# Soft Ground Improved by Rigid Vertical Piles. Experimental and Numerical Study of Two Real Cases in France

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# SOFT GROUND IMPROVED BY RIGID VERTICAL PILES. EXPERIMENTAL AND NUMERICAL STUDY OF TWO REAL CASES IN FRANCE

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## ABSTRACT

Settlements remain the major concern during the construction and along the life of a roadway embankment built over soft soils. A foundation ground reinforced by a system of rigid piles is an effective solution; it reduces and homogenizes the surface settlements.

Rigid piles reinforcement system has been strongly developed in France for the last fifteen years, however, confrontations of several design used methods highlighted important variations in results. This results in implementing a French National Research Project A.S.I.RI. (Grounds Improvement by Rigid Inclusions) gathering construction companies, engineering and design departments, universities and research centers. This project aims to propose guidelines for the design, and the construction of reinforced soils by rigid inclusions.

After a short presentation of the technique, this article is devoted to two French case studies of ground improvement by rigid piles, the Senette's street and the Ramp of Glain examples. In both cases a roadway embankment is intended to be supported by soft soil. The compressible soils are soft sandy clay from alluvial origin. The performance of the embankment supporting system is assessed based on monitoring data obtained from various instruments installed during construction including settlement cells, inclinometers, piezometers, total stress cells and Geodetect settlement strip. The study was made with finite differences and finite elements numerical tools. Some analytical methods are used. A confrontation between experimental, analytical and numerical results is then presented.

## INTRODUCTION

Columns-supported embankments have been used to allow fast embankment construction over soft soils. It combines three components: (1) embankment material, (2) a load transfer platform (LTP), (3) vertical elements extending from the LTP to the stiff substratum. Optional configurations can be made by adding geosynthetics or piles caps. The surface and embankment loads are partially transferred to the piles by arching which occurs in the granular material constituting the embankment. This causes homogenization and reduction of surface settlements. Soil arching is a natural phenomenon encountered in geotechnical engineering (Terzaghi, 1943). Friction along the piles is also involved in the improvement mechanism, leading to a complex soil/structure interaction phenomenon (Fig. 1).

Literature shows different approaches about the functioning of the rigid piles method. However, authors like Kempfert 1999; Hewlett et Randolph, 1988; Combarieu 1988; John 1987 (in Briançon 2002) all agree about the presence of the soil arching effect. Countries as Germany and England have based their rigid pile technique standards on these authors work; Kempfert for EBGEO and John for BS 8006.

In France, even if rigid piles have been widely used, guidelines for their use do not yet exist. A French national research project "ASIRI" (Amelioration de sols par Inclusions Rigides) is starting to solve this problem.

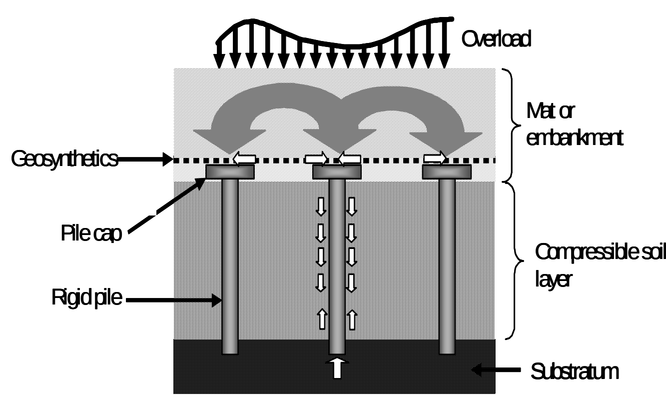



*Fig. 1. Rigid piles supported embankment.*

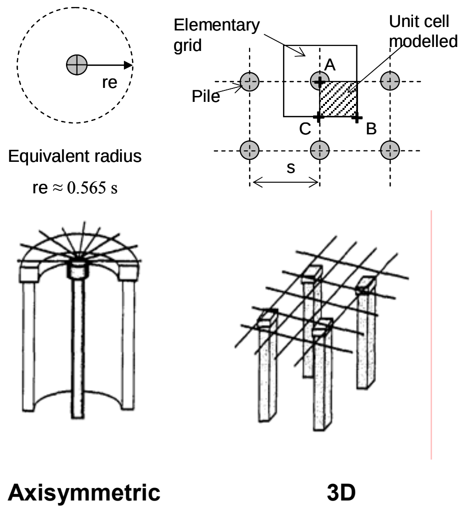



*Fig. 2. Different symmetric approaches.*

In this paper two cases of rigid piles reinforcement technique are presented, in both soil treatment was necessary to ensure stability of embankments and roads. The rigid piles technique was selected over stone columns or prefabricated vertical drains (PVDs), because of poor soil properties, the presence of underground structures, and mainly to allow rapid construction.

First, Rampe de Glain case is presented; settlement and stress predictions are made with 2D FEM and analytically. Usually, the design of this technique is made with numerical Finite Elements Methods tools in 2D axisymmetrical configurations while it's a fully three dimensional problem (Kempton et al.,1998). Then based on this first experience a numerical parametrical study is proposed on the Carrières sous Poissy case, where we compare and discuss results obtained with two and three dimensions numerical simulations (Fig. 2). Difference Finite Methods (FDM) computer softwares as Flac2D, Flac3D are used.

CASE STUDY. LA RAMPE DE GLAIN

General project information

The project is located at Bayonne, in the French region called Pyrénées Atlantiques. It consists on the construction of a street 270m long, to connect an entertainment complex to a main road. Difference between ground levels of the project and the main street is considerable; it can reach up 12m in some parts. A high embankment was necessary to make the connection (Fig. 3).

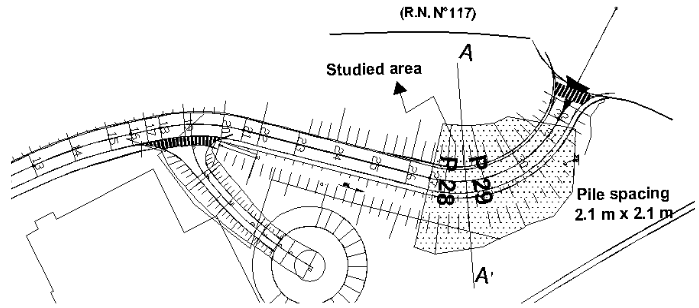



*Fig. 3. Rampe de Glain studied area.*

To fulfil the objectives of minimising settlements and assuring stability a group of piles was constructed. V.C.C (Vibro Concrete Colums) rigid piles were used in this project. Spacing was selected following the embankment height and making empirical analysis.

Ground conditions.

Geological information shows the presence of important soft deposits of the Quaternary's period mainly composed by plastic clays and alluvial deposits over a substratum from the Eocene's epoch composed by schist and calcareous rocks. Site investigation carried out defined the first 7m like a highly compressible clay layer (*Table 1*).

Table 1. Mechanical Menard's soils parameters.

| Soils nature | Layer average thickness (m) | Menard's Modulus $E_m$ (MPa) | Menard's Limit pressure $p_l$ (MPa) | $\alpha$ |
|---|---|---|---|---|
| Plastic clays | 6-8 | 2.4 | 0.3 | 1 |
| Sandy clays | 7-9 | 5.0 | 0.67 | 2/3 |
| Schist to sandstone | - | 21.6 | 2 | 1/4 |

Instrumentation

This survey is composed by: total pressure cells (PrTot), pneumatic piezometers and cells settlement system (TCP) from RocTest Telemac (Fig. 4).

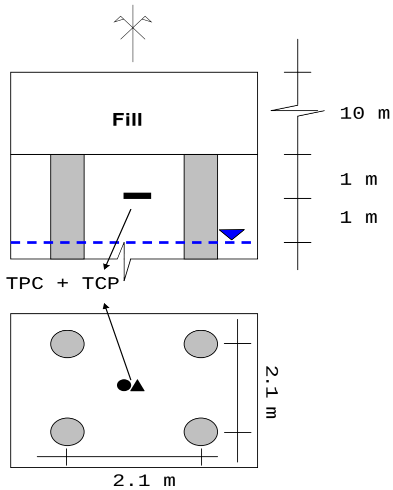



*Fig. 4. Schematic position of measuring cells*

Analysis and results

Combarieu's method was used to analyse the problem. Confrontation with numerical and experimental results will be presented.

Combarieu (1988) proposes a design method based on lateral negative friction (Fig. 5). This author made a global approach in which loads are transferred by lateral negative friction on concentric surfaces around piles that extend above them on the embankment material. He considers that soil arching will develop since soft soil will settle more than piles. Vertical stress applied to soft soil surface $q^+_s$ is calculated using ( 1 ) $m_R$ value ( 2 ) is obtained from non attached conditions on granular fills ($\lambda = 0$) and Ktan $\varphi_R$ values between 0.5 and 1.0. For further information on this method please refer to Combarieu, 1988.

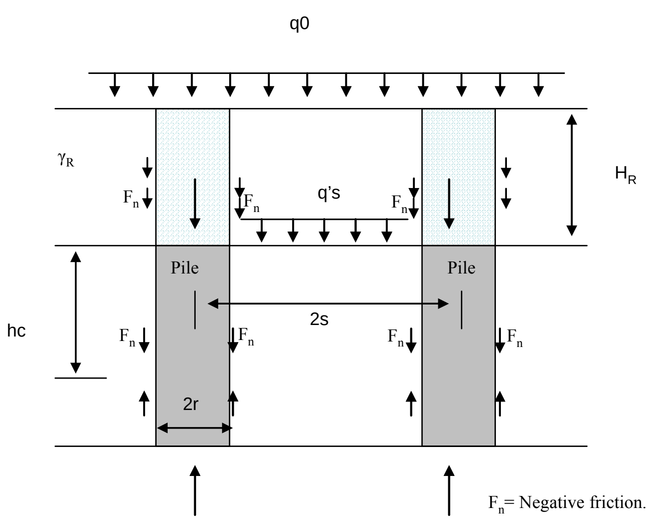



*Fig. 5. Combarieu's method (1988)*

$$q^+_s = \frac{\gamma_R}{m_R}(1-e^{-m_R H_R}) + q_0 e^{-m_R H_R} \qquad (1)$$

$$m_R = \frac{4rK\tan\varphi_R}{s^2 - r^2} \qquad (2)$$

Numerical analysis was made with Plaxis. Soils were simulated as nonlinear elastoplastic materials with the Hardening soil model constitutive model (HSM). Fig. 6 show the studied profile. 2.1 meter space piles configuration with maximum embankment high was studied.

The construction of the embankment was simulated by layers of 0.5 meters high each, at the end of its construction an overburden of 20KPa is added to its surface. The capping ratio defined as the proportion of the surface covered by the pile caps, is 3.6% for this case.

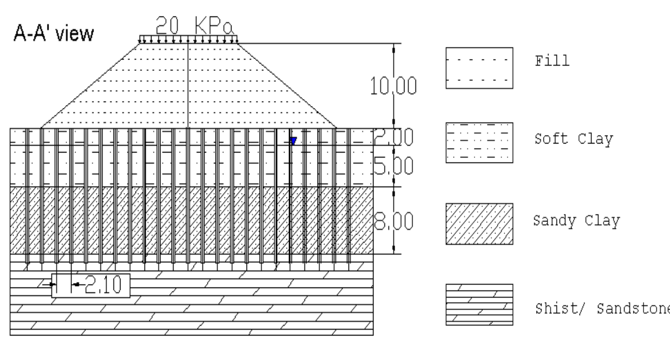



*Fig. 6. Studied profile (P28, P29).*

Due to the lack of information on the fills characteristic, we use Hostun RF dense sand (Flavigny et al 1990) to simulate its behaviour. Hardening soil model parameters were determined from laboratory triaxial tests. Triaxial tests made by Djedid, A (in Flavigny et al 1993) at a confining pressure of 100 KPa were took as reference. Back analysis on several triaxial test have been done to deduce the geotechnical parameters (Fig. 7).

$$Eoed = Em/\alpha \qquad (3)$$

$$E = Eoed\,\frac{(1+\upsilon)(1-2\upsilon)}{(1-\upsilon)} \qquad (4)$$

$$if\ \upsilon = 0.3\ then\ E = 0.74\,Eoed \qquad (5)$$

With:

Eoed = Oedometric's Module — E= Elastique de Young's modulus

Em = Ménard's Module — ν = Poisson's ratio

α = rheologic soil constant depending on the soil consolidation state

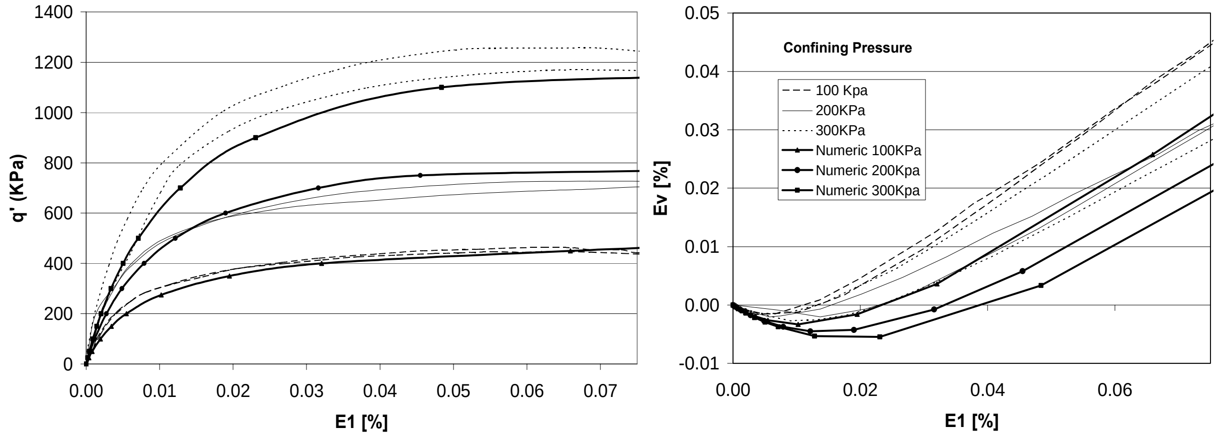



*Fig. 7. Results of triaxial drained test on dense Hostun sands simulated with Hardening soil model.*

Numerical model parameters are presented on Table 2. Soil parameters were obtained from Menard's pressiometer results, using equations ( 3 ), ( 4 ), and ( 5 ).

Table 2. HSM parameters

| Parameter | Fill | Plastic Clay | Sandy clay | Schist/sandstone |
|---|---|---|---|---|
| Volumetric weight | 18 | 17 | 17.5 | 19 |
| E50 ref | 37 | 1.78 | 5 | 63.9 |
| Eoed ref | 29.6 | 1.78 | 5 | 63.9 |
| Eur ref | 90 | 5.34 | 15 | 191.8 |
| C' | 0 | 5 | 5 | 0 |
| $\phi$' | 41 | 15 | 20 | 35 |
| $\nu$ur | 0.2 | 0.2 | 0.2 | 0.2 |
| $\psi$ | 14 | 0 | 0 | 5 |
| Power | 0.5 | 1 | 0.75 | 0.5 |
| Tensile strength | 0 | 0 | 0 | 0 |
| Reference pressure (KPa) | 100 | 20-30 | 50 | 70 |

$$E_{ur}^{ref} = 3E_{50}^{ref} \quad ; \; E_{50}^{ref} = E$$

Table 3 compares the results obtained with the two designing methods to experimental data. Combarieu's and FEM don't seem to approach experimental data. Combarieu's method predicts 18% less settlement and the double of pressure at the measured point. FEM results present similar behaviour than Combarieu's; 32 % less settlement and almost the double of pressure than experimental. Differences between the two analysis can be easily explain, Combarieu's calculus are simple elastic predictions obviously different than a elastoplastic numerical model, however, their difference with experimental results are hard to explain. Two assumptions are possible:

- Stress instrumentation is defective, maybe damaged after installation.
- Data is biased by installation procedure. It is known that Pressure cell devices placed between soft soils layers can result in erroneous readings.

So, a strong incertitude is to be considered on pressure readings.

Fig. 8 presents stresses experimental data evolution. A discharging on the soft soil can be seen; loads are not lost but transferred to piles until a state of equilibrium putting in evidence the phenomenon of soil arching.

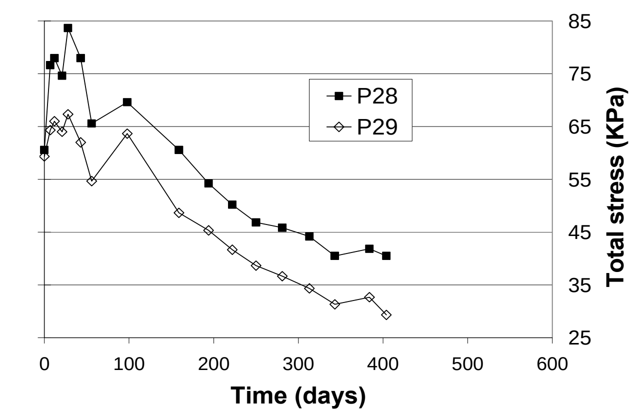



*Fig. 8.Stresses experimental data*

On Fig. 9, numerical settlement results are compared to experimental ones, final state predictions are acceptable however its evolution during and after construction is not well represented, consolidation time is hard to predict, this is a common geotechnical issue. In our case this may be cause by permeability laboratory overestimation.

Table 3. Experimental, analytical and numerical results.

| | Experimental | | Combarieu | FEM |
|---|---|---|---|---|
| | P28 | P29 | | |
| Settlement without RP (cm) | - | - | 66 | 49.5 |
| Settlement with RP (cm) | 21 | 22 | 18 | 15 |
| Effective pressure (kPa) | 40 | 30 | 82 | 67 |

Conclusions

Even if experimental data reveals settlement higher than expected, most of them occurred during the construction phase, residual settlement were followed by topographic surveillance and less than 2 cm settlement was measured on the embankment surface a year after the end of his construction.

Settlement predictions are acceptable but stresses can be contested, nevertheless this study gives qualitative information about rigid piles performance under embankments.

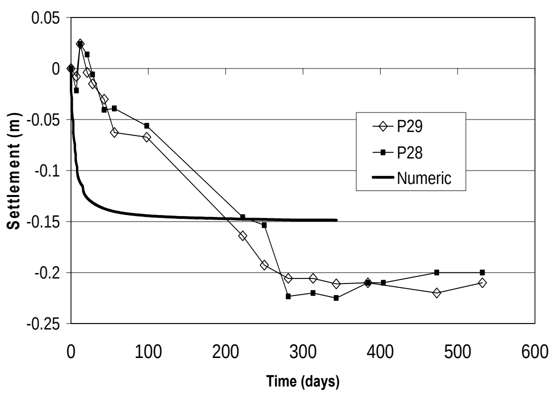



*Fig. 9. Numerical vs Experimental Data*

Indeed, the influence of several parameters acting in this reinforcement technique is still unknown; this resulted in a numerical parametric study presented for the Carrières sous Poissy case.

CASE STUDY. CARRIERES SOUS POISSY

General project information

The project is located in a city called Carrières-sous-Poissy in an urban site next to the Seine River about 35 km northwest from Paris. It consists in the construction of two residential buildings, a new road "Senette's street" and a pedestrian street "Mail" (Fig. 10). The streets were built on a fill of 3.5 meters average height. Geological information shows the presence of alluvional deposits over plastic clays, Meudon's marls and Campanienne's silts. Piles (35 centimeters of diameter) are made up with Driving Back Auger (DBA) technique.

After reviewing design parameters it was decided to delimit two zones, "Senette's Street" with 1.9 meters square spaced inclusions, and "Mail" with 2 x 2.2 meters rectangular spaced inclusions.

Ground conditions

Many soil campaigns were made, mainly using Menard's pressuremeter in-situ test (Fig. 11). Oedometer tests were also made to define compressive soils. Water table is followed with piezometers.

The soils met near the future project and taken into account for the study are as follows:

- Fills: constituted by silts, sands and gravels. Average thicknesses 2m.
- Modern alluvional deposits of the Seine: constituted by muddy silts, with finely sandy layers. Thicknesses 4 to 8 meters.
- Ancient alluvional deposits (AAD): constituted by sands and beige, yellowish gravels. Thicknesses 2 to 4 meters.

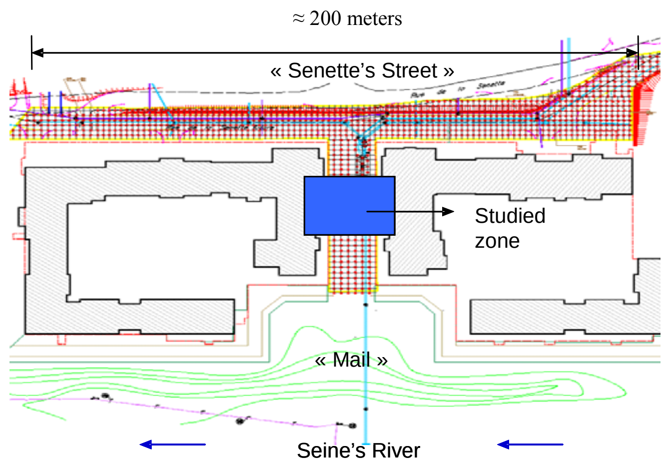



*Fig. 10. Residential Building Project at Carrières sous Poissy. Rigid piles treated zones.*

The watertable level is located two meters under the ground surface, which corresponds to the level of the alluvial watertable, in connection with the Seine River.

The oedometric tests made on undisturbed samples show that the first two meters of silty clay above the watertable are overconsolidated (OCR ≈ 2). It is considered that the grounds under the two meters of depth are normally consolidated.

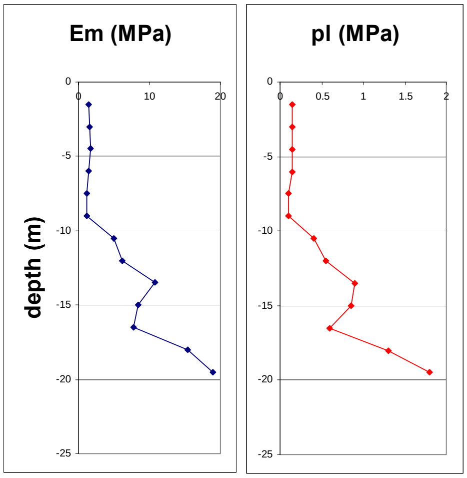



*Fig. 11. Pressuremeter tests. (Em = Menard's pressuremeter Modulus, pl = Menard's limit presssure)*

Experimental Survey

Survey instrumentation was installed after pile's construction (Fig. 12); it will permit the analysis of the load transfer phenomenon on pile caps and the verification of our settlements prediction.

This survey is composed by: Geodetect pressure cells and settlement plates; Geodetect is a geotextile-based monitoring system developed for the measurement of strain. It consists of a high strength geotextile, equipped with optic fibres connected to a monitoring device.

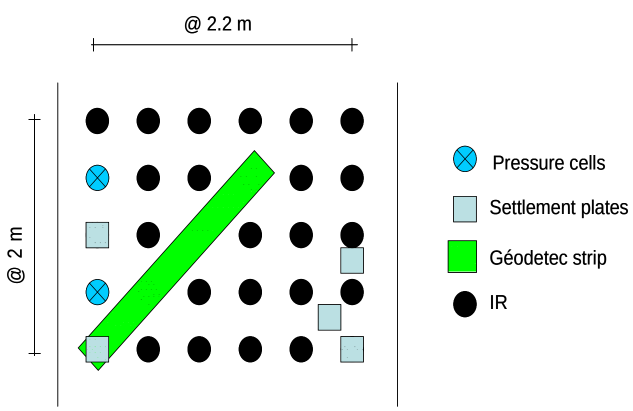



*Fig. 12. Situation of the surveillance instrumentation*

Numerical modeling

Introduction. Numerical analyses were performed to predict settlements and stresses on the system. Also it was important to evaluate the performance of two different materials proposed to constitute the LTP. First, a dense well graded gravelly sand and then a silt-cement treated mixture. Three dimensional and two dimensional axisymmetrical numerical analyses were performed using Flac3D and Flac computer programs (Itasca, 1993, 2002). Results were compared and discussed.

Behaviour modelling

The embankment fill and the substratum were modelled as linear elastic perfectly plastic constitutive material, with Mohr-Coulomb failure criteria. Soft soils were modelled with the Modified Cam Clay model.

The study was driven for a specific zone previously chosen to be instrumented to monitor the behaviour of the embankment; its profile is shown in Fig. 13.

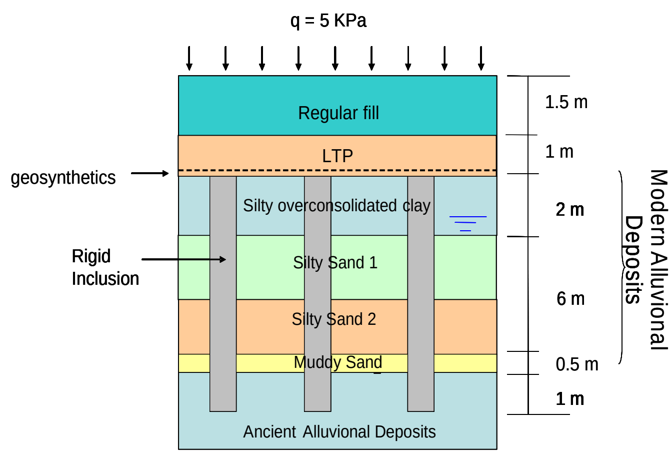



*Fig. 13. Studied case*

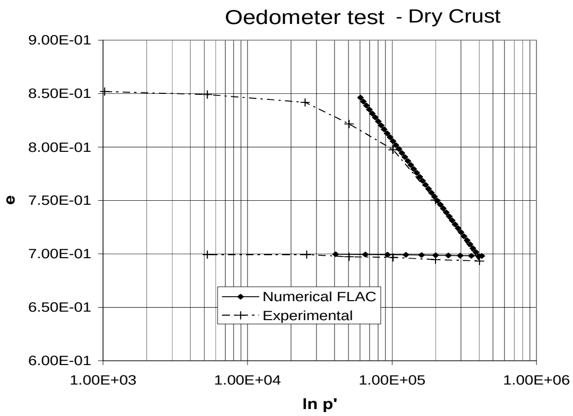
Parameters for the Mohr-Coulomb fill (regular and LTP) materials were chosen arbitrary based on the literature and empirical correlations (Table 4). Young modulus for the ancient alluvial deposit was obtained from the pressuremeter tests results (**Error! Reference source not found.**).

Cam Clay parameters where obtain by fitting laboratory oedometric tests to numerical simulations (Table 5).

Table 4. Mohr-Coulomb embankment's fill parameters

| | | C'(KPa) | Φ' | Ψ' | E (MPa) | γ (KN/m³) |
|---|---|---|---|---|---|---|
| Embankment | Granular LTP | 0 | 40 | 10 | 50 | 20 |
| | Cohesive LTP | 20 | 30 | 0 | 50 | 20 |
| | Regular fill | 5 | 30 | 0 | 30 | 20 |
| AAD | | 0 | 35 | 0 | 50 | 19 |

Table 5. Parameters fitting example and results.




| Parameters | | | | |
|---|---|---|---|---|
| | Dry Crust | Silty Sands 1 | Silty Sands 2 | Muddy Sand |
| σ'p (kPa) | 55 | 80 | 55 | 85 |
| σ'0 (kPa) | 25,5 | 48 | 52 | 82 |
| v | 1.85 | 1.97 | 1.97 | 4.31 |
| λ | 0,08 | 0,13 | 0,08 | 0,60 |
| vλ | 2.7 | 3.4 | 2.8 | 11.2 |
| φ' | 25 | 25 | 30 | 20 |
| M | 0.98 | 0.98 | 1.2 | 0.77 |
| K ($10^{-3}$) | 1.4 | 3.1 | 2.6 | 3 |
| γ ( $KN/m^3$) | 18.5 | 17 | 17 | 12 |

The numerical analysis included the substratum soil so strains under the inclusions are permitted. However some simplifications were made in our model:

- Setting up the piles by Driving Back Auger is not simulated. We do not take into account pile expansion due to this method.
- Geosynthetics were used on the project but omitted in our study. French experience has been based mostly on piled structural fills systems without geosynthetic reinforcements (Briançon, 2002 ).

Only the 1.9 meters square spaced inclusions configuration (pedestrian street) is presented with a 2.5 meters high embankment. The rigid piles were driven down to 9.5 meters in order to fully cross the 8.5 meters of compressible soils and to be embedded in the stiff gravelly sand layer. The concrete modulus was set to 7000 MPa, i.e. a low cement dosed formula.

The construction of the embankment was simulated by layers of 0.5 meters high each, at the end of its construction an overburden of 5KPa is added to its surface. The capping ration α defined as the proportion of the surface covered by the pile caps, is of 2.7% for this case.

Modelling results

First, a simulation without soil reinforcement was made; the calculated settlement at the interface between compressible soil and LTP was around 16 centimetres for both, 2D and 3D, calculations. This result justified a soil treatment to avoid damage to existing structures (pipelines, walls, etc.).

Next step was to simulate the soil behaviour with rigid piles. We studied the points were maximum settlement was expected to happen; in the middle of the square formed by four inclusions for the 3D case (3D B) and at the exterior limit for the axisymetrical model (2D B). Settlements over main axis were also followed, i.e. pile and maximums settlements axis. Fig. 14 shows the observed points and axis.

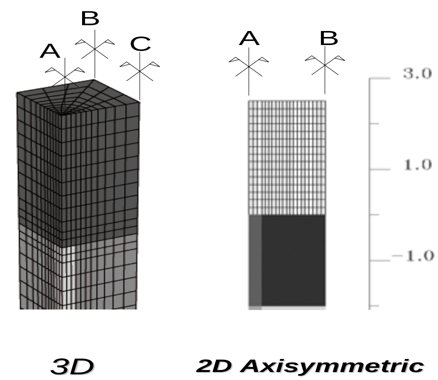



*Fig. 14. Studied points and axis for 3D and 2D Flac models.*

Fig. 15 shows settlement's evolution at surface due to embankment construction for both 2D and 3D models. It can be seen that main maximum settlements are divided by ten. For both cohesive and granular LTP two-dimension results are higher than the three-dimension configuration; 3.7% in the granular case and 5.9 % for the cohesive one.

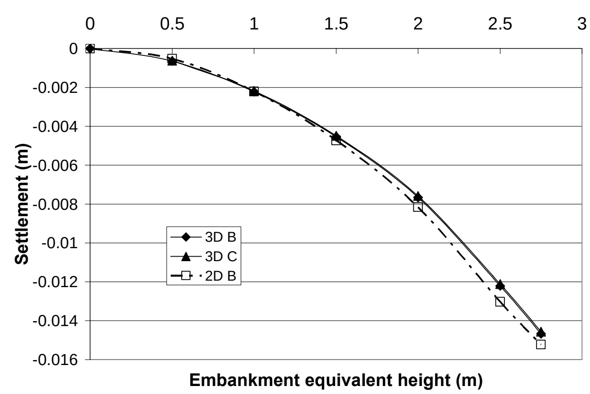



*Fig. 15. Settlement previsions 3D vs 2D axysimetrical. Granular LTP case.*

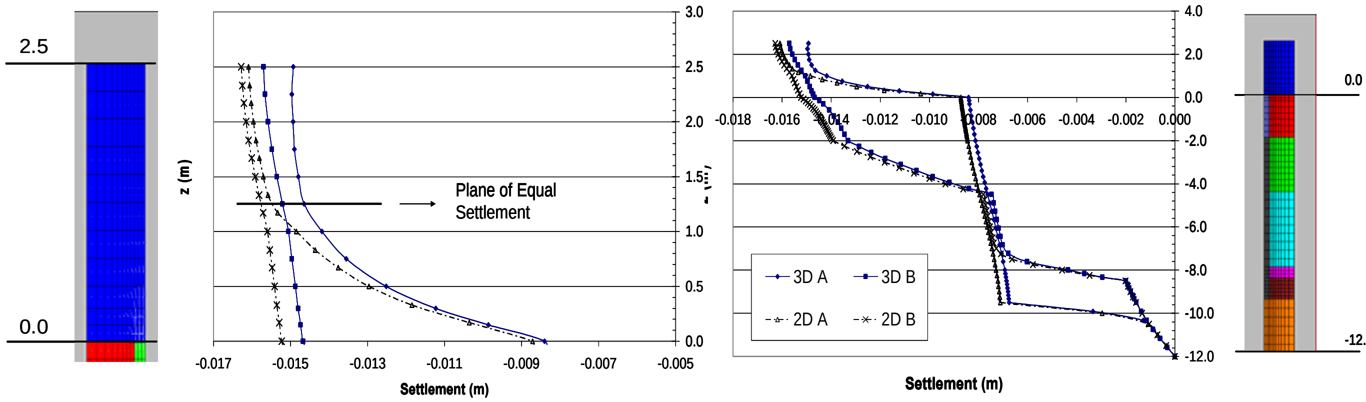



*Fig. 16. Final vertical displacements all along the rigid inclusion and maximum settlement's axis.*

Fig. 16 compares final vertical displacements along rigid pile and maximum settlement's axis for the 2D and 3D configuration. The upper plane of equal settlement defined as the plane from where differential settlements at the embankment surface are neglected is situated 1.25 meters above the ground surface, this is for the granular LTP case. For Coherent LTPs the equal settlement plane is situated at a lower level. The other plane of equal settlement, "the neutral point", defined as the plane where inclusions and compressible soil's settlement are similar is situated 6 meters under the ground surface. Until this limit negative friction is to be expected, under it positive friction is developed. At this level the inclusion is submitted to a maximum normal stress which has to be considered for the concrete's resistance design.

Efficacy is used to assess the degree of arching in the fill. The efficacy E of the pile support was defined by Hewlett & Randolph (1988) as the proportion of the mat weight carried by the piles (Eq. 1).

$$E = \frac{F_p}{W} = \frac{F_p}{A\gamma Hr} \qquad \text{Eq. 1}$$

Where $F_p$ is the load applied on a pile and $W$ the weight of the embankment's surface (A) covered by a single inclusion. This parameter has a value equal to the capping ratio α when there is no arching effect.

Fig 17 depicts efficacy evolution during embankments construction for both, granular or coherent fill. The graph highlights the significance of the coherent fill's efficacy in the first meter, about 85 % higher than for a granular fill at the first 50 centimetres and around 40% at 1 meter. After the first meter, efficacies become very similar. This confirms the important influence of cohesion (C') demonstrated by parametric works done by Jenck, 2005.

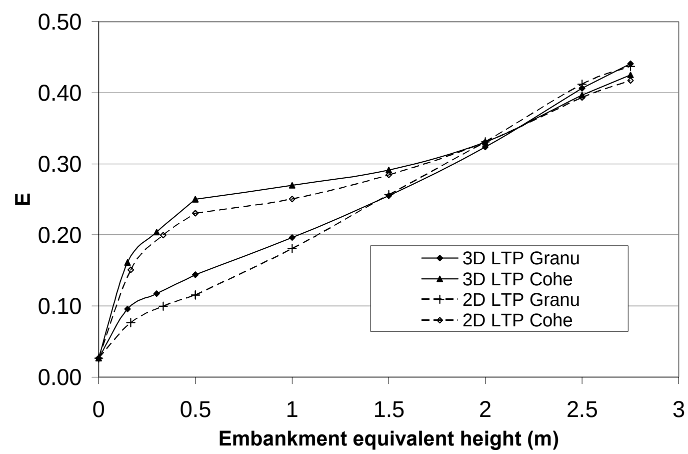



*Fig 17. Efficacy evolution for a granular and a cement treated fill. 3D and 2D results.*

<u>Influence of the substratum rigidity</u>

Several cases of embedment were tested to show the influence of the substratum by modifying the elastic properties of the Ancient Alluvial Deposits (AAD). The parametric study deals with the previous studied case; the numerical tests are presented in Table 6. The constitutive model used for the AAD was elastic linear perfectly plastic with a failure criteria of Mohr Coulomb type. The LTP used was the granular non cohesive one.

Results were compared for each studied case (Fig. 18). Obtained settlements are smaller for rigid bedrock than those resulting of a so called "floating inclusion" (E10 E50 & E∞). However, results shows a limit value when an increase of the bedrock stiffness does not affect the efficacy value. From this limit, and for this particular study, the embedding substratum seems not to be a very important parameter for the Efficacy. If we take the previous case E50 as a reference, the differences with the E10 and E∞ cases are about 30% and 1% respectively at the final state. The plane of equal settlement was also observed; significant variation was remarked between these cases: softer the substratum is, quicker the plane of equal settlements is obtained.

Table 6 Studied cases and parameters.

**Substratum properties**

| **Numerical test** | E (Young Modulus MPa) | ν | φ | C (KPa) | ψ |
|---|---|---|---|---|---|
| E10 | 10 | 0.3 | 35° | 0 | 0 |
| E30 | 30 | | | | |
| E50 | 50 (previously studied) | | | | |
| E∞ | Infinite (nodes fixed) | | | | |

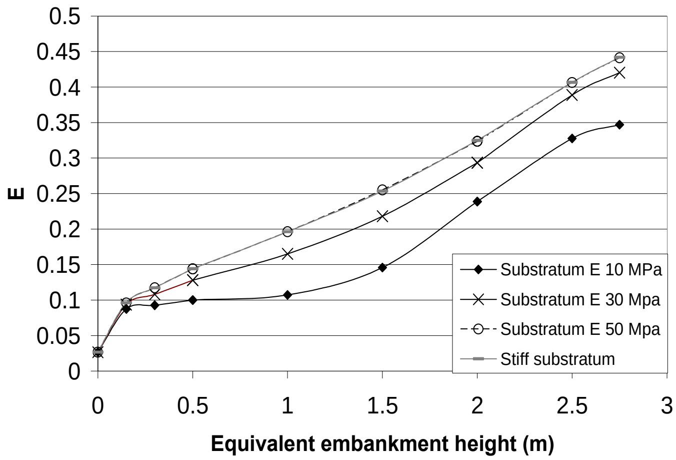



*Fig. 18. Influence of the bedrock stiffness on rigid inclusions performance.*

Table 7. Settlement variations due different bedrock stiffness

| | Max Settlement (mm) | Upper Equal settlement plane (m) |
|---|---|---|
| E10 | 38 | 0.6 |
| E30 | 20 | 1.0 |
| E50 | 15 | 1.25 |
| E∞ | 8 | 1.25 |

<u>Conclusions</u>

Carrieres sous Poissy case highlights the major influence of LTP and substratum mechanical characteristics on rigid piles technique performance.

Based on this study the coherent material was chosen to be used as LTP. Nowadays, experimental post project data is been gathered obviously to confront and validate, or not, our selections.

GENERAL CONCLUSIONS AND PERSPECTIVES

Rigid inclusions soil reinforcement technique was selected to treat alluvial soil intended to support high road embankments. Simulations and experience show that this technique will reduce settlements so that they will be safe for structures stability and performance (Simon, 2006).

Carrieres sous Poissy's case shows that two-dimensional axisymetrical model seems to be sufficient to make an accurate prediction of settlements and performance of the rigid inclusions soil reinforcement system. However, comparison between 2D and 3D was only made for a pile situated on the center of a pile network (symmetry hypothesis).

No conventional materials were used in both cases:

- Cement treated fill was selected to be used as the Load Transfer Platform in Carrieres sous Poissy case instead of a classical gravely platform. Economic reasons were the principal argument for this choice. Even if the final performance of both LTP materials was similar for this case, uncertainties exist on their Young modulus evolution due to loading, not represented here by a Mohr Coulomb criteria; Jenck (2005), demonstrated that non linear behaviour has significant importance in soil arching development.
- In France, geosynthetics are not used with the rigid pile's technique; it seems that this will not be an important point to assure load transfer between LTP and the piles.
- A low cement dosed concrete was used in this case (10MPa < Rc < 15 MPa).

The Rampe de Glain case is a common example of divergence between design methods; establishment of guidelines for this technique will treat the problem.

Load distribution between piles and soil matrix is still poorly understood. However, embankment mechanical parameters and its thickness, seems to be a critical factor on the design procedure of the rigid pile's technique (spacing, diameter, LTP thickness, geosynthetic reinforcement).

Experience has shown that the Driven Back Auger rigid piles and VCC systems have a positive influence in the performance of the column reinforced embankments, in this paper their effect was not consider but it began to be studied for further cases. Survey instrumentation was installed after rigid inclusion's construction; we will obtain experimental data soon that will be compared to numerical simulations.

Experimental data from both studied cases is still been gathered. Comparisons with parametric numerical studies will permit to better understand rigid pile's performance.

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