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Optimization of a Nuclear Reactor Foundation Based on Field Geotechnical Testing and Numerical Modeling

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Abstract. The Argentinian National Commission of Atomic Energy (CNEA) developed the RA-10 project, an experimental multipurpose nuclear reactor with a power of 30 MW, located in the Ezeiza Atomic Center, in Buenos Aires, Argentina. The foundation of the reactor is a reinforced concrete mat placed on a sedimentary pile including Pleistocene indurated fine soils and deep Miocene clays. The paper presents various aspects of the design of this foundation, including the calibration of numerical models for the 3D ground-structure interaction analysis and the structural optimization of the mat in terms of stresses, deformations and interaction among adjacent buildings.

Keywords. Nuclear power plant, optimization, finite element modeling, in-situ testing.

1. Introduction

The Argentinian National Commission of Atomic Energy (CNEA) developed the RA-10 project, which is an experimental multipurpose 30 MW nuclear reactor located in Ezeiza, close to the main airport of Buenos Aires city. The facility consists of four main buildings occupying an area of approx. 15000 m^2 (Figure 1): the reactor itself (RB), a neutron guide facility (NGB), and an auxiliary (AB) and service buildings (SB).

The foundation of the RB building comprised a 0.8m-thick reinforced concrete mat placed at a depth of 11.0 m over 1.5 m of engineered fill. The foundation of the NGB building entailed a 1.4m-thick mat founded at a depth of 1.4 m over 3.0 m of engineered fill. In both cases, the engineered fill rests on the Pampeano Formation (PFm), a cemented and indurated loess loam exhibiting high shear strength and stiffness. Due to operation requirements, the relative settlement and angular distortion between these two major buildings, and the internal distortions of both foundations, were highly restricted.

The existence of a deep layer of normally consolidated, old Miocene clay raised concern with respect to the long-term behavior of the foundation and the relative settlements of the various buildings of the facility if this clay deposit happened to activate primary compression, a risk that could not be disregarded for NGB due to its weight and surficial foundation. It was then decided that a full numerical analysis of the problem be

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performed and predictions about settlements, contact pressures and long-term structural demand be performed taking into account the 3D nature of the problem, to ensure that requirements on angular distortions be met.

2. Geotechnical characterization

2.1. Geological background

The site stratigraphic profile corresponds to a high terrace of the Pampeano Formation (PFm), which is a Pleistocene modified loess, preconsolidated by desiccation and cemented with calcium carbonates and manganese oxides [1-3, 8]. The PFm is generally subdivided into three units: upper, middle and lower Pampeano [1]. The upper Pampeano comprises medium plasticity compact silts and clays with calcareous nodules in a poorly cemented matrix; the middle Pampeano consists of hard and fissured medium plasticity silts and clays with a cemented matrix, and with pre-overburden pressures in excess of 1.0 MPa [4][5]; while the lower Pampeano is formed by medium/high plasticity clays exhibiting poor cementation but high overconsolidation. Underlying the PFm are the Puelche and Paranaense Formations are found; the first one corresponds to Pliocene fine dense sands; the second is a Miocene plastic, normally consolidated stiff clayey silts/silty clays which show an apparent slight overconsolidation due to aging.

2.2. Geotechnical testing and characterization

An extensive testing program was carried out at the site. Field tests comprised 25m-depth SPT borings, 80m-long MASW lines and PLT tests (Figure 1). Laboratory tests included characterization, triaxial and oedometer tests on undisturbed samples. The upper PFm was found up to a depth of 3.5 m, predominantly CL, with N_{SPT} ranging from 5 to 11 (average of 9), a mean effective cohesion of 15±5 kPa and a friction angle of 30°±2. The middle PFm is found below up to the end of the borings, predominantly ML, with N_{SPT} ranging from 17 to 50 (average of 32), an effective cohesion of 40±20 kPa and an effective friction angle of 34±2°.

Figure 1. Project layout. Main buildings (up) Location of PLT and MASW testing (down).

3. Constitutive model and calibration of parameters

The Hardening Soil with Small-strain Stiffness (HSSmall) constitutive model embedded in Plaxis and described in [6, 7] was used in the finite element models.

Calibration was heavily supported by in situ tests and past experience, because it is very difficult to obtain good quality undisturbed samples of Pampeano soils [3, 8].

The reference shear modulus G_o^{ref} and the stress-dependency exponent m were obtained by fitting the shear wave velocity profile obtained from MASW lines (Figure 2) and are in reasonable agreement with previous data [9]. The unload/reload stiffness E_{ur}^{ref} was estimated as 30% of the small-strain stiffness [10] and validated using PLT data [11]. The secant stiffness E_{50}^{ref} was in turn estimated as 30% of E_{ur}^{ref}, a common assumption when calibrating HS-Small for stiff soils. The rest of the parameters were adopted from previous studies [11].

A simple axisymmetric finite element model was used to validate the calibrated HSS parameters against PLT measurements on site. The model reproduced all stages of the test including the excavation of the test pit, represented by distributed loads at the soil surface. Model results, shown in Figure 2 were deemed a satisfactory validation of the set of parameters employed for the surficial soils. A remarkable fit was obtained up to a plate contact pressure of 300 kPa, while a slight underprediction of unload/reload stiffness was observed.

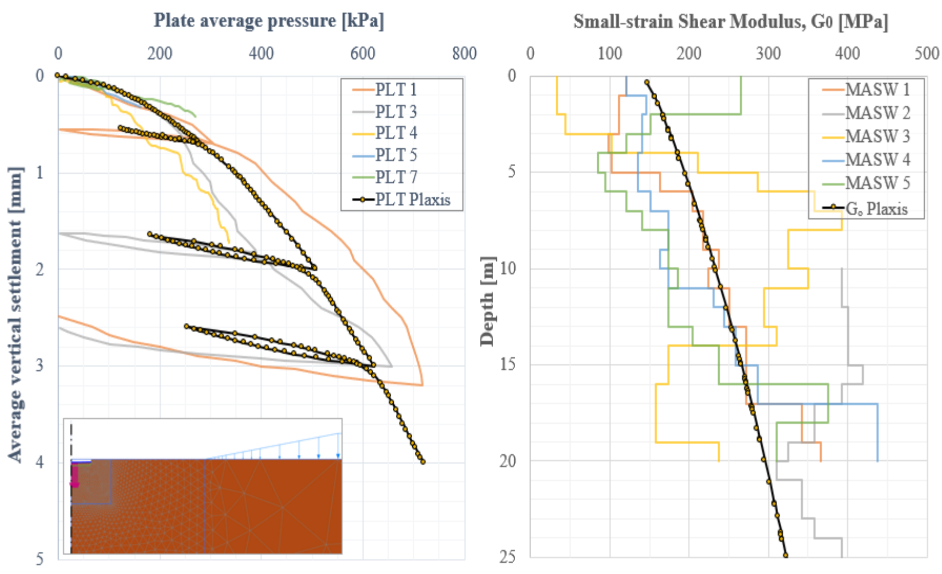

Figure 2. HSS stiffness calibration using PLT and MASW measurements.

Due to lack of specific site information, the material parameters for the Puelche and Paranaense Formations were estimated based on previous experience. Of particular importance is the estimation of the pre-consolidation pressure of the deep Miocene clays. Despite underlying the whole city of Buenos Aires, these clays have not been the subject of much research. Largely based on the behavior of a previous large foundation for another reactor [1], a value $OCR = 1.2$ was adopted in this study. Parameters are summarized in Table 1.

Table 1. HSS parameters.

Description	**Symbol**	**Pampeano Fm**	**Puelche Fm**	**Paranaense Fm**
Soil unit weight	$\gamma\ [kN/m^3]$	18	20	17
Effective friction angle	ϕ' [°]	32	36	25
Effective cohesion	$c'[kPa]$	20	0	0
Dilatancy angle	ψ [°]	3	0	0
Unload/reload stiffness at 100 kPa	$E_{ur}^{ref}\ [MPa]$	190	260	85
Secant stiffness at 100 kPa	$E_{50}^{ref}\ [MPa]$	95	100	30
Oedometric stiffness at 100kPa	$E_{oed}^{ref}\ [MPa]$	95	100	30
Stress exponent	$m\ [-]$	0.50	0.50	0.80
Small strain shear modulus at 100kPa	$G_o^{ref}\ [MPa]$	280	350	105
Reference shear strain	$\gamma_{0.7}\ [-]$	1E-4	1E-4	1E-4
Poisson´s ratio	$\nu_{ur}\ [-]$	0.20	0.20	0.20
Pre-overburden pressure	$POP\ [kPa]$	1000	400	-
Over-consolidation ratio	$OCR\ [-]$	-	-	1.2
Horizontal pressure coefficient	$K_0\ [-]$	0.70	0.45	0.50

4. Modeling the Reactor Building Foundation

4.1. Model configuration

A 3D model in Plaxis was employed to assess settlements and contact pressures of the RB mat for service loads; and to evaluate structural demand for ultimate states, which includes dead (D), live (L) and seismic (EX – EY) loads. Seven load combinations were employed: dead loads (D), dead plus live loads (D + L), factored dead plus live loads (1.2 D + 1.6 L) and factored dead, live and seismic loads (1.275 D + 0.8 L ±EX and 1.275 D + 0.8 L ±EY). Structural loads were input as point loads obtained as the reactions of rigid supports in structural models. This means that, while the stiffness of the foundation itself was properly considered, the stiffness of the building was not taken into account. The model has 100 m x 200 m in plan view and a depth of 60 m, enough to include the full thickness of the Miocene clays (Figure 3).

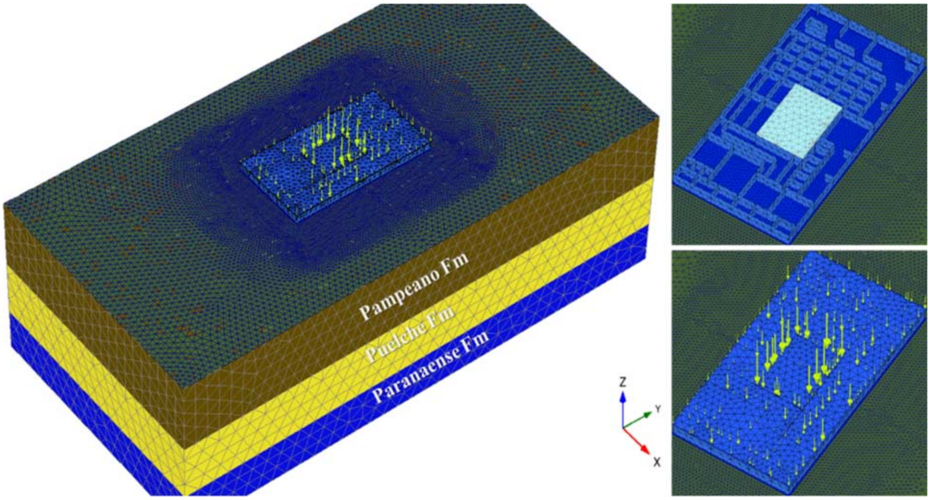

Figure 3. Reactor Building. 3D model mesh.

The RB mat, 31 m x 58 m x 0.8 m thick, is located at the center of the model. To accurately represent the RB soil-structure interaction, most of the structural details of the mat were included in the model: reinforcing beams, shear walls, variable thickness slabs and even the concrete infilling of large voids. All structures were modelled as isotropic linear elastic materials.

Modelling stages can be summarized as follows: i) initial stress calculation following K_0 procedure; ii) replacement of the soil above the foundation level for an equivalent load of 200 kPa; iii) deactivation of this equivalent load at the mat area to simulate the excavation; iv) activation of the structural elements; v) activation of point loads for the different load combinations. Loads are located in the upper slab in agreement with the shear wall axes that reaches the RB mat, and were input using a Python script, required to deal with hundreds of loads for each of the seven load combinations.

4.2. Results

The first objective of the study was to analyze whether the engineered fill - 1.5m-compacted-gravel – would add enough value to the performance of the foundation to justify its construction.

Figure 4 shows a comparison of the mat settlements and soil contact pressures for service loads (D+L) considering the RB mat founded over the gravel fill and placed directly over the Pampeano Formation. It is demonstrated that the gravel fill has a minor effect on the overall response of the foundation, an expected result due to its reduced thickness with respect to the mat dimensions. Furthermore, it was demonstrated that with or without the engineering fill, the vertical displacements at the center of the slab, reach a maximum of 34 mm, below the max allowable value of 60 mm. Contact pressures range from 50 to 300 kPa, with localized peaks of 600 kPa in small regions at the perimeter of the foundation where heavily loaded shear walls are located.

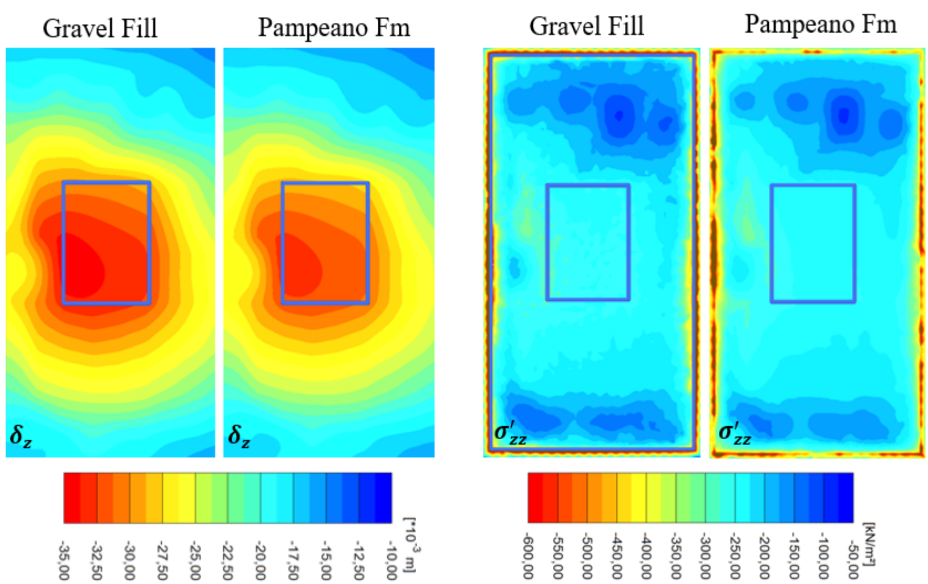

Figure 4. Settlements (left) and contact pressures (right) below the Reactor Building mat for service loads.

The RB mat structural capacity is assessed by means of the argentinian design code for concrete CIRSOC 201-2005, which largely follows ACI-318. The design bending moment is 660 kN.m/m, while the design shear force is 592 kN/m. These values were determined by the structural design of the building and the mat, assuming it was resting on a Winkler-type spring foundation.

The inclusion of the inelastic settlements of the foundation soils due to compression of the deep clays and distortion of the surficial layers produced an additional structural demand that was not considered in the structural model of the mat. Figure 5 shows the zones is which the structural design capacity is exceeded, superposed for all factored load combinations. It is observed that the mat capacity is generally sufficient and is only exceeded where the mat has large jumps in it's flexural stiffness, namely the central bulk fill and the shear walls. Those areas were reinforced to satisfy this additional structural demand and the mat was built according to this optimized design.

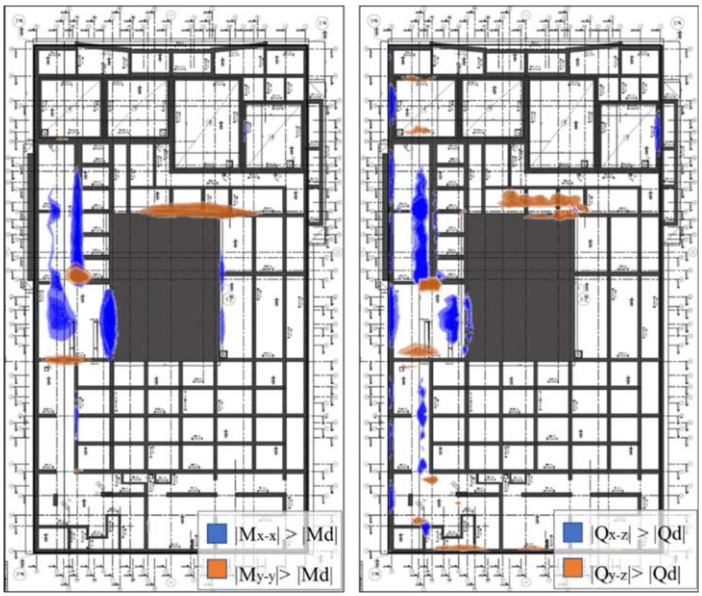

Figure 5. Areas of the RB mat where design bending moments (left) and shear forces (right) are exceeded.

5. Interaction Between Reactor and Neutron Guide Buildings

5.1. Model configuration

A 2D plane-strain numerical model was done to evaluate the angular distortions as a product of the interaction between NGB and RB. A 20m-stripe, common of both buildings, was evaluated. Equivalent 2D loads were estimated using the tridimensional loads with theories of beam and slab on elastic medium (Figure 6).

The NGB mat is 70 m wide, 1.4 m thick and placed over 3.0 m of engineered fill, while the RB mat section is 31m wide, 0.8 m thick (but heavily stiffened by shear walls) and placed at a depth of 11 m. Both structures were modeled using isotropic, elastic plate elements (Figure 6).

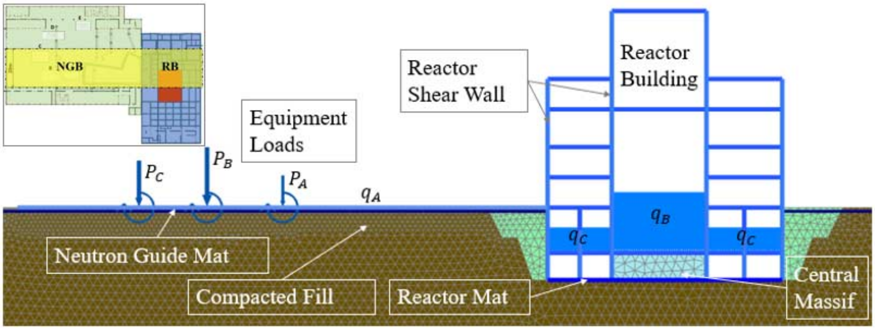

Figure 6. Interaction among buildings. 2D model mesh.

Five equipment are installed on top of this mat stripe: two spectrometers (1880 kN and 2520 kN), two diffractometers (800 kN and 1880 kN), and one scanner (1920 kN). Although the full construction procedure was modelled, the aim of the model is to evaluate deformations that might lead to equipment malfunction. Therefore, the results focus on deformations after the installation of these equipment, which was assumed to happen after the RB construction was finished. Then, six load stages were analyzed: i) NG equipment and live loads; ii) to vi) where 10, 20, 30, 40, and 50 % of the RB live loads, respectively, were added.

5.2. Results

The evolution of settlements and angular distortions along both mats are presented in Figure 7. A key requirement is that settlements must be within a cone projection of 5E-5 rad from neutron source. In addition, the angular distortion of the guides must be preferably below 5E-5 rad, with a maximum of 1.5E-4 rad. It is shown that both requirements are accomplished.

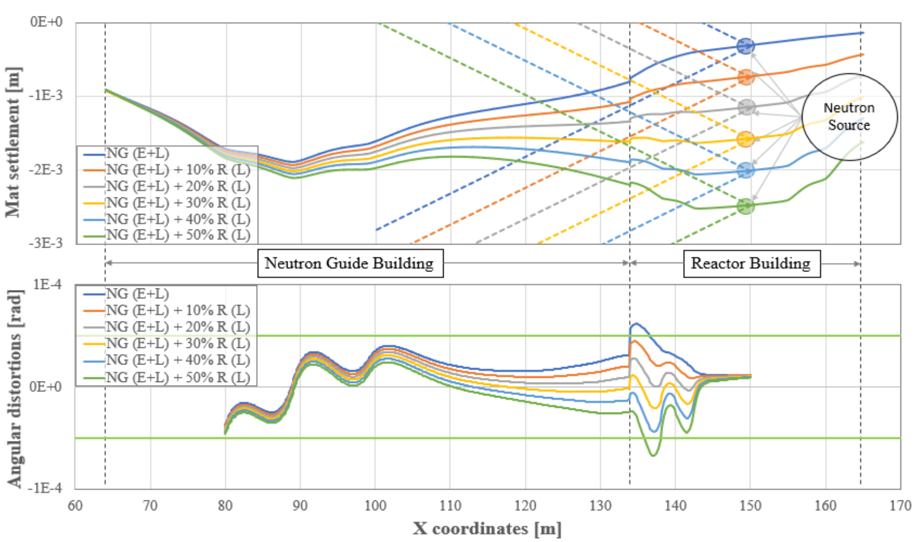

Figure 7. Influence of loading procedure on mats settlements (up) and angular distortions (down).

6. Conclusions

CNEA developed the RA-10 project, which is an experimental multipurpose 30 MW nuclear reactor. Foundations of the RA-10 buildings were designed employing structural-type models where the mat was supported by Wrinkler-type springs. Due to concerns around the settlements of the building due to deep, normally consolidated clays, a full numerical analysis of the ground-structure interaction problem was performed.

The feasibility of the foundation system was studied by means of 2D and 3D numerical models simulating the ground-structure interaction including the primary compression of the underlying clays. Due to the scarcity of site-specific data, the calibration of the models largely rested on previous experience, complemented with some in-situ testing including SPT, MASW and PLT tests. The back-analysis of PLT results employing the material parameters used in the modeling was used to prove that enough predictive capability was obtained, at least for the surficial materials.

The result of the study was an optimization of the design where an engineered fill made of compacted gravel was removed and the structural capacity of the foundations was slightly upgraded to take into account the effect of the inelastic settlement of the underlying soils, while accomplishing strict requirements of settlements and angular distortions, common to these high-sensitivity facilities. To date, the foundation and the building were built to completion.

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