b shows the deformation experienced in the case of performing the reinforcement of jet grouting under the structure.

In quantitative terms, the expected deformation for each case of study is a permanent lateral displacement of about 1.50 m in the unimproved situation and about 0.25 m in the case of the reinforced foundation with jet grouting.

These values meet the deformation requirements for the caisson wall after the earthquake: A permanent displacement less than 60 cm and a tilt angle less than 5º.



(a)



(b)

Figures 7. (a) Cross section of the Caisson walls foundation without jet grouting, (b) Cross section of the Caisson walls foundation with jet grouting.

A very noticeable evidence of the improvement introduced by the jet grouting reinforcement under the foundation is shown in Figure 8. It illustrates the relative shear strength mobilized during the dynamic stresses and how the jet columns are able to resist these effects without overcoming their shear resistance. In this way an overall satability of the whole caisson wall is achieved.

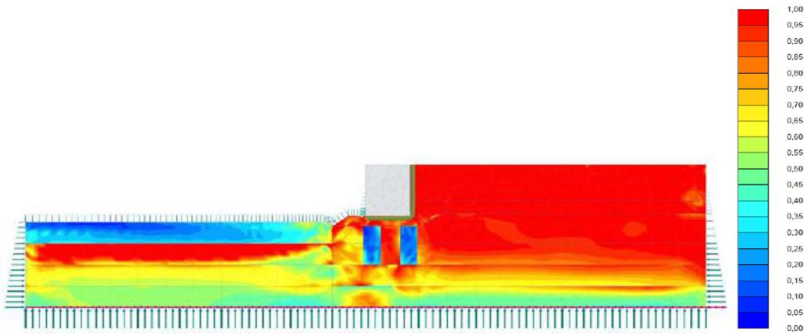


Figure 8. Mobilized shear resistance of the soils.

5. Particularities of the works development

Due to operational issues and project schedules it was decided to implement the proposed solutions only for the reinforcement of the foundation for both the caisson wall and the LNG tanks. These works were carried out in several stages and lasted a total time of 15 weeks.

Design tests using the ACI (Acoustic Column Inspector), as described in [5], were performed to define the execution parameters to achieve the required diameter and length.

The challenges faced were mainly to achieve the large diameter of jet grouting columns required for some foundations, as well as the pre-drilling works in the caisson wall to reach the foundation level, especially regarding the quarrystone due to its strength and disposition.

Another aspect of special relevance was the need of ensuring the verticality of the drillings to perform the jet grouting columns in order to obtain a proper overlap. This was successfully achieved by getting deviations in the range of 1%.

6. Conclusions

Jet Grouting has proven to be a very effective and versatile technique for the treatment and improvement of low resistance soils in areas with seismic risk. It achieves the reduction of deformations to acceptable levels and constitutes an optimal solution to mitigate the dynamic and liquefaction effects induced by the earthquake. According to the calculation performed, the expected total displacement of the caisson is 0.25 m and meets the requirements of the project.

Likewise, it has been demonstrated its effectiveness to carry out localized works at different depths and in inaccessible areas. In addition, and depending on the situation, it not only allows the proper support of excavation pits but also turns to be an optimum system to guarantee great reduction on the treated soil permeability, achieving both watertight enclosures and vertical hydraulic barriers.

A major issue in this works is to ensure the verticality of the jet grouting columns especially at such depths as minimal deviations in surface mean large deviations at the column toe. This is achieved with an adequate control system of the actual deviations during the works and it was verified during the process that the vertical deviations in this project was no greater than 1%.

7. References

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