

Soil reinforcement strategy for an EPR2 nuclear power plant project in northern France: contribution of back-analysis

Minh Tuan Hoang, Martin Cahn, Jesús Pérez Herreros, Fahd Cuira
Terrasol Setec, France, martin.cahn@setec.com

Van Son Vo¹, Cyril Simon²
¹Edvance, France. ²EDF TEGG, France

ABSTRACT: The construction of two EPR2 reactors, each with a capacity of 1670 MW, is planned in northern France near an existing 900 MW nuclear power plant. The site is characterized by the presence of a thick layer of deformable soil. Compared to the existing plant, the EPR2 installations have significantly greater mass and surface area. Without adequate ground reinforcement, the site's geotechnical context and settlement monitoring from the existing plant raise concerns about potential significant settlements during EPR2 construction and over time. This paper presents the back-analysis of the settlements observed at the existing plant, which enabled the refinement of the local geotechnical model, and outlines the chosen soil reinforcement strategy.

KEYWORDS: Soil reinforcement, EPR2, back analysis, Flanders clay, rigid inclusions.

1 INTRODUCTION

In northern France, immediately adjacent to an existing 900 MW Nuclear Power Plant comprising six reactors, the construction of two EPR units with a capacity of 1670 MW each is being considered.

The site is characterized by soft soils layers, with near-surface Flandrian deposits from the late Quaternary period, approximately 30 m thick, overlying the Flanders Clay layer, whose thickness is estimated at about 100 m. Given the nature of the ground, building an EPR2 unit on this site presents a significant challenge, as substantial absolute and differential settlements are expected. Historical measurements at the existing 900 MW units have shown settlements of 25 to 30 cm; however, the EPR2 structures are expected to impose significantly greater loads over a much larger footprint, thereby amplifying the settlement concerns and the associated design complexities.

This study reports on the back-analysis of settlements recorded at the existing plant, which allowed refinement of the local geotechnical model and guided the definition of the soil reinforcement strategy.

2 GEOTECHNICAL MODEL

2.1 Geological context

During the feasibility study phase, the geotechnical model was initially based on the re-evaluation of various in-situ and laboratory tests carried out mainly at the existing power plant site. The analysis was supplemented by a comparison with geotechnical investigation carried out for other infrastructure projects near the future power plants.

The available investigations in the Flanders sands and silts allow the distinction of four horizons:

- **The Flandrian Sands (N1 to N3)**, about 30 m thick. These sands exhibit a high degree of heterogeneity and may include a silty interlayer (N2–N3) of variable thickness, whose strength is incompatible with the loads imposed by the EPR2 facilities;
- **The Flanders Clays**, approximately 100 m thick. This regionally recognized formation consists of an overconsolidated, low-permeability plastic clay. The geological history of the site, together with the available geotechnical investigations, indicate that the upper part of this layer (about 10 to 20 m) may be altered. Below 20 m depth, the behavior of this formation can reasonably be

assimilated to that of a homogeneous elastic medium whose stiffness gradually increases with depth. The preconsolidation increment is estimated at 2.5–3.0 MPa (this value, however, cannot be guaranteed within the upper altered zone);

- **The Ostricourt Sands and the Louvil Clays**, whose deformability is negligible compared to that of the Flanders Clays.

The following Figure 1 presents the geotechnical model adopted for the study of reinforcement design.

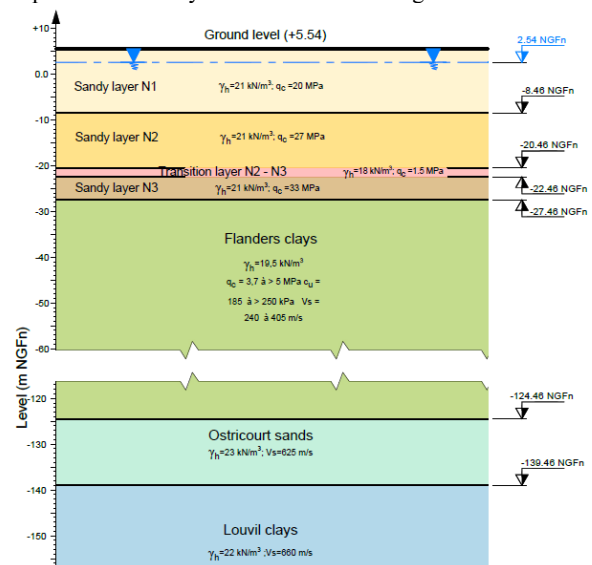


Figure 1. Geotechnical profile of the project

The groundwater table within the sands was observed at elevation +2.54 NGF.

2.2 Characterization of the Flandrian Sands Layers

More than 150 CPT from the site of the existing power plant were analyzed. Evaluation of the q_c values shows that, although some heterogeneity can be identified, levels N1, N2, and N3 exhibit similar average characteristics. The N2 level has better properties than N1 and characteristics comparable to those of the N3 horizon.

The N2–N3 transition layer was identified in each area. Its properties are clearly inferior to those of the adjacent horizons. This horizon appears to be of limited thickness, and its elevation can vary by up to ± 1 m. As illustrated in Figure 2 significant

dispersion in q_c values is observed due to the presence of sandy beds within this layer; however, regardless of the location, it exhibits relatively poor resistance.

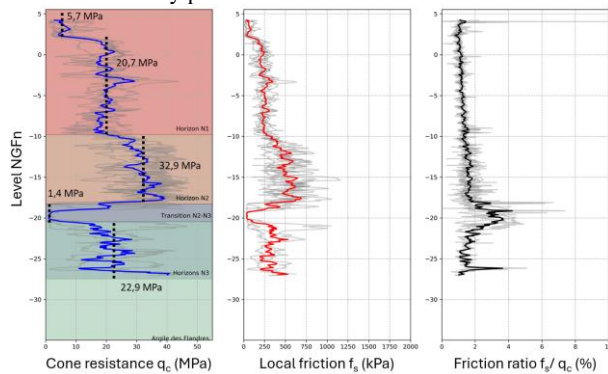


Figure 2. Typical CPT profiles in Flandrian sand

The available pressuremeter tests corroborate well the CPT results, and it was established that a correlation between E_M and q_c could be established with $E_M \sim q_c$ in the Flandrian Sands and for the N2-N3 transition layer $E_M \sim 1$ to $3 q_c$.

The available identification tests also allowed to clearly identify the N2–N3 transition layer, which is characterized by a fines content exceeding 35% and a water content of up to 45% (with an average of 25%, compared with 20% for the other sandy horizons).

Laboratory tests also made it possible to characterize the shear strength of all the sand horizons, while compressibility indices $C_s/(1+e_0)$ ranging between 0.08 and 0.10, depending on water content (15% to 46%), were measured in the N2–N3 transition layer. In this layer, the vertical consolidation coefficients C_v range from 2×10^{-9} m²/s to 3×10^{-7} m²/s, depending on whether the material is more clayey or sandy. The creep coefficient C_α can be estimated from the time–settlement curves obtained during compressibility tests, beyond the in-situ effective vertical stress, the values range between 5×10^{-4} and 5×10^{-3} , increasing with the clay content of the tested material.

2.3 Characterization of the Flanders Clay

The Flanders Clay, of Ypresian age, is an overconsolidated, highly plastic, very low-permeability clay, homogeneous over a considerable thickness (up to 100 m). It is described as a brown to gray clay, possibly containing sandy and/or silty beds.

The formation appears fissured, with slickensided surfaces observed in core samples, the frequency and spacing of these discontinuities varying laterally. It serves as a regional marker layer. The results of the site-specific investigations were thus supplemented with data from other infrastructure projects in the region and from the literature, notably from the work of Josseume (1998). Josseume confirmed the very high overconsolidation of the clay through high-pressure tests, with an overconsolidation pressure of approximately 3 MPa at the top of the layer. This value is consistent with the estimated elevation of the natural ground surface during the Pliocene (about 200 m above the current top of the clay). Therefore, for the stress levels induced by the EPR2 facilities (up to 600 kPa at the surface), only the compressibility characteristics of the clay within its overconsolidated domain are of interest.

The following Figure 3 presents a compilation of various laboratory tests illustrating the variation of natural water content with depth within the Flanders Clay. The results highlight a decrease in water content with depth, indicating an increase in density, which parallels the decrease in the recompression factor $C_s/(1+e_0)$. The recompression indices are likely overestimated due to the poor quality of the “intact” samples, which exhibited numerous open fissures.

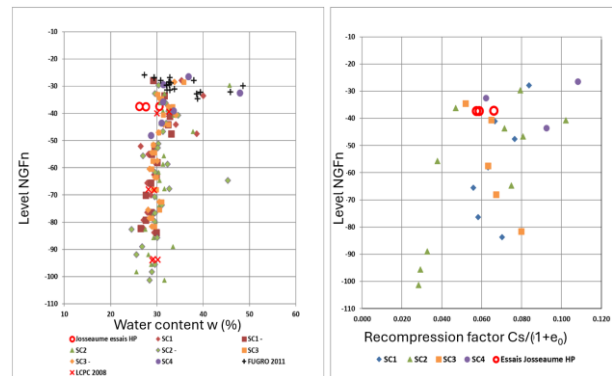


Figure 3. Flanders Clay – Water Content and Recompression Factor

The tests conducted by Josseume in the upper part of the layer (10 m below the top of the clay) yielded a $C_s/(1+e_0)$ ratio close to 0.06. It is important to emphasize the strain range to which these values correspond, namely about 1% to 5%. This link with the strain range is essential for assessing whether these values are appropriate for estimating the settlement of a structure. Based on the analysis of settlement measurements on the existing nuclear power plant, an envelope value of $C_s/(1+e_0) = 0.035$ can be defined for a strain level less than or equal to 0.5%. Oedometer creep tests were carried out at stress levels from σ'_{v0} to $2 \times \sigma'_{v0}$ and yielding creep coefficient C_α values from 4×10^{-4} to 2×10^{-3} . These low values are associated with the overconsolidated domain of the material.

The undrained shear strength C_u values measured in triaxial tests indicate a significant increase of C_u with depth with values ranging from about 120 kPa in the upper part to more than 300 kPa in the lower part. This increase in strength with depth is well correlated with the available deep CPT results. The transition between the N3 sands and the Flanders Clay is very distinct, characterized by a decrease in q_c and an increase in the friction ratio R_f . The average q_c measured over the first few meters ranges between 2 and 4 MPa. A friction ratio of 5% is observed in this layer, indicative of a highly clayey material.

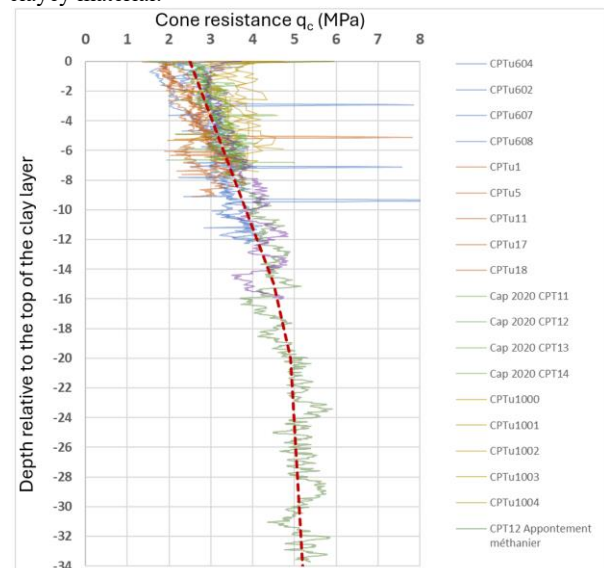


Figure 4. CPT profiles in Flanders Clays

The CPT profiles presented in Figure 4 highlight a linear increase of q_c with depth over the first 20 meters. This behavior reflects the potentially altered nature of the upper fringe of the Flanders Clays. Beyond 20 m depth, the CPT measurements appear to indicate a stabilization of the measured resistance,

which is more consistent with the strongly consolidated character of this horizon. This trend is fairly consistent with the variations observed in wave propagation velocities which indicate a shear modulus G_0 (at very small strain levels) ranging from 95 MPa to 220 MPa, corresponding to a gradient of approximately 2.5 MPa/m.

3 ANALYSIS OF SETTLEMENT DATA FROM THE EXISTING POWER PLANT

3.1 Analysis of available data

The existing Nuclear Power Plant consists of 6 nuclear reactors units with a nameplate capacity of 900 MW each. The first pair of reactors entered service in 1980, the second in 1981, and the last in 1985. Monitoring of measurements across all buildings indicates a significant variation from one unit to another, mainly in terms of settlements recorded at the end of construction. This variability can be attributed, on the one hand, to the spatial heterogeneity of the Flandrian sands (which contribute significantly to the magnitude of settlements), and on the other hand to the phased nature of the construction works, which may have a non-negligible impact on the magnitude of "delayed" settlements observed (resulting both from the consolidation mechanisms of the foundation soils and from interaction effects with neighbouring loads applied after construction).

The back analysis presented in this article focuses on the last pair (units 5 and 6) for which the largest number of measurements are available.

The plants are founded on a general raft foundation (1.2 m to 4 m thick), reinforced with a ternary slab. This slab serves to homogenize settlements and to purge the loosest sand layers encountered beneath the foundation. The net loads are about 400 kPa under the heaviest buildings and 150 kPa under the lighter ones.

For monitoring settlement, about 100 sensors were installed around each pair during construction. The settlement evolution shows an immediate settlement occurring during the construction and a deferred settlement developing over several years. As illustrated in Figure 5, under reactor building 5, 12 sensors were deployed, revealing a maximum observed settlement of 30 cm in 2018 (30 years after construction), about 20% of which occurred during a consolidation phase. The deferred settlement is about 1.5 to 2.0 mm per year over 30 years, reducing to around 1 mm per year in recent measurement. A logarithmic equation, $s(t) = a \ln(t) + b$ (with t in days) can be used to approximate the evolution of deferred settlement and to estimate the long-term settlement. This approximation leads to an additional settlement of 2 to 3 cm over 50 years.

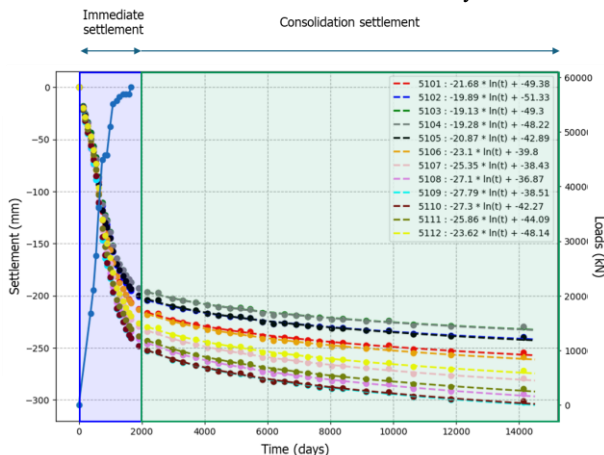


Figure 5. Measured settlement under reactor building 5

Settlement distribution with depth measured by multi-tassometers under reactor building indicated that under 85% of construction, settlement in the Flandrian sands was about 11 cm. Linear extrapolation suggests approximately 13 cm in Flandrian sands at the end of construction. Given a total settlement of about 21 cm at that time, an immediate settlement of around 8 cm occurred in the Flanders clay. Assuming consolidation settlement primarily occurs in Flanders clay, the long-term settlement at 50 years in this layer is estimated at 17 cm. This leads to a distribution where short-term settlement (21 cm) is 60% in sands and 40% in clays, while long-term settlement (30 cm) is 40% in sands and 60% in clays.

3.2 Qualitative interpretation of measured settlements

The deferred settlement observed are mainly attributed to the clayey layers: N2-N3 transition and Flanders clay.

For the N2-N3 layer, compressibility tests indicate a coefficient of vertical consolidation (C_v) of approximately 8×10^{-8} m²/s. With a drainage path of 1 m, 90% consolidation occurs in about 4 months, suggesting that this layer behaves as drained and its contribution to the consolidation settlement is negligible. However, its creep coefficient (C_a) of 0.5 to 5×10^{-3} can result in 0.8 to 8 mm of creep settlement over 30 years. This represents 2% to 15% of the total 5 to 6 cm of consolidation settlement observed over the same period. Consequently, more than 85% of the consolidation settlement is attributed to the Flanders Clay.

For the Flanders Clay, with a vertical consolidation coefficient (C_v) of about 1.4×10^{-6} m²/s, 90% consolidation time is estimated at 50 years for a 50 m drainage path. Considering anisotropy with $C_h = 2 \times C_v$, this time reduces to approximately 30 years. These results indicate that consolidation of the Flanders Clay under the existing plant takes 30 to 50 years, which should result in an "inflection" in the settlement curve. Since such an inflection is not evident in the available monitoring data, it suggests that the observed consolidation settlements are primarily due to the primary consolidation mechanism, whose completion is likely masked by progressive development of creep mechanism (secondary consolidation) which most likely occurs within the upper fringe of the clay layer. Creep tests on the Flanders Clay indicate a creep coefficient (C_a) of approximately 0.001. Assuming creep initiates 30 years after construction, it would produce about 1 cm of settlement over 10 years, corresponding to an average rate of 1 mm/year. This magnitude aligns with currently observed rates, indicating that creep is expected to primarily govern the future evolution of deferred settlements under the plant.

The measured settlements, ranging from 15 to 25 cm in the short term and 20 to 30 cm in the long term, correspond to average strains of 0.1% to 0.2% in the short term and 0.2% to 0.25% in the long term, relative to the total 120 m soil thickness. These strain levels typically correspond to a generally elastic soil response. However, the strain rates in the N2-N3 transition layer suggest behavior potentially beyond the elastic range. For the Flanders Clays, the long-term settlement of 17 cm over a 97 m thickness results in an average strain of 0.17%. This strain value is, however, higher in the upper part of the layer, particularly within the altered fringe.

3.3 Back-analysis

A back-analysis was conducted with the aim of proposing a calculation model that accurately reproduces the observed settlements, both in terms of amplitude and depth distribution. Various models were examined, including linear and non-linear elastic behaviors.

Methods described as *hybrid*, in the sense that they combine predefined analytical solutions for the supporting soil with numerical solutions for the foundation, have been employed. (Cuira *et al.*, 2008 and Hoang *et al.*, 2020). These models have been used to generate the soil flexibility matrix, which served as input for numerical analysis where the raft foundations were discretized into finite element as illustrated on the Figure 6.

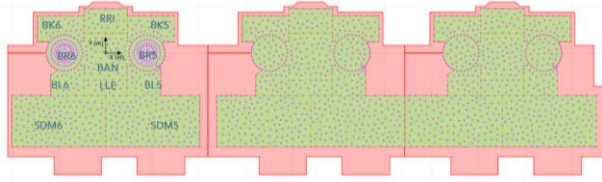


Figure 6. Hybrid model for 6 reactors of units

For the back-calibration, modulus of the Flandrian sands were assumed to follow a relation $E = \beta_s E_M$ (where E_M is the pressuremeter modulus), while in the Flanders clays, $E = \beta_a G_0$ (where G_0 is the maximum shear modulus). The values of β_s and β_a were then adjusted on settlement measures for each building using a least-squares method. This calibration provided Young's modulus for each layer and each building, which can also be analyzed in relation to the corresponding strain level.

From this calibration, a correlation between modulus and strain was established for each layer. For the Flandrian sands, the results allow the definition of an average Young's modulus per layer, valid for a strain range of 0.4% to 2%. The Figure 7 shows the non-linear elastic model developed for Flanders clays layers, meaning the modulus changes according to the strain level. The Flandres clay layer was divided into five 20 m sub-layers, grouped into two families: Flandres clays 1-3 and Flandres clays 4-5. Each family's values were normalized with a reference modulus (E_{max}), with trend curves described by the form $E/E_{max} = 1/(1+\epsilon/\epsilon_0)$. The application of nonlinear elasticity, along with strain-dependent modulus degradation, has been previously demonstrated to provide reliable estimates of settlements for large raft foundations, notably through back-analyses of monitored settlements at several nuclear power plants in France (Hoang *et al.*, 2018).

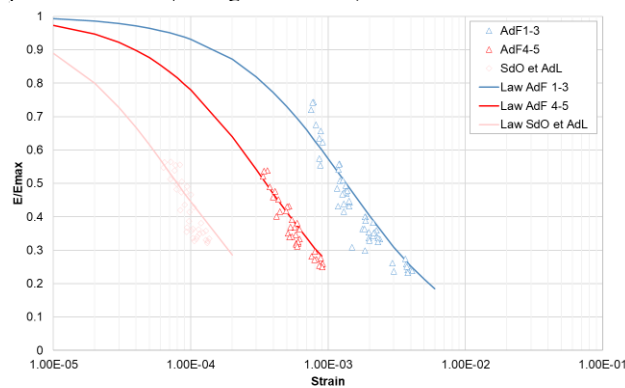


Figure 7. Non-linear elastic model for Flanders clays

The implementation of this calibrated model showed good agreement between calculated and measured settlements. The settlement with depth under reactor building demonstrated that the distribution between Flandrian sands and Flanders clays is consistent with measurements. As illustrated in Figure 8, settlements in the deeper layers under Flanders clays were found to be negligible.

Further comparison of settlement cross-section and maps between the model and the measurements (see Figure 9 to 11) confirmed the overall consistency of the calibration.

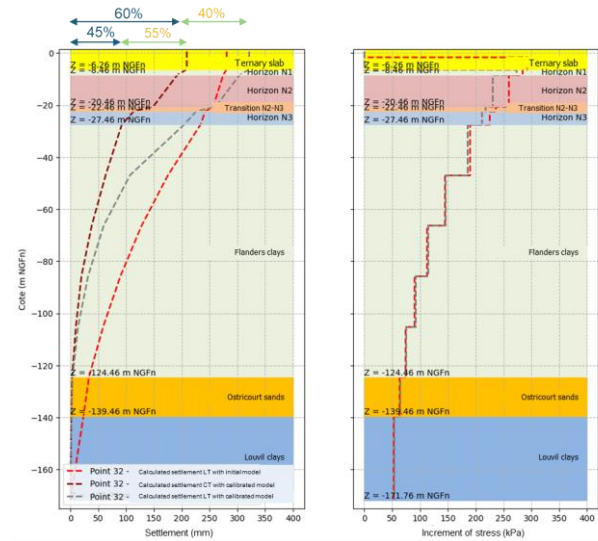


Figure 8. Settlement and stress increment under reactor building 5

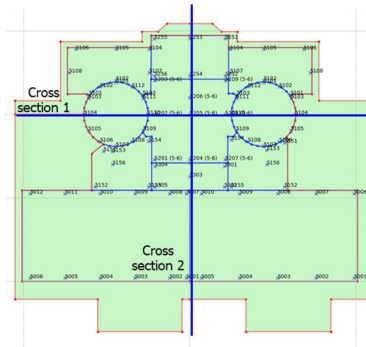


Figure 9. Location of the cross sections

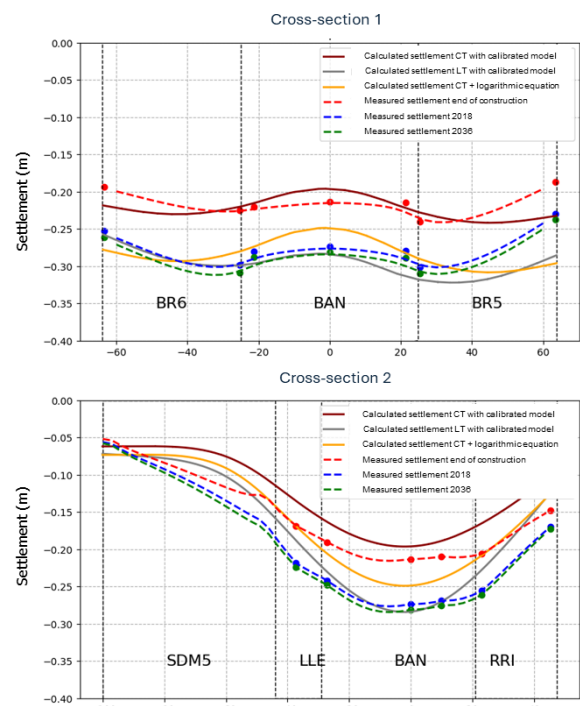


Figure 10. Comparison of settlement cross section 1 and 2

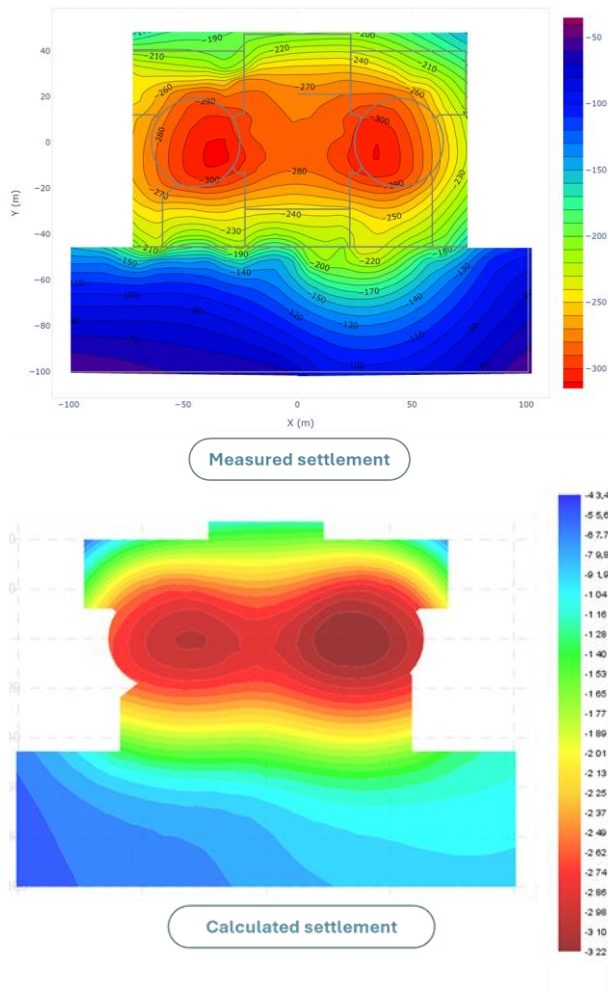


Figure 11. Comparison of 50 years settlement maps (in mm)

This calibration allows the determination of the average Young's moduli for each layer, as given in Table 1, that were then used to design the reinforcement for the future EPR2.

Table 1. Average Young's moduli for each layer from the calibration

	Z _{base} [NGFn]	E _{moy} [MPa]	Strain level [-]
Sandy layer N1	-8.46	20	1.50%
Sandy layer N2	-20.46	40	0.40%
Sandy layer N2-N3	-22.46	8.5	2.00%
Sandy layer N3	-27.46	36	0.40%
Flanders Clay 1	-46.86	34	0.50%
Flanders Clay 2	-66.26	56	0.24%
Flanders Clay 3	-85.66	76	0.14%
Flanders Clay 4	-105.06	97	0.09%
Flanders Clay 5	-124.46	142	0.05%
Ostricourt Sands	-139.46	570	0.01%
Louvil Clay	-171.76	700	0.01%

4 SOIL REINFORCEMENT STRATEGY

In the absence of any ground reinforcement, predicted settlements under the EPR2 loads range from 60 to 80 cm, depending on the chosen geomechanical model and the construction phasing sequence.

Beyond the primary objective of reducing absolute and differential settlements to levels considered acceptable for the safe operation of the facilities, any ground improvement solutions at site must address three key challenges:

- Limiting the impact of the pronounced heterogeneity of the Flandrian Sands;
- Avoid bearing resistance failure in the intermediate N2–N3 layer;
- Controlling settlements, particularly delayed ones, within the Flanders Clay.

4.1 Challenges related to sand heterogeneity

The mitigation of the effects of the heterogeneity of the Flandrian Sands can be achieved at least in two ways:

- Homogenization of the mechanical properties through vibro-compaction (with the possible incorporation of ballast material within the N2–N3 transition layer);
- Limitation of the stress in these horizons through the inclusion of resistant elements (soil mixing, rigid inclusions, etc.).

The second approach has been adopted at the feasibility study phase.

4.2 Challenges related to the bearing capacity of the N2–N3 layer

The N2–N3 transition layer represents a mechanically weak horizon within the Flandrian Sands. Based on available CPT test results, the bearing capacity for this layer is estimated at approximately 500 kPa. Without any ground improvement, the allowable stress would be exceeded beneath the heaviest EPR2 facilities. Consequently, reinforcement of this layer is required, ensuring that stress levels under both SLS and ULS conditions remain within regulatory limits. The critical design criterion is the limitation of the stress transmitted to the ground under SLS loading, which corresponds to a safety factor of approximately 3 relative to the bearing capacity. Therefore, the stress increment under SLS QP must not exceed 170 kPa within this layer.

4.3 Challenges related to settlement in Flanders clays

In the absence of any ground improvement, the stress increment applied at the top of the Flanders Clay is estimated at 400 kPa beneath the heaviest BDM2 facilities, corresponding to a strain level of approximately 0.8–1% over the first 20 meters.

- Based on correlations with CPT results, the allowable stress under service load, is exceeded, exposing the structure to delayed mechanisms (creep) that may affect its operational performance;
- A strain level of 0.8–1% is unacceptable for structures sensitive to displacements.

Ground reinforcement is therefore necessary to ensure elastic response of the subsoil, particularly within the upper portion of the clay layer, enabling the control of settlements and the prevention of creep phenomena. The range of stresses and strains induced by the loads of the existing plant defines a verified elastic domain that can guide the design of reinforcement solutions. The following design criteria have thus been defined:

- Vertical strain over the first 20 meters: $\varepsilon_{\text{vert}} \leq 0.5\%$
- Vertical stress increment at the top of the clay layer: $\Delta\sigma \leq 250$ kPa. This criterion also coincides with the regulatory ELS limit.

4.4 Reinforcement solutions considered

This reinforcement strategy and the criteria associated led to the distinction between two categories of buildings: ‘light’ buildings, inducing a net stress increment in the ground that is less than or similar to that of the existing 900 MW power plant, and ‘heavy’ buildings inducing a net stress increment in the ground greater than that of the existing 900 MW power plant.

The reinforcement system considered following the feasibility study distinguishes between three types of solutions:

- Reinforcement for ‘heavy’ buildings consisting of deep piles inclusions anchored in clay eventually completed by soil-mixing caissons anchored in sand, a purge of the first sand layer (N1) or alternatively vibroflotation (Deep RI solution);
- Reinforcement for ‘light and deep’ buildings consisting of soil-mixing caissons or “short” piles anchored in sand (SM solution or shallow RI);
- Reinforcement using good-quality fill for ‘light and shallow’ buildings.

For both buildings treated by RI or SM, a load transfer platform between 2.5 and 3 meters thick is placed between the foundation slab and the soil reinforcement. The reinforcement process is thus similar to that of rigid inclusion (ASIRI, 2012; Cuira *et al.*, 2009).

Although the nature and thickness of the platform have little influence on absolute settlement, it plays a crucial role in distributing loads more uniformly under the foundation slab and dissipating deformation energy in response to seismic loads (Alzate *et al.*, 2022; Shen *et al.*, 2024)

The design justification of these solutions was based on the following principles:

- Verification of the structural resistance of the reinforcement elements (soil-mixing, rigid inclusions, etc.);
- Verification of the geotechnical resistance of the reinforcement elements: mobilized shear strain of supporting soil, SLS bearing capacity, mobilization within the load transfer platform, etc.;
- Verification of the performance of the reinforcement solutions with respect to the design criteria:
 - Ability to reduce absolute and differential settlements;
 - Ability to reduce the mobilized strain of the soils, particularly within the N2–N3 transition layer and the upper part of the Flanders Clays;
 - Ability to improve the reliability of settlement predictions within the Flanders Clays.

These verifications were carried out through a "graduated" modeling approach, involving the successive and then combined use of semi-analytical or simplified numerical models (1D/3D), 2D finite element models, and 3D finite element models at the unit-cell, building, and site scales.

5 CONCLUSIONS

The feasibility studies conducted for the construction of an EPR2, some principles of which have been presented in this article, benefited from the extensive operational feedback from the monitoring of the adjacent nuclear power plant. Combined with geotechnical investigations, this monitoring made it possible to define a robust geotechnical model and to develop a reinforcement strategy. A principle of reinforcement using rigid inclusions, ensuring compliance with all established criteria, was thus designed. The chosen reinforcement system notably allows for reliable settlement estimates by engaging the in-situ soils within a stress and strain range already validated. This reinforcement principle is intended to be refined through the ongoing geotechnical investigation campaign on the site and a full-scale test plot of the reinforcement principle.

6 REFERENCES

- Alzate A.M., Frattini N., Cuira F., Simon B., 2022, *Projet ASIRI+ : Amélioration des modèles simplifiés existants pour le calcul des semelles et radiers sur inclusions rigides*. JNGG 2022, Lyon, France
- ASIRI. 2012. *Recommandations pour la conception, le calcul, l'exécution et le contrôle des ouvrages sur sols améliorés par inclusions rigides verticales*, IREX-Presses des Ponts, Paris, France
- Cuira F., Simon B., 2009. *Modélisation 3D simplifiée d'une plaque sur sol multicouche élastique*, *Revue Française de Géotechnique* N°124, 3-17.
- Cuira F., Simon B. 2009. *Deux outils simples pour traiter des interactions complexes d'un massif renforcé par inclusions rigides*, 17th ICSMGE, Alexandrie
- Josseume H. 1998. *Propriétés mécaniques de l'argile des Flandres à Dunkerque et à Calais*, *Revue Française de Géotechnique* N°84, 3-26.
- Hoang, M.T., Cuira, F., Dias, D. et Miraillet, P., 2018. *Estimation du rapport E/EM : application aux radiers de grandes dimensions*. JNGG 2018
- Hoang M.T., Cuira F., Burlon S., 2020. *Vers une approche de calcul unifiée de tassement des semelles et des radiers* JNGG 2020, Lyon, France
- Shen Y., Pérez-Herreros J., Cuira F., Burlon S., Semblat J-F., 2024. *Seismic bearing capacity of shallow foundations on soil reinforced by rigid inclusions*. ECSMGE 24