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Ground Improvement in Zárate

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Abstract

A port facility project named Terminal Zárate is under construction 90 Km North of Buenos Aires, Argentina. As part of the construction process, a 3.5 m thick hydraulic fill and a 3.0 m thick compacted soil embankment were placed on top of a soft soil deposit. A tight construction schedule demanded that the placement of the fill be completed before six months, which in practice meant that an improved soil drainage system needed to be installed. A higher preload was chosen instead, taking advantage of the existence of some sandy seams in the clayey soil. A monitoring program was used to check estimations and to assess when the required degree of consolidation was achieved. Some relevant results are presented here, along with the mechanical characterization of the soils involved in the problem.

Introduction

The Hidrovia Paraná – Paraguay is the main fluvial transportation system of the Mercosur (Argentina, Brazil, Chile, Paraguay, Uruguay). It connects the Atlantic Ocean with Argentina, Paraguay and Brazil through the Río de La Plata, Paraná and Paraguay rivers. Several port facilities habe been installed along it’s coastlines, mainly near Buenos Aires.

Murchison, a one-century-old port operator in Argentina, started a terminal project during 1994, named “Terminal Zárate” and located in Zárate City, at the right margin of the “Paraná de las Palmas” river. The city, 90 km North of Buenos Aires, is strategically settled on the main transportation route of Argentina, serving the northern part of the country through highway, railway and fluvial systems. This fortunate location motivated in turn an explosive industrial development in the area during the last two decades. Today, Zárate and it’s neighbour Campana City are important centers for the chemical, cement, siderurgic, oil, beer, automotive and alimentary industries.

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Terminal Zárate is being built on a 1.300.000 m^2 lot, with 1.100 m of exclusive – access coastline. The first stage of the project, operating since 1995, is an automotive terminal, with 600.000 m^2 of paved yards named “Autoterminal Zárate”. It has two car carrier berths and other facilities exclusively dedicated to this particular activity.

The second stage of the project, and the most important one, is a container terminal facility that shall be developed on the remaining area, i.e., some 700.000 m^2 of land. This project is now under construction and is the subject of this contribution.

The car – carrier aerial view is shown in Figure 1. A small part of the area belonging to the second stage of the project is visible at the right side of the photo.

Figure 1. Aerial view of Autoterminal Zárate.

The site is located on the river valley, resting on top of 26 m of soft alluvial sediments. Almost the whole area is flooded during high tides, so the ground level must be elevated at least 3 m above surface level to assure adequate operating conditions throughout the year. The earth fill on top of soft soils triggers a consolidation process wthich is the most important design problem the project has to deal with.

The construction of the second stage of the project began during late 1998, at an area of about 80.000 m^2, 400 m along the coastline. A key factor in the design process was the selection of the ground improvement method, and finally a preload was selected. According to the schedule the preload process should last six months at the most, this period being only a fraction of the theoretical lapse necessary for full consolidation of the soil deposit.

Past experience of the authors decided them to disregard conservative time predictions, to place a heavy preload fill and to implemment a modest field monitoring program, their results being the contribution of this paper.

General Soil Profile

The right margin of the Paraná de las Palmas is formed by a thick deposit of Holocene sandy silts and clays of the Postpampeano formation that lay on top of heavily over - consolidated silty clays of the Pleistocene Pampeano formation, or on top of very dense, quartzitic sands from the Pliocene Puelche formation. In the site area, Postpampeano soils are 26 m thick, and are immediately followed by dense sands. Figure 2 shows a typical boring log.

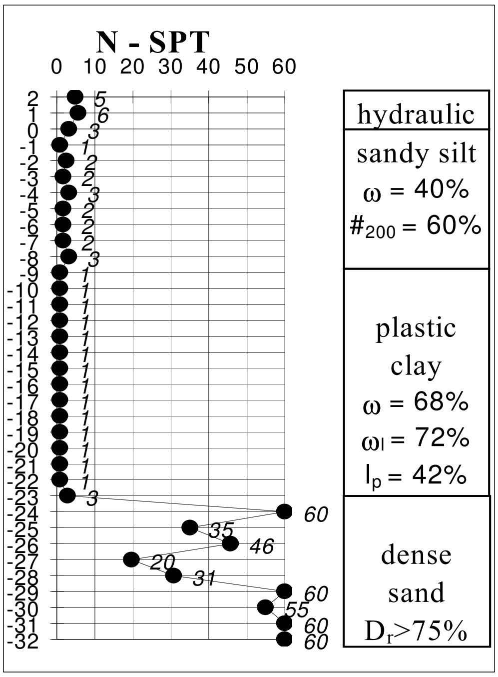

Figure 2. Typical boring log.

A modified, 2 ½" OD sampler driven by a donut trip hammer was used for the SPT test. Field N values were corrected by a factor of 0.80 to account for the larger diameter of the sampler, but no further corrections were made. Figure 3 represents a simplified transverse profile at the site area.

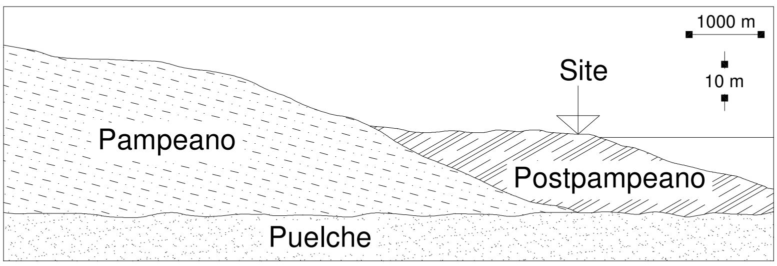

Figure 3. Simplified transverse profile at the site area.

Postpampeano formation: Postpampeano formation is a soft, uncemented and K_0 NC layered soil deposit of fluvial and marine origin. Classification data plot in the Casagrande chart over a wide range of values, but tend to congregate around a line as shown in Figure 4. The equation of this line is

$$IP = 0.86(\omega_L - 23) \ \pm 12\%$$

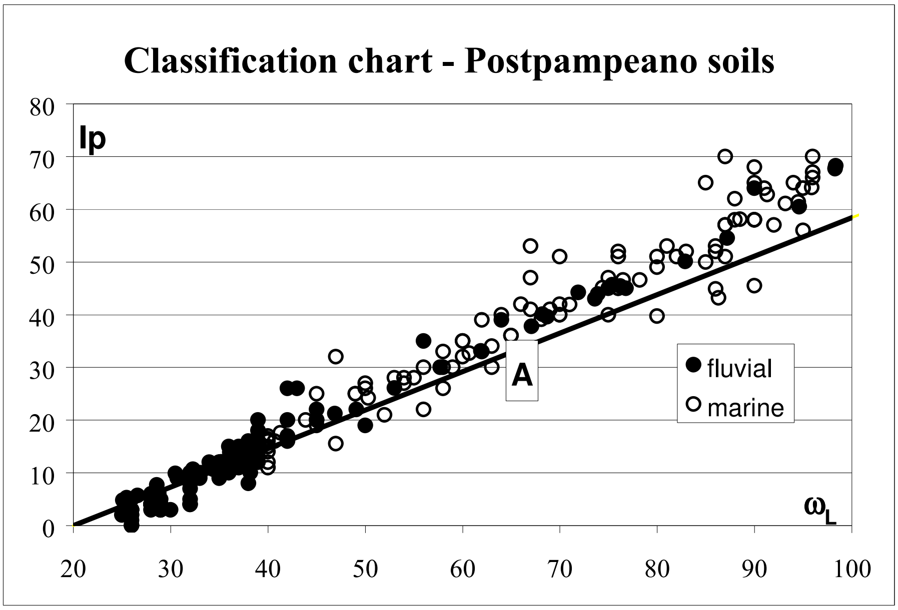

Figure 4. Classification tests of Postpampeano soils.

At the site area, the upper part of the formation consists in an fluvial silt layer. At a depth of about 10 m below surface, soil changes to plastic, moderately sensitive marine clays with silty and sandy seams and lenses.

Laboratory EOP oedometer test results are bounded by Terzaghi and Skempton correlations (Terzaghi, 1996)

$$C_c = 0.009(\omega_L - 10) \ \pm 30\%$$

$$C_c = 0.007(\omega_L - 10) \ \pm 30\%$$

Partial remolding during sampling and the lack of a well-developed structure may explain some of the lower values reported for C_c. The coefficient of consolidation is roughly related to the liquid limit through the following expression, (C_v in cm^2/sec)

$$\log_{10}(C_v) = -\omega_L / 20 \ \pm 20\%$$

This expression is a fit of 20 selected laboratory test results and has been favourably checked for a few years with subsequent available data. It is applicable in the range $40<\omega_L<80$.

Non-plastic seams impose a strong hydraulic anisotropy to the formation, deeming the application of Terzaghi equations useless for consolidation time computations. Seams are spaced from a few centimeters to over two meters, and some of them are not continuous. A very limited local experience exists with the use of CPT in these soils, and no CPT logs were available to check the spacing between sandy layers. According to local experience, the term H^2/C_v , applied to the complete soft layer, should be estimated in the range 14 to 36 millon seconds, mainly depending on spacing and thickness of the sand seams.

Initial undrained moduli of deformation for clays and plastic silts are roughly related to the undrained shear strength s_u by the expression

$$E_{ui} = 400 s_u \pm 30\%$$

where s_u is measured in CTCU tests of samples obtained by static penetration ot the modified 2 ½” OD sampler. Backward analyses of the behavior of real structures show that the in situ undrained initial moduli of deformation lies in the range E_{ui}=15 - 30 MPa, see Sfriso, (Sfriso, 1997). For the particular problem being presented in this paper, the actual value of E_{ui} used has little effect on the results of the analysis, as it is mainly a one dimensional consolidation process.

In terms of effective stresses, E_i can be evaluated by Jambu expression (Jambu, 1963)

$$E_i = C\left(\frac{\sigma_3}{p_{ref}}\right)^n$$

where the parameter C is upper bounded by the correlation (Núñez, 1997)

$$C = \frac{2000}{\omega_L - 10} p_{ref} \pm 30\%$$

p_{ref} is a reference pressure, usually 100 KPa. n = 1.0 adequately models the elastoplastic behavior of unstructured soils and simplifies analytic calculus.

For sand and non plastic silt layers, the most used parameters are $C \sim 20 - 40$ MPa, $n \sim 0.80$.

Friction angles for the silty and sandy soils sheared under mean overburden pressure are in the range 25° – 30° with a total deformation to failure of 8% - 10%, for fixed piston samples.

Puelche formation: The Puelche formation consists in a thick deposit of very dense, fine to medium quartz sands of Pliocene to Pleistocene origin that extends over a very wide area of the central – eastern region of Argentina. This layer provides firm soil strata competent enough to resist pile foundations of very heavy structures, and almost all the port facilities that exist along the Paraná coastline are founded on these sands. For the purposes of this project, the Puelche formation acts merely as a rigid draining boundary for the Postpampeano soils.

Fill and Ground Improvement Program

The fill and paving self weight loads are about 45 KPa and 10 KPa, respectively, the design loads from container storage being 15 KPa. On the whole, about 0.98 m of average settlement was expected for a maximun load of 70 KPa. To speed up the construction process and to meet the project schedule, an additional surcharge of 70 KPa was specified, totalizing 115 KPa. The ultimate expected settlement for this load was 1.24 meters, while the settlement forecasted for the service loads was expected to occur four to six months after toping out the preload.

A two layer fill was placed at +1.00 / +1.20 m. level. Mean load actually placed was 123 KPa, see Figure 5. The description of each layer is:

- Hydraulic granular fill, vibrated at the surface after deposition of the full layer, 3.20 m to 3.60 m thick.
- Non-plastic fine soils from the Pampeano formation, locally used as embankment materials for paving and roads, 3.40 m to 3.60 m thick.

The sand was obtained from the riverbed, not far from the location of the project, and formed part of the permanent fill. Specifications stated that mean relative density should be greater than 65%, this requirement being met merely by hydraulic deposition and vibratory compaction from the surface. The sand layer served both to reach an adequate topographic level and as a draining layer for the underneath consolidating soils.

The silty material for the preload was obtained at a quarry located 1.0 km away from the site area, and was later reused as permanent fill in nearby areas. A small fraction (0.80 m) of the silt layer remained as permanent fill, while the rest of it was merely dumped, with no compaction requirements. This simple construction procedure was convenient to promote competition among many small local contractors.

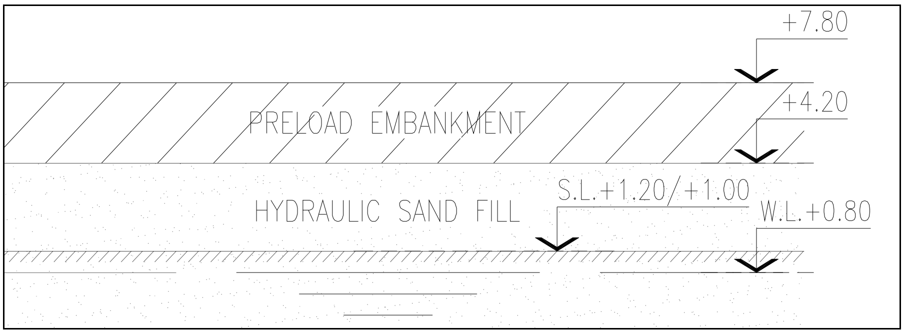

Figure 5. Fill and preload embankment levels.

The river shore has a steep 1:3 slope immediately aside of the fill embankment, so there was some concern regarding overall slope stability. A rough computation of the pore pressure generation – dissipation process showed that there was enough shear resistance in the natural soils for the construction schedule implemented, as long as the forecast of pore pressure dissipation was accurate enough.

Soil Investigation and Field Instrumentation

A limited soil investigation program was conducted to confirm the estimated thickness of the strata and to detect the existence and spacing of permeable seams and layers within the clay deposit.

Three 30 m deep borings with SPT were made, and six fixed piston samples, 76 mm dia. and 80 cm long, were recovered at depths varying from 5.10 m to 19.30 m. With these samples, six EOP oedometer tests and six CTCU tests were performed. The result of the investigation program is a set of parameters used for the mechanical and hydraulic characterization of the deposits and for the computation of soil response to the earthworks. These parameters are listed in Table 1.

Table 1. Soil parameters of the four layers.

		Fill: 4.5 to +1.5	Silts: 1.5 to –11	Clay:-11 to –24	Sand below –24	
γ	KN/m^3	20	18	16	21	total unit weight
k	m/sec		10^{-6}	$5\ 10^{-8}$		estimated mean field permeability
s_u	KPa		0.30 p_v	0.22 p_v		undrained shear strength p_v is eff. overburden pressure
ϕ	deg	34	28	24	40	friction angle at effective overburden pressure
ψ	deg	4 - 6	0	0	8 – 10	dilatancy angle
C	p_{ref}	900	400	138	2000	ref. stiffness at reference pressure
n	-	0.70	0.80	1.00	0.40	Jambu exponent
ν	-	0.20	0.30	0.25	0.20	Poisson ratio
C_v	cm^2/sec	-	$1.2\ 10^{-2}$	$6.5\ 10^{-4}$	-	consolidation coefficient
Cc	-	-	0.24	0.50	-	compression index
σ_{vc}	KPa	-	100	130	-	preconsolidation pressure

The results of the oedometer tests are presented in Figure 6. Vertical stresses were normalized to the estimated overburden pressure of each sample.

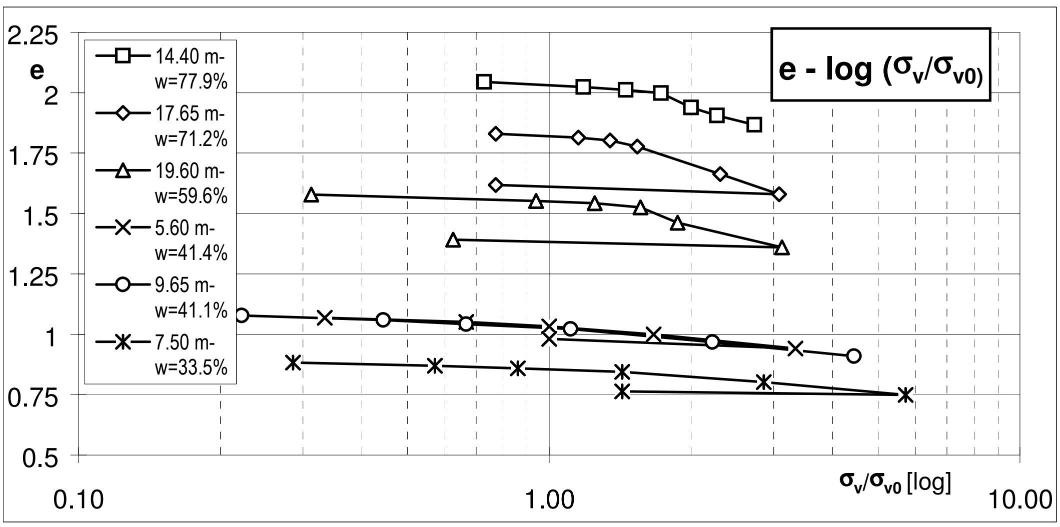

Figure 6. Normalized results of oedometer tests of fixed piston samples.

Field Monitoring

A monitoring program was designed to control the consolidation process. Three settlement indicators were installed in the upper silty layer, while another three indicators and three piezometers were placed in the clay layer. These instruments were put in place as soon as the hydraulic fill was dense enough to walk on top of it. By that time, a settlement of 50 cm had already occured. Piezometer readings and topographic surveying were performed twice a month. No piezometers were placed in the non-plastic silt layer, as the estimated drainage time was too short for a discontinuous recording procedure.

The average results of the instrumentation program is plotted in Figure 7, where *q* represents the preload, *u* is the excess pore pressure, d_1 is the settlement of the silty layer, d_2 is the settlement of the clay layer, and d_1+d_2 is the total settlement observed at surface. A consolidation degree of 80% was achieved within three months ater topping the preload out, and ultimate primary consolidation is expected before one year. A primary consolidation degree of 80%, calculated independently by pore pressure dissipation and settlement readings, was the specified value for initiation of preload removal, so the tight schedule requirement of six months for total preload time was successfully reached.

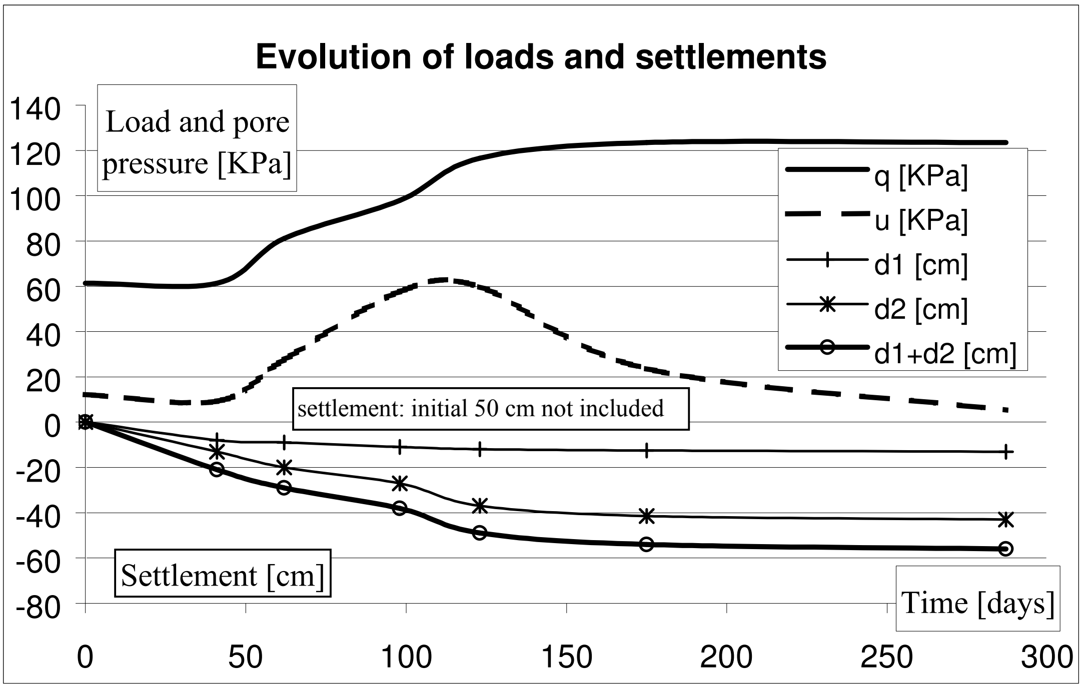

Figure 7. Results of the field instrumentation program.

Results presented in Figure 7 are average values of all the instruments and devices installed. One of the piezometers happened to be placed at or near a permeable seam, and it’s readings are completely different (roughly one third) from the other two instruments results. Malfunctioning was disregarded by tests performed on the instrument after removal.

The concern related to the stability of the margin slope and management decisions influenced in the fact that over three months were needed to complete the construction of the preload fill, and a significant portion of the total settlement occurred during this time.

Due to the fact that the fill remained in place for a longer time than originally expected, the clay layer almost gained full consolidation for 123 KPa, 70 KPa being the pemanent increase of vertical pressure. This slight overconsolidation shall probably minimize the effects of secondary compression of the clay deposit, and long time behavior of these soils is not expected to have a major influence in the overall perfomance of the facility.

Conclusions

An adequate planning and an inexpensive monitoring program allowed for considerable savings in an important port project in Argentina. Past local experience, along with a limited soil exploration program were the sole background used to disregard the application of expensive methods for drainage control, i.e. vertical drains, pumping, piling, etcetera.

The existence of some seams and sandy layers embedded in the clay deposit provided the preferred drainage path for a 26 m soft soil deposit loaded with 123 KPa in less than five months. This heavy load placed directly on the coastline and on top of a 1:3 natural slope, imposed settlements greater than 1.0 m, and overall deformations and excess pore pressures reasonably similar to initial predictions. The soil parameters used in the prediction process and the average results of the monitoring program were presented.

The controlled soil improvement achieved resulted in an effective preload surcharge greater than the maximum estimated working load for the port facility, this condition being very favorable for the design of thick concrete pavements needed for the operation of the terminal. The overall construction time was nine months, starting at hydraulic fill and ending at commencement of removal of preload fill.

By the time of submission of this paper, the wharf is under construction, the preload is almost totally removed and placed elsewhere in the project area, and some hundreds of containers are being placed on top of the compacted fill that remains. Settlements almost ceased after fill removal and are no longer recorded, except for a few control points near the coastline.

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References

Janbu, N. (1963). "Soil compressibility as determines by oedometer and triaxial tests". *Proc. Eur. Conf. Soil Mech. Found. Eng., Wiesbaden, I, 19-25.*

Núñez, E (1997) "Interacción suelo – estructura: Predicción, Comportamiento e instrumentación de campo*". In: 1 st Congreso Paraguayo de Ingeniería Geotécnica, Asunción, 1-19.*

Sfriso, A. (1997). "Formación Postpampeano. Predicción de su comportamiento mecánico". *In: 3rd CLIGJ*, *Caracas, pp. 1-10.*

Terzaghi, K., R. Peck, and G. Mesri (1996). *Soil mechanics in engineering practice*. 3rd Ed., Wiley, 1996.

Related material

Bolognesi, A. and O. Moretto (1959). "Propiedades del subsuelo del Gran Buenos Aires". *1st CPMSC, I, 303-314.*

Bolognesi, A. and O. Moretto (1959). "Cimentaciones con pilotes solicitados por fricción negativa". *1 st CPMSC, I, 315-325.*

Núñez, E., O. Vardé, A. Bolognesi and O. Moretto (1967). "Algunas relaciones entre los métodos de cálculo de la carga permisible y el comportamiento de pilotes de hormigón". *III CPMSIF, Caracas, II. 673-704.*

Núñez, E. and O. Vardé (1968). "La compresibilidad de los suelos como fenómeno asociado al desarrollo de su resistencia". *1st RAMSIF, La Plata, 155, 199.*

Núñez, E. (1986) "Geotechnical conditions in Buenos Aires City". V *Intern. Congress IAEG, IV, 2623-2630.*

Núñez, E. (1994). "Propiedades mecánicas de materiales granulares incoherentes". *In: Anales Acad. Nac. Cs. Ex. Fís. Nat., Buenos Aires, 46, 71-89.*

Trevisán, S. (1968). "Predicción de asentamientos de terraplenes en el tramo VI de la autopista La Plata – Buenos Aires". *1st RAMSIF, La Plata, 191, 202.*