Alternative foundation for the constructionof tetrapod structuresin the construction yard of Cobos maritime terminal at Tuxpan, Veracruz

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ABSTRACT
This paper describes alternative foundation for the smooth sliding of 1200-tons tetrapod steel structures in the construction yard of Cobos maritime terminal, without the supporting soil having sustained total failure due to stress shear or otherwise excessive deformations. It provides a general overview of unconventional projects whose solution is far from representing very elaborate and expensive alternatives; on the contrary, it was sought to return to the basics of geotechnical engineering.

RESUMEN
Este trabajo incluye la alternativa de cimentación para el correcto deslizamiento de tetrápodos de 1200 t en el patio de construcción de la Terminal Marítima de Cobos, sin que el suelo de apoyo haya sufrido falla total debido al esfuerzo cortante o en su defecto deformaciones excesivas. Se da una perspectiva general de proyectos poco convencionales, cuya solución no representa alternativas muy elaboradas y costosas, al contrario busco regresar a los principios básicos de la ingeniería geotécnica.

1 INTRODUCTION

In civil engineering works the designer has to confront a series of problems to be resolved. From the most complex to those that could be considered as simple, mention should be made that all of them pose a certain degree of complexity. Along these lines, projects exist that are no daily routine such as constructions based on shallow and/or deep foundations, excavations, retaining elements, etc.; these “uncommon” projects are dealt with in this paper.

When marine geotechnical studies are performed, it is intended to collect information from the subsoil to be able to build a foundation for offshore platforms, being this the second stage necessary for commissioning of the project. The first stage involves the construction of all structures constituting such offshore platform. As part of them we can find secondary steel structures such as tripods, tetrapods, hexapods and octapods that are made in manufacturing yards located at the wharf zone.

For such purpose it is necessary to have a competent supporting soil for the yard because of the heavy loads transmitted by these elements and it is therefore necessary to carry out a soil mechanics study at the area where such structures will be built for the purpose of guaranteeing at all times the availability of said substructures at their offshore location. It is therefore necessary to seek solutions that can be adapted to this type of projects, in what refers to safety, cost, and most importantly, time.

1.1 Background

The function of most offshore platforms is to extract crude oil and natural gas that invariably come together. In some well the fluid predominates, but always with a certain percentage of gas; in others, the opposite combination is available. This geologic characteristic makes it necessary to separate in offshore facilities both types of hydrocarbons for subsequent pumping inland, because the final destination for both is perfectly differentiated: the gas is concentrated at the repumping plant of Atasta, Campeche whereas the crude oil is conveyed to the port of Dos Bocas in the state of Tabasco, built for this purpose.

The Campeche Sound concentrates the largest offshore platforms ever built in Mexico, in both production rate and size. However, the construction of the large offshore platforms is carried out in other sites such as Tuxpan, Veracruz and Ciudad Madero, Tamaulipas.

1.2 Objectives

Provide a foundation alternative for the smooth sliding of 1200-tons tetrapod steel structures at the construction yard of the Cobos maritime terminal, with no sudden failure occurring in the supporting soil or otherwise no important deformations are experienced so as to prevent the structures from being handled, hauled and shipped for sea transportation.

1.3 Scope

Provide a simple, low-cost and quick solution to problems that usually represent a high complexity because projects of this type are not so usual. It has been sought to go back to basic engineering that is sometimes set aside in the search for more sophisticated and more costly solutions.

2 SURVEILLANCE OF THE SITE

2.1 Location

The site under study is located at the municipality of Tuxpan, a port lying at the northern part of the Huasteca Region in the state of Vercacruz; its coordinates are 20° 57´ of latitude north 97° 24´ of longitude west. It neighbors to the north with Tamiahua and Naranjos, to the west with Temapache, and to the south with Tihuatlán and Poza Rica (Figure 1).

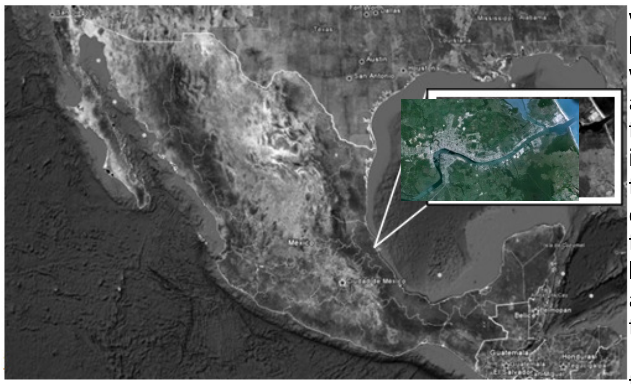

Figure 1. Location of Tuxpan, Veracruz.

2.2 Geology

The zone being studied corresponds to the Tampico-Misantla Basin (López R, 1979) with outcroppings from the marine Cenozoic (Figure 2).

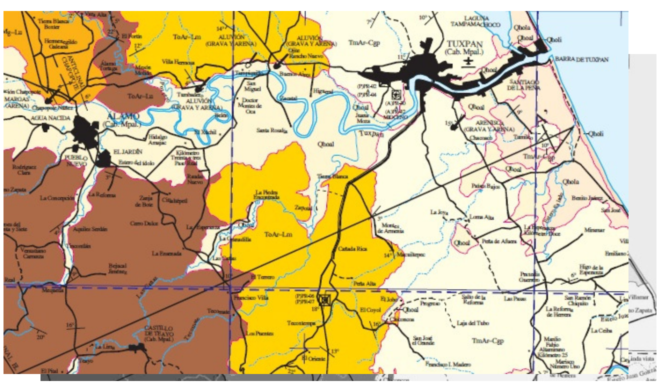

Figure 2. Regional geology. *Servicio Geológico Mexicano*.

This basin lies at the Coastal Plain of the Gulf of Mexico, bounded to the north by the sierra of Tamaulipas, to the south by the massif of Teziutlán, to the east by the Gulf of Mexico and to the west by the Eastern Sierra Madre (Raísz E, 1964). The stratigraphic sequence outcropping in this region is constituted by marine sediments of the Jurassic, Cretaceous and Tertiary. The oldest units outcrop toward the southwestern part, whereas the most recent are located to the east of the unit.

The sedimentary units typical of the zone being studied belong to the formation Tuxpan TmAr-Cgp, varying the stratigraphy from sandstones, clay-shales and siltstones that are partially overlain by alluvial and lacustrine deposits (clays and silts); as a result, mention can be made that their contacts are discordant because of their surrounding environment.

2.3 References of the site

According to the background of the site itself, information was received that structures had been previously built but none of them with the dimensions and weight that were considered for their intended construction. This aspect is important and the details should be always taken into account by the designer because they provide information about any preloading of the subsoil and therefore, regardless of the fact that fine soils are encountered such as those indicated in the geological table and that because of their nature are located at the banks of the river, they are not indicative that settlements are likely to occur such as those commonly detected in this type of deposits.

In addition, mention should be made that when the tetrapod structure is hauled along the construction yard it has to cross the wharf that is supported by pipe piles and that, in due time, have to be analyzed too.

3 DEVELOPMENT OF THE SOIL MECHANICS STUDY

3.1 Stratigraphic description of the site

In general terms the type of soil explored was as follows:

- 0.00-1.00 m. Fill material constituted principally by boulders, gravel, sand and more than 30% of fines (clays).
- 1.00-2.80 m Brown greenish sandy clay with soft consistency
- 2.80-3.40 m. Fine sand with very loose relative density with lenses of greenish brown clay lenses. The blow count from the SPT test was of about 3.
- 3.40-6.60 m. Highly plastic clay with clay lenses and bits of greenish gray shells
- 6.60-11.60 m. Highly plastic clay with organic matter (peat), greenish gray or dark brown in color, with very soft consistency.
- 11.60-14.80 m. Greenish gray clay with fine sand and very soft consistency.
- 14.80-16.80 m. Fine sand with very loose to loose relative density interspersed with greenish gray clay lenses.
- 16.80-19.80 m. Greenish gray clay with lenses of fine sand.

According to this stratigraphy, it is considered that there exists a soil constituted principally by fine soils, although mention should be made that there are actually two pervious sand strata through which the pore water pressure would dissipate at the time of their eventual consolidation.

Among the laboratory test results obtain special mention should be made of the values of the natural water content. Along the whole stratigraphic column, except at a depth from 8.50 to 10.40 m, water contents smaller than 50% were obtained, whereas at this separate section the values of the water content exceeded 100% with a peak value of 200%. This result is relevant because it indicates that if the soil has no high values of water content there would be no high pore-water pressure to dissipate.

4 FOUNDATION ALTERNATIVES

4.1 Initial alternatives proposed

The first option considered implied the use of strip footings; however, in the first instance there appeared the uncertainty about their actual behavior, i.e. how their rigidity could be guaranteed at the time of supporting the transient load transmitted by the tetrapod weight. The corresponding calculation were made using the general formula to determine the load bearing capacity proposed by Terzaghi and applying the coefficients of Vesic obtained in 1970. Due to the uncertainty in the actual behavior of the strip footings and to the magnitude of project, a parameter of 3 was used as safety factor. The analyses proved that the admissible load bearing capacity was not sufficient to stand the pressure transmitted to the soil and therefore there was the possibility of experiencing a "sudden" failure in the ground.

It is also worth mentioning that for the purpose of not worsening the difficulties expected in such long term, particularly during construction, the strip footings could not be deeper than two meters, because of the phreatic water level below.

Immediate settlements became equal to 7 cm and this value could hinder the dragging of the tetrapod into the barge On the other hand, long-term settlements became of the order of 12 cm.

Because of these results another alternative was reviewed of using drilled shat foundations cast *in situ* with a diameter of about 60 cm, distributed in the full length of the skids, separated a distance of approximately 15 meters. This solution demanded the possibility of having available sufficient load bearing capacity to withstand the loads transmitted by the tetrapod structure and at the same time preventing large settlements that became equal to about 4 cm.

The fact of using drilled shaft foundations was a matter of time since there was no time available because of the priority of placing the tetrapod structure offshore and also due to the rent of the barge that was coming to tow, load and carry the structure to the location where it was going to be placed. In addition to time another factor came into being that we as engineers should seriously take into account, namely the cost.

4.2 Alternative using a rockfill embankment

After these alternatives were analyzed in a general way an immediate solution was required to be built in a short time and at low cost.

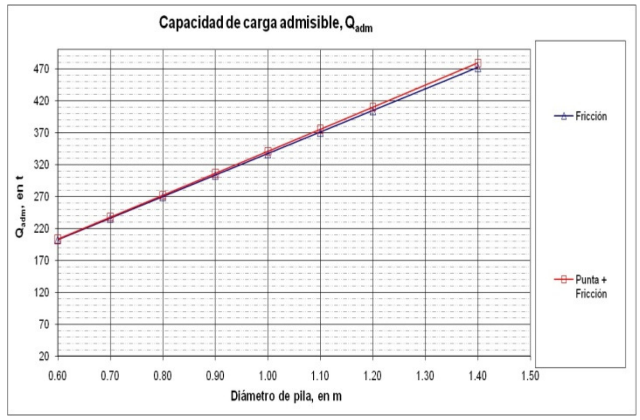

Figure. 3. Admissible load bearing capacity of drilled shaft foundations in terms of diameter.

Beyond any doubt a set of more complex solutions could have been sought for, as well as more sophisticated analyses, but as mentioned before the idea was to go back to the basic principles of geotechnical engineering.

One of them was to recall the principle of stress dissipation as well as that involving moduli of elasticity of stiffer strata overlying strata with smaller values of such modulus, a principle in which, according to a paper published by Burmister (1945) attention is paid on the reduction of the deformations whenever the modules increases more the 10 times. This principle that is certainly valid oftentimes is set aside when seeking for (an in most of the cases not obtaining) much more sophisticated solutions.

Keeping this principle in mind helps us to define the most feasible solution to resolve this type of problems. This alternative, that in the strict meaning of the word was not actually related to soil improvement, because the superficial ground had been previously scraped in full, helped decrease the settlements and gave an assurance that the soil was not bound to fail.

Construction of the rockfill embankment implied placement of boxes with a width slightly larger than the skid separation, being of approximately 1.50 m and covering the distance from the casting yard to the wharf, with a depth of about two meters. This depth was not increased to prevent the effects of the phreatic water level.

The boxes were filled with controlled material; in this case it was anticipated to use rock fragments with sized not to exceed 1.5" free from sand and fines.

The type of rock to be used corresponded to the material with the highest compressive strength and having the optimum shape (angular) so as to obtain a minimum void ratio.

5 RESULTS

5.1 Considerations and methodology

The considerations taken into account were as follows:

The rockfill embankment was covered with a steel plate having a thickness of one inch and the skids were going to be supported by this plate. The plate would be used to distribute uniformly the weight transmitted by the tetrapod.

Boussinesq´s theory is valid because, according to it, the stress transmitted to the subsoil was dissipated beyond a depth of 2 m as much as 73%.

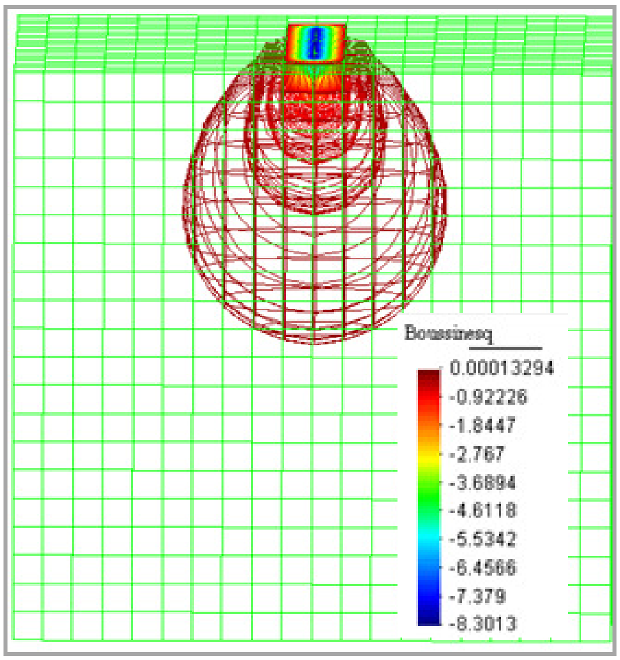

Figure 4. Stress distribution in the soil mass.

The strength of the rock is determined assuming a highly fractured rock mass and the potential failure of the walls of the subsoil is determined using the theory of Christoulas *et al*. (2000).

5.2 Settlements with no rockfill embankment

The immediate settlements that were calculated for the skids became of about 7 cm. The calculated immediate settlements are shown in Figure 5.

According to the analyses carried out settlements in the short term would be of about 7 cm, whereas in the long term they would reach 15 cm, with an intermediate value of 12 cm after six months.

According to the information available, the tetrapod structure takes from three to six months in being built, depending on its dimensions.

As mentioned before, the major problem for moving the tetrapod structure was a possible failure of the subsoil due to shear strength, causing in the ground a sudden failure. This point is strengthened because settlements are not as high as those expected; however, although the soil would have had the capacity to bear the load of 8.33 t/m^2, it was assumed that the calculated settlement would hinder to a large degree hauling of the tetrapod structure into the barge because of its heavy weight.

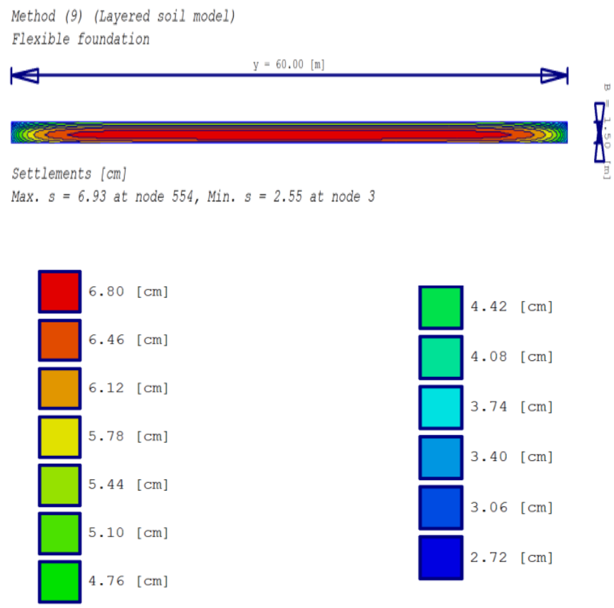

Figure 5. Settlements with no rockfill embankment.

5.3 Settlements with rockfill embankment

The analyses performed with the rockfill embankment gave the results indicated in Figure 6.

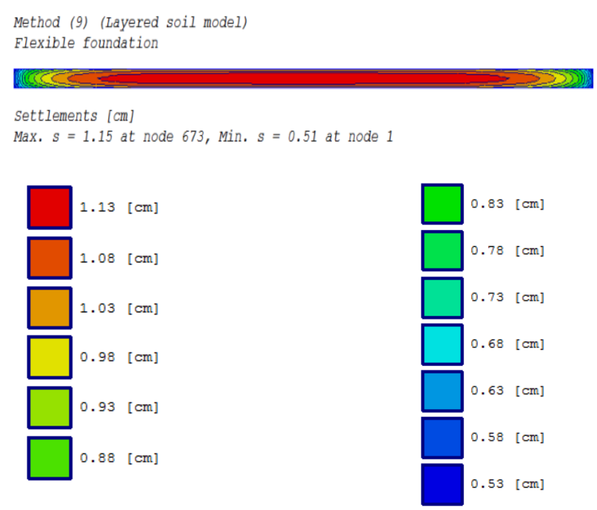

Figure 6. Total calculated settlements at the end of six months.

5.4 Strength of the rock mass

The rockfill embankment was placed using dumped material and a small vibratory roller in 30-cm thick layers with the purpose of achieving a minimum void ratio.

The study suggested the execution of an analysis for a poor quality rock according to the rock quality index suggested by Bieniawski (1984); this RMR index takes into account among other parameters the roughness, the filler, the weathering, the RQD value, etc. The rock was assumed to be highly fractured.

By using this index and the method proposed by Hoek and Brown (1999), the strength that would have the rock as a mass and not as an undisturbed rock was calculated because as an undisturbed rock (individual property) very high values are generated not representing features such as fracturing of the rock mass. According to Hoek and Brown (1985), the shear strength of the rock mass is given by:

$$\sigma_1 = \sigma_3 + \sigma_{ct}\sqrt{m\frac{\sigma_3}{\sigma_{ct}} + s} \quad (1)$$

where: σ_1 and σ_3 = major and minor principal stresses, respectively; σ_{ct} = unconfined compressive strength of the rock, in MPa; *m* and *s* are determined by means of the following equations.

$$m = mi \exp((RMR-100)/14)$$

$$s = \exp((RMR-100)/6) \quad (2)$$

The type of rock to be considered was a sedimentary rock classified as rudite class (fragmentary), gravel sized.

The results obtained are presented in Table 1.

Table 1. Results obtained for the strength

Criterion of Hoek and Brown (1999)	
Strength of the rock mass	Compressive strength of the rock mass
σ_i = 23 t/m^2	σ_{mi} = 9 t/m^2

5.5 Strength of the soil walls

The assumption made involves the criterion of gravel columns available taking into account that in the case of an individual gravel column a higher strength exists because of its circular shape; however, it is a good approximation to assume them as boxes.

According to Christoulas *et al.* (2000), the load bearing capacity of the soil walls can be calculated from the following expression:

$$Qadm = (al * Cu)/FS \quad (3)$$

where: *al* = Area of the box; *Cu* = Undrained cohesion of the clay = 40 kPa. Therefore

Qadm= 640 t

Mention should be made that the load considered is of about de 600 t; however, the load transmitted to the walls is smaller.

According to these results the rockfill embankment would provide an adequate stability to the soil, both in strength and in deformability. The tetrapod structure that was hauled for subsequent assembly offshore can be observed in Figure 7.

Figure 7. Tetrapod structure at Cobos marine terminal.

6 CONCLUSIONS

The load to be transmitted to the soil was higher than that to be borne by the subsoil if a shallow foundation is assumed; in the case of a deep foundation time and cost repercussions would develop.

The immediate settlements assumed to be experienced by the soil assuming no sudden failure due to shear stress were of about 7 cm, whereas those considering the construction of a rockfill embankment were in the order of 1.5 cm due to the increase of the modulus of elasticity and to the dissipation of the stress in the subsoil, although it should be mentioned that in the field those settlements were actually smaller than one centimeter. In addition, it was assumed that the rockfill embankment acted as a highly fractured rock mass and that according to the theory applied its load bearing capacity was higher than the stress transmitted to the soil, whereas for the case of the walls of the boxes, their bearing capacity was higher for a vertical load equal to 600 tons.

The designer should also be aware that a variety of theories will also be available to help him in finding out the solution that best fits the actual conditions of soil behavior. Under these guidelines, mention should be made that in the professional practice we have to follow a practical approach, without setting aside safety and correct functioning of the foundation structure to be built; however, sometimes simple and quick solutions should be sought for and oftentimes it is indispensable to resort to the basic principles of engineering and decide if in certain cases the principle of deliberation is applicable: “Being all the factors the same, the most simple solution in many an occasion tends to be the right one”.

REFERENCES

Bieniawski, Z.T. (1984). “Rock mechanics design in mining and tunneling. Ed. Balkema

Braja M. Das (2001) “Principios de Ingeniería de Cimentaciones”, Ed. Thomson. Cuarta edición

Burmister, D.M. (1945). “The theory of stresses and displacements in layered systems”, Journal of Applied Physics, Vol. 16

Christoulas, S. et al. (2000). “An Experimental Study on Model Stone Columns”. Soils and Foundations, Vol. 40, No. 6, pp. 12-22

Hoek and Brown, E.T. (1993). “Rock Mechanics for Underground Mining”

(Raísz E, 1964). Contribución a Servicio Geológico Mexicano