Use of the Menard Pressuremeter in the Quality Control of Stone Columns for an LNG Tank in South-East Asia.

Utilisation du pressiomètre pour le contrôle de colonnes ballastées utilisées en fondation d'un réservoir GNL en Asie du sud est.

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ABSTRACT: Quality control for stone columns often resorts to in-situ testing so as to check the continuity of such inclusions in the ground on the one hand, and their mechanical characteristics (friction angle, Young's modulus) on the other hand. Such control tests are frequently jeopardised by implementation difficulties: "refusal" to penetration or tendency for the testing probe to "walk out" of the stone column, for instance. The paper presents an example of the use of the Menard pressuremeter in the control of stone columns used as a ground improvement technique for the foundation of an LNG tank in South-East Asia. The difficulties encountered in the implementation of the tests and the solutions adopted to solve them are described. The results obtained are detailed, showing that the nature and the characteristics of the soils, the intensity of the compaction (linked to the amperage in the vibrator) and the Menard modulus measured at different levels of the stone column are interrelated. As a consequence it is shown that, rather than specifying a given Young's modulus to be achieved in the stone columns it is preferable to look for a minimum ratio between the modulus of the stone column (E_c) and that of the soil (E_s), since this ratio is the one that is used in the determination of the settlement reduction factor through the so-called "homogenisation method".

RÉSUMÉ : Le contrôle des colonnes ballastées fait fréquemment appel aux essais in-situ dans le but de vérifier, d'une part la continuité de ces éléments de renforcement du sol, et d'autre part leurs caractéristiques mécaniques (angle de frottement, module). La mise en œuvre des ces essais de contrôle se heurte souvent à des difficultés d'exécution : refus dans la colonne ou difficulté à « rester » dans la colonne et donc à vérifier leur partie inférieure, par exemple. L'article présente un exemple d'utilisation du pressiomètre Ménard pour le contrôle de colonnes ballastées utilisées en fondation d'un réservoir GNL en Asie du sud est. Les difficultés rencontrées dans l'exécution des essais et les solutions adoptées pour les pallier sont décrites. Les résultats obtenus sont détaillés en montrant que nature et caractéristiques des terrains, intensité du compactage (liée à l'intensité dans le moteur du vibreur) et module pressiométrique mesuré dans la colonne ou dans les différents niveaux de la colonne sont des paramètres interdépendants. Par voie de conséquence, il est démontré que plutôt que de spécifier un module d'Young donné à atteindre pour les colonnes ballastées il est plus judicieux de rechercher un ratio minimum entre le module de la colonne (E_c) et celui du sol (E_s), ratio qui intervient dans la détermination du facteur de réduction des tassements par la méthode dite d'homogénéisation.

KEYWORDS: Ground Improvement, Stone columns, Quality assurance, Quality control.

1 INTRODUCTION.

Stone columns are commonly used to improve bearing capacity, reduce settlements, accelerate primary consolidation, mitigate the risk of liquefaction in seismically prone areas, replace conventional deep foundation systems, etc.

They consist in installing and compacting coarse granular material into soft or loose ground under the form of cylindrical elements with a diameter that can be variable over the depth depending on the density or stiffness of the existing ground. The vertical elements are arranged in a triangular or square pattern. The diameter of the stone column, the size and shape of the treatment grid determine the inclusion factor (also called "replacement ratio"):

$$a = A_c/A \quad (1)$$

Where:
A is the tributary area of a stone column,
A_c is the cross section of a stone column.

Quality assurance and control (QA/QC) are firstly made through digital recordings of the stone columns construction parameters (Figure 1):

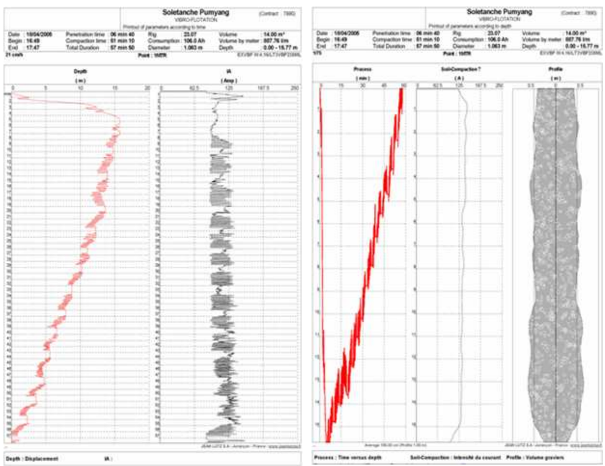

Figure 1. Digital recordings for stone columns.

Although such recordings may be considered as generally self-sufficient for quality control, the technical specifications of a given project or the local standards of a given country may claim for further investigations and checks, such as:

1.1 Excavation of the top of some stone columns to check the diameter achieved (Figure 2)

Figure 2. Measurement of the diameter of a stone column close to ground surface.

1.2 Load tests on one or more stone columns (Figure 3)

Figure 3. Load test on a single stone column.

1.3 In-situ testing in the stone columns

This is usually done with static cone penetration tests, dynamic cone penetration tests or Menard pressuremeter tests, with the following tentative goals: check the continuity of the stone column, making sure there is no necking or interruption in the stone column, and/or check the mechanical characteristics of the stone columns. Since many stone column projects calculation methods refer to the modulus of the stone column (E_c) the purpose of stone column in-situ testing will be to try and measure E_c through the penetration resistance qc or qd or the Menard pressuremeter p_L and E_M.

Problems are however very frequently encountered in the implementation of such tests: refusal of the probe may take place at shallow depth in the highly compacted stones; the probe may not remain vertical in the stone column or the stone column may have not remained vertical during construction; the stone column can then only be tested over a short length, while the bottom part shows the resistance of the ground in the vicinity of the stone column; when hitting highly compacted stones the testing probe may be diverted from its normal course and "walk out" of the stone column; one may not succeed in obtaining the desired / specified penetration resistance or limit pressure; this often occurs in very soft ground.

Sanglerat (2000) gives examples of the use of a static-dynamic cone penetration apparatus to test stone columns. His second example refers to a site where 4.5 metres of loose silts overlie dense gravels and stiff marls and where 5 to 6 metre long stone columns were built. Figure 4 shows the pre-treatment cone resistance (top) and the cone resistance in the stone column (bottom), perfectly in line with the French regulations which stipulate that a minimum value of 10 MPa should be achieved.

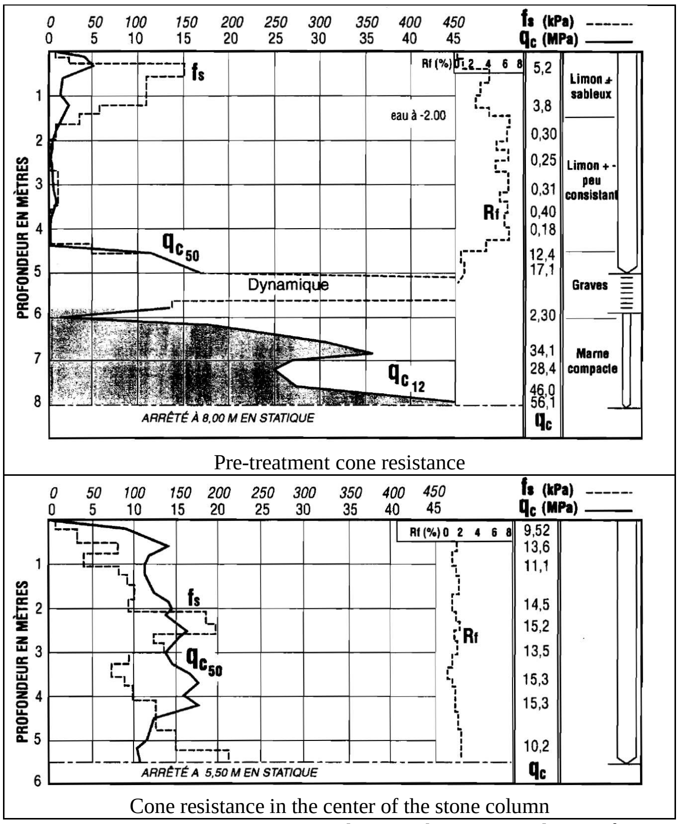

Figure 4. Cone penetration tests results in a short stone column, after (Sanglerat 2000).

His third example shows a case where some 11 m of loose silts are overlain by 5 metres of sand. Figure 5 shows the pre-treatment cone resistance (left) and the cone resistance in the stone column (right). In this case, the cone resistance in the stone column satisfies the regulation ($q_c >= 10$ MPa) from ground level to 7 metres depth and does not in the major part of the silt layer below: either the cone probe walked progressively out of the stone column between 4.5 and 7m depth (the cone resistance decreases in a gradual way) or the stone column becomes softer and does not satisfy the regulation. In the absence of friction ratio measurements in the stone column with the static-dynamic penetrometer used the doubt cannot be dispelled.

One shall however note that between 10 and 12m depth:

$q_c = 0.5$ MPa in the virgin ground,

hence $E_s = 2.5 \times q_c = 1.25$ MPa

and $q_c = 2.7$ MPa in what may or may not be the stone column, hence $E_c = 3.8 \times q_c = 10$ MPa

which would lead to $E_c / E_s = 10 / 1.25 = 8$

Such a ratio is in the usual range for stone columns (see § 3.1.1 below) and the cone may have actually stayed in the stone column.

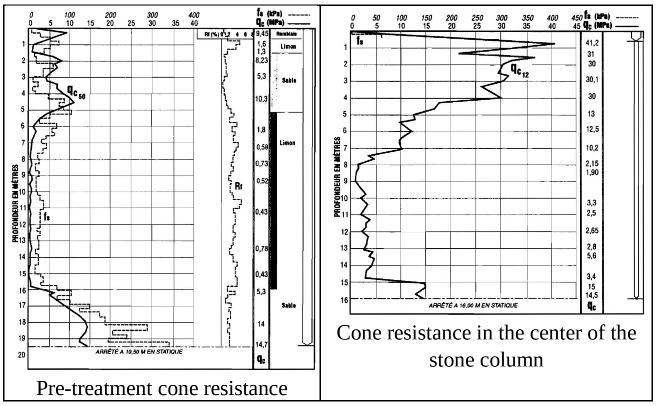

Figure 5. Cone penetration tests results in a long stone column, after (Sanglerat 2000).

2 LNG TANK EXAMPLE

2.1 General description of the site

The 80 m diameter tank was to be built on the following soil characteristics:

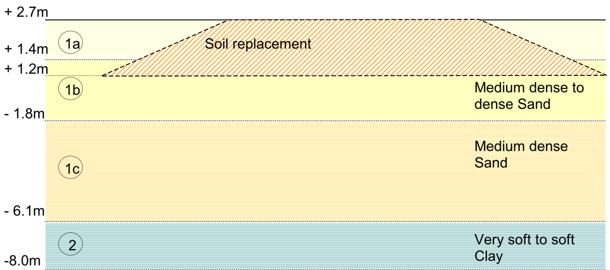

Figure 6. General soil profile at the new tank location.

The characteristics of the above layers are summarised in Table 1:

Table 1. Soil characteristics.

Layer	*N (SPT)*	*q_c (MPa)*
1 – Sand	15	10
2 – Clay	0 to 4	0.5 to 1

The sand layer was quite homogeneous according to the SPT and CPT results. However, for the clay layer, 2 zones were singled out due to the earlier existence of a former tank, the centre of which was offset from the new tank axis. This former tank applied some 150 kPa on the ground and part of the soft clay had therefore already consolidated under this load.

2.2 Description of the ground improvement works

Ground improvement was performed using the wet top feed stone column technique. The columns were built with two different diameters: 1,200 mm in the clay layer and 600 mm in the sand layer, as shown on the cross section below:

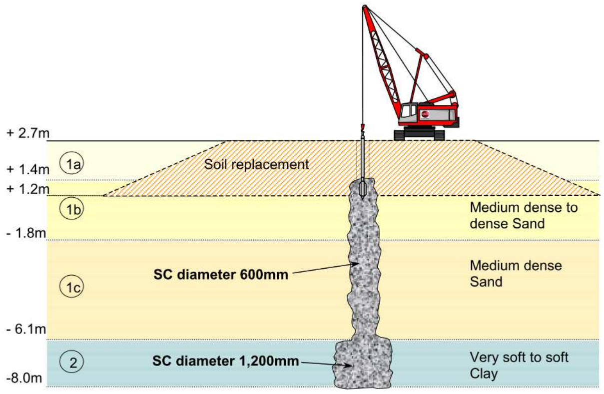

Figure 7. Cross section of the ground conditions with improvement.

The tank dead loads are detailed below:

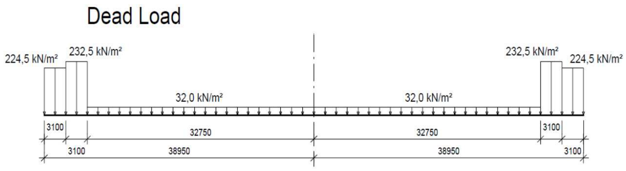

Figure 8. Tank geometry with dead load at the end of construction.

As far as live loads are concerned the most critical stage is during the hydrotest. The ground improvement was therefore designed for it, as shown below:

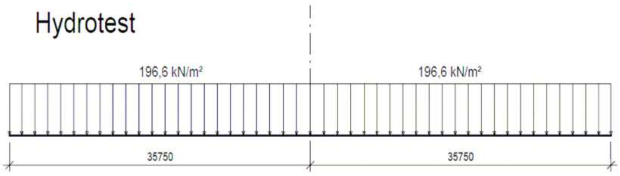

Figure 9. Live load during the hydrotest.

2.3 Testing programme and the corresponding specifications

In addition to the digital recordings of the construction parameters for each and every stone columns plate load tests (PLT) were specified on top of some columns and pressuremeter tests were specified to be done in a certain number of columns. The target for PMT was a limit pressure p_L of 1.5 MPa and a Menard modulus E_M of 15 MPa. These values were proposed based on the French « Recommandations sur la conception, le calcul, l'exécution et le contrôle des colonnes ballastées sous bâtiments et ouvrages sensibles au tassement ».

2.4 First results, problems associated

The PMT results obtained in the columns were homogeneous and satisfactory down to some 5 m depth. Further down the results were variable and, in the clay layer, the characteristics measured were simply the same as those of the unimproved soil. Figures 10 and 10bis show the comparison between PMT results in the natural soil and in a would-be stone column centre:

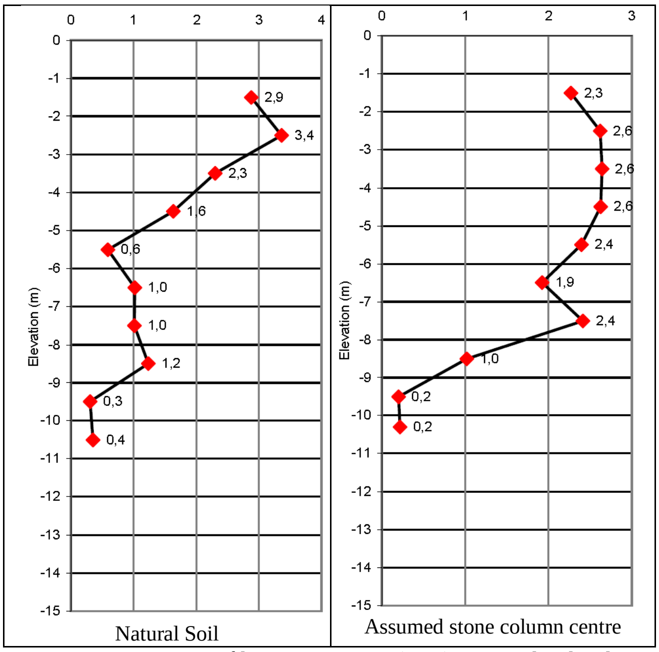

Figure 10. Comparison of limit pressures p_L (MPa) in natural soil and in stone column.

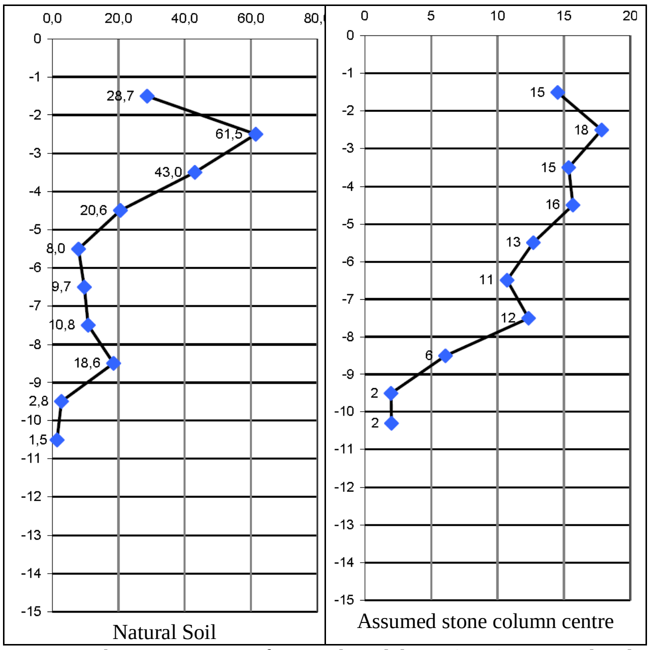

Figure 10 bis. Comparison of Menard moduli E_M (MPa) in natural soil and in the assumed stone column centre.

The high limit pressures and Menard moduli in the top part of the natural ground (replacement sand fill) are due to the fill placement method, with the use of heavy machinery. In this part of the profile the stone column is very dense (p_L in excess of 25 MPa).

In the medium dense natural sand below there is a gradual decrease of the characteristics of what is believed to be the stone column and at clay level the last two readings are no different from the virgin clay characteristics.

Since the digital recordings showed that stones were installed in large quantities and to large compaction intensities in the clay layer, there is a clear discrepancy between the two and one can suspect that the pressuremeter probe walked out of the stone column.

2.5 Solution adopted to settle the issue

As the probe walked out of the column in the sand layer, “spare” stone columns were installed in the clay layer, only followed by shear backfilling with sand over the sand layer height. The sand backfill was left uncompacted to make sure that the pressuremeter probe would remain at column location over the full depth.

On the other hand, to test the improved sand layer PMTs were carried out in between 4 stone columns (at the so-called weakest point, the location further away from any column).

3 ANALYSIS OF THE RESULTS OBTAINED

3.1 Inside the stone columns

3.1.1 *Within the clay layer*

A typical result in a “spare” stone column aiming at testing the column within the clay layer was as shown below:

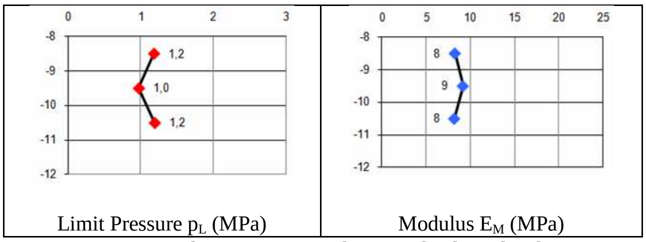

Figure 11. PMT results in a “spare” column at clay layer level.

For such tests, the limit pressure p_L was around 1.1 MPa and the Menard modulus E_M around 8.5 MPa. Although clearly higher than the values in the virgin clay such results fell short of the specified p_L = 1.5 MPa and E_M = 15 MPa.

However an E_M value of 8.5 MPa translates into a Young’s modulus of 8.5 / α = 34 MPa, with α = 0.25 in the stone column gravel, i.e. some 11 times the Young’s modulus of the virgin clay layer (3 MPa).

Such a ratio is higher than what is usually taken into account for such comparisons, e.g. between 6 and 10, the reason for this being the very high compaction intensity (180A) that was used for the construction of the stone column in the clay layer.

3.1.2 *Within the sand layer*

Within the sand layer, the results were as shown below:

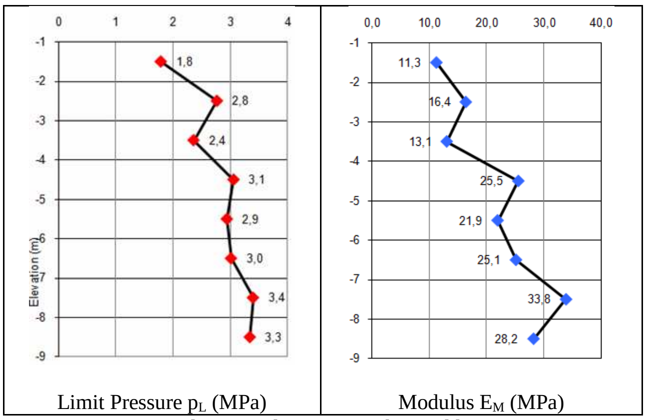

Figure 12. PMT results in a column to test the sand layer.

For such tests, p_L was around 3 MPa and E_M around 25 MPa, thus a Young's modulus E_y of 100 MPa, 3 to 5 times the Young's modulus of the original sand layer (20 to 30 MPa).

3.2 In between the stone columns in the sand layer

In between the stone columns in the sand layer, the results were as shown below (Figure 13):

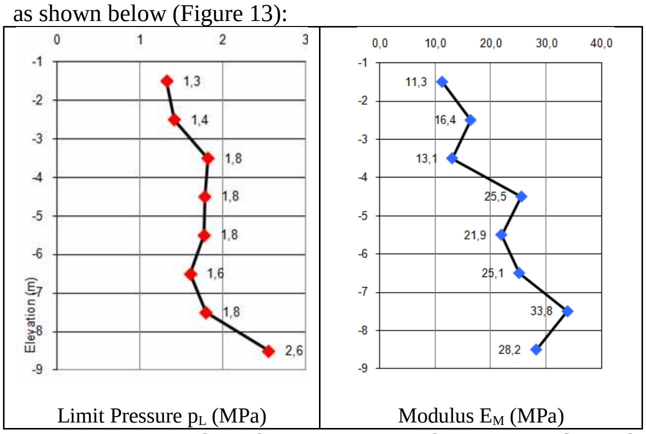

Figure 13. PMT results in between 4 stone columns to test the sand layer.

For such tests, p_L was around 1.8 MPa and E_M around 25 MPa, thus a Young's modulus of 75 MPa (α = 0.33 for sand), 3 to 4 times the Young's modulus of the original sand layer (20 MPa) and actually quite close to the modulus of the stone columns (Figure 10bis), which means that the Client's design with stone columns in the sand layer could have been changed into stone stone columns in the clay layer only and pure vibrocompaction above.

3.3 Interdependence of the various factors

From the above one can see that the nature and the characteristics of the soils, the intensity of the compaction (linked to the amperage in the vibrator) and the Menard modulus measured at different levels of the stone column are interrelated:

- in sand quite high characteristics can be achieved but the modulus of the stone column will not exceed 3 to 5 times the original virgin ground values
- in soft clay the apparently rather low values achieved are actually up to 6 to 12 times the ones of the unimproved ground
- in both cases, medium dense granular soil or soft clay, the ratio between the modulus of the stone column and that of the virgin ground will also be a function of the compactive effort applied.

4 CONSEQUENCES FOR FUTURE SITES

In soft ground, rather than specifying a given Young's modulus to be achieved in the stone columns, it is preferable to look for a minimum ratio between the modulus of the stone column (E_c) and that of the soil (E_s). This ratio is the one that is used in the determination of the settlement reduction factor using the so-called "homogenisation method".

4.1 The "homogenisation method"

The determination of the quantities of stones to be installed and compacted to reach the required final improvement is based on worldwide known and accepted calculation methods.

Such methods are plenty but two are more widely used:

- the "homogenisation method" (Blondeau et Dhouib, 2005)
- the Priebe method (Priebe, 1995)

Both methods yield a settlement reduction factor "n" equal to the ratio of the settlements of the untreated ground to the settlements of the improved ground.

In the particular case of loaded zones with large dimensions the settlement reduction factor is equal to the ratio of the stiffness modulus of the improved ground to the stiffness modulus of the untreated ground.

Furthermore for heavy loads or large depths in the ground the two methods are linked and are actually equivalent in the sense that Priebe's "n_2" settlement reduction factor is "capped" by the settlement reduction factor derived from the "homogenisation method".

$$n_{\max} = 1 + \frac{A_C}{A} \cdot \left(\frac{E_{Column}}{E_{Soil}} - 1 \right) \qquad (2)$$

The equivalent homogenised ground modulus can then be calculated (Eq. 3):

$$E_{equivalent} = a \cdot E_{column} + (1-a) \cdot E_{soil} \qquad (3)$$

with:

$E_{equivalent}$: stiffness modulus of the improved ground body,

E_{column}: stiffness modulus of the compacted stone column,

E_{soil}: stiffness modulus of the surrounding ground, possibly improved by the vibrocompaction process and the stone column installation process,

"a": inclusion factor, as defined previously (Eq. 1).

The ratio between the modulus of the column and the modulus of the surrounding soil, as based on experience, usually ranges from 8 to 10 in compressible material.

According to the results presented above, the ratios E_c/E_s are around 3 in sand and 11 in clay as detailed in Table 2:

Table 2. Ec/Es ratios.

	Natural soil		*Improved soil*		*Ratio*
	E_M (MPa)	*E_s (MPa)*	*E_M (MPa)*	*E_c (MPa)*	*E_c/E_s*
Sand	10	20 to 30	25	100	3 to 5
Clay	2	3	8.5	34	11

4.2 Comments

As shown above the important parameters in the calculation of a stone column project are the inclusion factor "a" and the modulus ratio E_c/E_s.

As a consequence the modulus of the stone column should not be specified to a certain absolute value, a value that may actually never be reached in soft ground. All that needs to be specified is a minimum modulus ratio (6 to 8 for example).

Debats et al. (2013) back-analyzed the settlements measured in the preloading phase of large oil tank foundations in Panama, showing the perfect applicability of the "homogenisation method" to a stone-column improved ground when an average modulus ratio of 8 is used.

In other terms, standards or regulations in which a minimum cone resistance or Menard limit pressure is specified to be reached in the centre of the stone columns may prove inapplicable to soft soil conditions.

5 CONCLUSION

An example of the use of the Menard pressuremeter in the control of stone columns as a ground improvement technique for the foundation of an LNG tank in South-East Asia was presented together with the difficulties encountered in the implementation of the tests and the solutions adopted to resolve the issues. The results obtained show that the nature and the characteristics of the soils, the intensity of the compaction and the Menard modulus measured at different levels of the stone column are interrelated. As a consequence it is shown that, rather than specifying a given Young's modulus to be achieved in the stone columns it is preferable to look for a minimum ratio between the modulus of the stone column (Ec) and that of the soil (Es), since this ratio is the one used in the determination of the settlement reduction factor through the so-called "homogenisation method".

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