Latest experiences in complex soil investigation in the Spanish Plateau of Castile

S. Ortiz Palacio
University of Burgos, Castile and Leon, Spain.
Eptisa Servicios de Ingeniería, S.A. Burgos, Castile and Leon, Spain

J.A. Porres Benito
University of Burgos, Castile and Leon, Spain

ABSTRACT: In recent years, many geotechnical specialists must confront the need of accomplishing difficult designing tasks under very restricted budgets (tasks such as the design of complex foundations, avoiding slope movements, the development of earth containing structures, evaluating and repairing settlements, etc.). This implies that soil *in-situ* probing and lab testing may be quite limited on many occasions. Therefore, it usually falls on the specialists the responsibility of optimizing the resources at hand to fully characterize the morphological and mechanical soil qualities, gaining the best safety factors as possible in the process. In order to illustrate this challenging duty of optimization of the problem of quality soil characterization vs. probing budget, some of the latest and most interesting of the authors' experiences in the Spanish Plateau of Castile will be covered.

1 INTRODUCTION

Geotechnical researchers and engineers should be considered –in the context of any constructive process- as crafters of structural ellipsis: we can understand a structure as a narrative discourse (in which different elements interlink together, establishing a tensional dialogue with the supporting ground), so the geotechnical team comes in as the right tool for making sure this narration doesn't break up. It will sustain its very fabric -every material, every beam, every joint, that is, every phrase in the structural discourse-. In other words, geotechnics must ensure stability, equilibrium and resistance for the structure while preserving its aesthetical and functional values, but always remaining hidden in the process, or, at most, hardly glimpsed.

This vocation for staying at the backstage, far behind the limelight of the creative process of any civil or architectural project sometimes leads to a common unfair situation: the effort of the geotechnical investigator may be not properly understood or valued as much as it deserves.

In addition, in the actual economical maelstrom, many new constructive initiatives are deeply budget-conscious, and soil investigation has sadly become one of the usual targets of budgetary cuts, as its real importance is too often underestimated.

2 PRACTICAL EXPERIENCES

2.1 *Burgos Courthouse (Castile and Leon –Spain)*

This neoclassical building, finished in 1874 after the design by Architect David Ruiz Jareño on the premises of the "Mínimos de la Victoria" Convent, became scheduled for structural and functional improvement from 2.005 to 2.012. Its original structural conception was simple and yet, very powerful, as can be seen in Figs. 1 and 2: two symmetrical cells of three concentric lines of thick masonry walls sustained wooden framed floors.

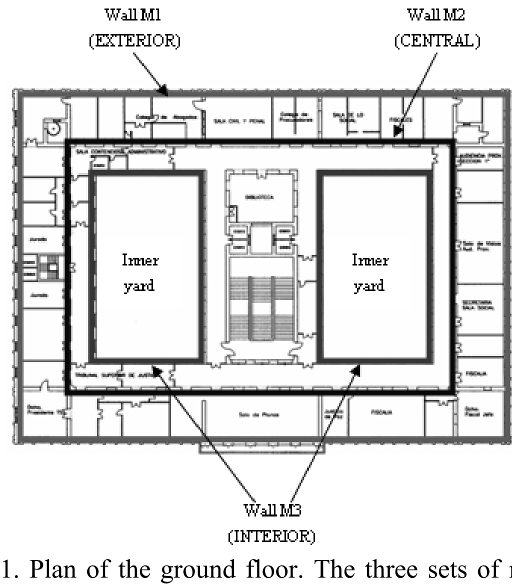

Figure 1. Plan of the ground floor. The three sets of masonry master walls are highlighted.

The new Project would eliminate those floors, as an answer for the new design loads, while keeping the vertical elements in the newly shaped building.

As the Court was still fully functional in the early stages of design, the first approaches to geotechnical probing were limited to the external walls of the building (M1). Thus, subsoil characteristics for the central (M2) and interior (M3) foundations were not properly investigated in this stage.

Furthermore, the newly designed floors would prove to transmit up to 348 kN/m (this is a value for the serviceability limit state) on the central wall foundation (M2). As the soil consisted on 3.8 meter-thick alluvial gravels (from nearby Arlanzón River) laying on stiff tertiary loamy clays, the bearing capacity of the soil would be tested to the limit.

The immediate problem was a crucial one: the value for the bearing capacity security factor heavily depended on the internal friction angle, which had not been properly researched by previous investigations under the central wall (M2). Thus, a new probing campaign was advised but, due to deadlines, budget restrictions and accessibility at the time of design, it could not be properly developed. Only DPSH penetration tests and hand-made pits would be performed inside the building, so the final data were very scarce (see fig. 3).

In this unfavorable setting, the best approach to solve the problem ended up being the delimitation of reasonable wide apart maximum and minimum soil parameters, as more narrow solutions could not be safely obtained.

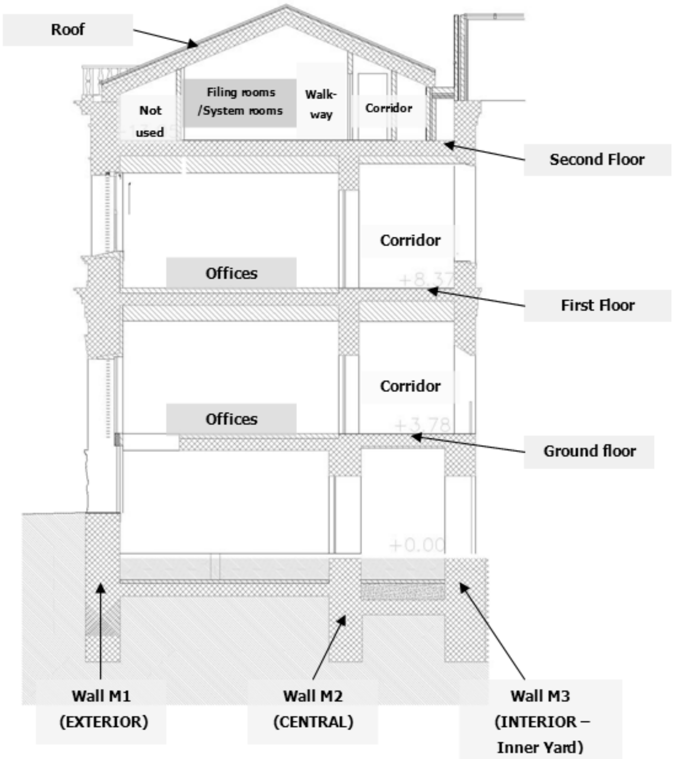

Figure 2. Vertical section for one of the building symmetrical cells.

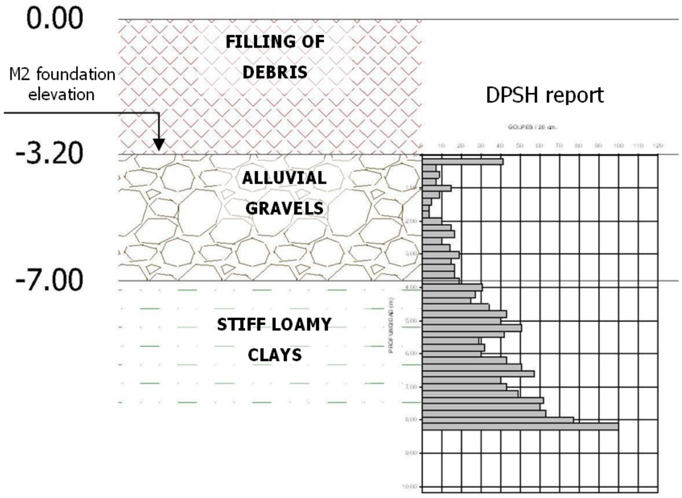

Figure 3. Soil profile vs DPSH values.

After analyzing both the peripheral data (SPT tests mainly) and inner wall foundation results (that is, DPSH registers), friction angles could be delimited between 29º and 34º. Whilst both the inner yard walls (M3) and the outer walls (M1) showed safety factors above 3 for the worst friction angle, 29º, the central wall foundation safety factor ranged from 1.74 to 2.75. Conclusions were simple: the soil below the central wall would need to be improved, or a new foundation should be developed for this vertical element.

At this stage, the Architectural Project Manager asked for new advice, this time on the suitability of a Compaction Grouting to improve the gravels bearing capacity. Once consulted, the authors expressed their doubts of the success of such a method on the Arlanzón gravels (Al-Alusi (1997) showed very low density improvement for gravel-sized soils, while some other authors –as Henríquez (2007)- admit some potential but not too clear improvement for clean gravels), but nevertheless, a thorough theoretical investigation was anyway commissioned. On this matter, Henríquez (2007) details that the Compaction Grouting effectiveness when performed on coarse gravels rises if the fine grained particles fraction is less than 25% and SPT results don't exceed 15 to 20 blows. In this case, the final modulus of elasticity can reach a value obtained from this expression:

$$E = \gamma_s \cdot \frac{P_G}{\Delta_\gamma} \tag{1}$$

where E = modulus of elasticity; γ_s= particle density; P_G = grout injection pressure; and Δ_γ= density increment.

After this first analysis and with the aided by the assessment of a highly specialized independent contractor of the injection field, it was determined that injection pressures should build up to 6 MPa to get a sufficient densification (that is, in order to rise the friction angle to guarantee a sufficient bearing capacity safety factor), aiming for 80 cm diameter bulbs. After that, the most critical condition would be the uplifting of the central wall as it would be mostly unloaded during the injection stage. Leaning on previous works such as El-Kelesh et al. (2001), El-Kelesh & Matsui (2002) and Warner (1992), an inverted truncated-cone failure model of uplifting stresses was developed specifically to understand this mechanism, as shown in Fig. 4:

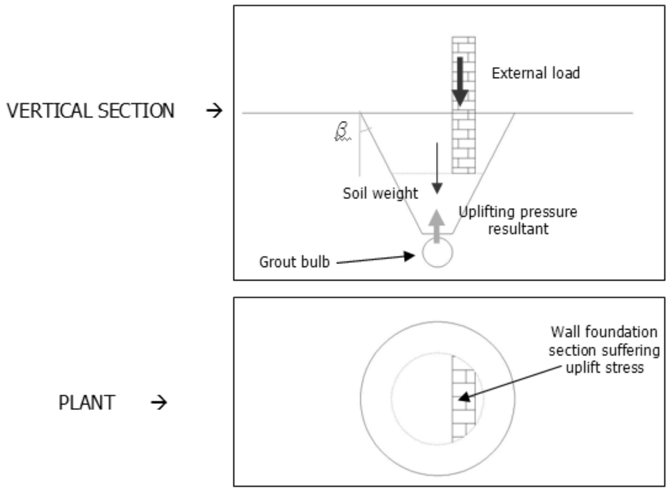

Figure 4. Vertical section and plant for the uplifting model.

After cross-checking both the pressure for a) obtaining a sufficient ratio of densification of the soil and b) for avoiding uplifting of the central wall, it was discovered that both conditions could not be harmonized. The upward and downward forces would be:

$$E_P = P_G \cdot \pi \cdot r^2 \tag{2}$$

$$R_w = \gamma \cdot V + q_s \cdot S \tag{3}$$

where E_P = uplifting pressure resultant; P_G = grout injection pressure (6 MPa); r= grout bulb radius (0.40 m); Rw= downward loads at wall base; γ= bulk unit weight of the soil above the grout bulb (20 kN/m^3); q_s= pressure at the base of the wall due to external loads (214 kPa); and S= wall foundation surface suffering the uplift stress (6.3 m^2).

In this case, according to the available data, it was determined that E_p= 3016 kN and R_w= 2373 kN. To avoid this uplifting net force, lower injection pressures should have to be applied: then, the effectiveness of densification would be lost.

Also, according to Bielza (1999), these low mobility injection methods wouldn't be expected to be very efficient due to low confinement in shallow injections. Therefore, the Compaction Grouting solution was ultimately abandoned. In the end, the most optimal resolution for this central wall was its demolition, as it didn't have any architectural or aesthetical interest whatsoever.

In this case, poor geotechnical data led to the need of making a delimitation analysis of the problem, having to examine both the worst and best scenarios translated from the available investigations. Once the Remodeling of the Court House began, the Contractor commissioned the execution of several rotary drills which allowed obtaining, eventually, some extensive soil parameters which fell safely in the previously estimated range of values.

2.2 *18-floor buildings for Burgos Urban Masterplan*

The studio of Architects Herzog & de Meuron has developed a full Urban Remodeling concept for the city of Burgos which includes a double 18-floor building and a 3-floor pavilion in the free space between them. All three structures will be sharing three intercommunicated underground levels. For the foundation design, twelve rotary drills and twelve DPSH tests were initially commissioned.

According to the structural design team, due to the long spans concept of the buildings, the towers will be transmitting very high concentrated loads to the soil beneath, while the small –in comparison- pavilion will be very light (even lighter than the mass of soil to be extracted for its underground levels during the excavation stage).

The area planned for the buildings reveals one of Burgos most complex soil configurations attending to geotechnical criteria. Figure 5 shows a typical soil profile of the area:

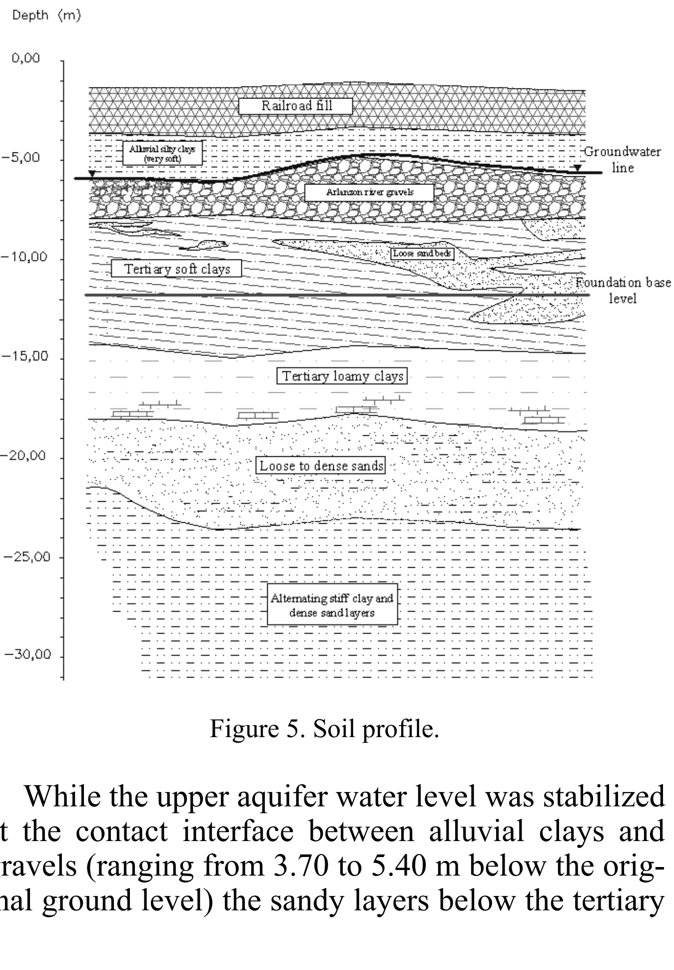

Figure 5. Soil profile.

While the upper aquifer water level was stabilized at the contact interface between alluvial clays and gravels (ranging from 3.70 to 5.40 m below the original ground level) the sandy layers below the tertiary loamy clays –acting as a confined aquifer ranging from -18 to -23 m (approx.)- had a piezometric head which reached just 60 cm below ground level, as measured by means of piezometrical devices that were installed into several of the drills. Further hydro-geological investigations led to discover that this confined aquifer had, at least, two powerful feed origins. Drilling operations through these sands were backed up by bottom water injection as siphoning would make otherwise impossible the drilling.

As the designers aimed for a piled slab foundation for both the 18-floor buildings, the original depth of the first twelve rotary drills revealed itself as insufficient, as the bottom clay and sand layers weren't resistant enough for the heavy duty piles being projected at the time of field research. Thus, another four drills were commissioned, this time engaging a deep probing up to 52 m. Drilling maneuvers were still more difficult as deep sand layers (and smaller sandy seams) showed once again very shallow hydraulic heads.

On the other side, as it has been previously pointed out, the pavilion will be a very light structure. Aided by FEM analysis, it was estimated that the uplifting pressure born under the tertiary clays would lift this building, due to the combination of high punching shear upheave and low shear strength of the upper clays -according to in situ and lab tests-. To avoid this failure mechanism, deep prestressed anchors were designed after thorough direct shear and triaxial tests were conducted.

As a corollary for this epigraph, budgetary restrictions were never an issue during geotechnical research, so probing campaigns and tests could be adapted to the designers' needs, as soil studies and foundations design went hand in hand during the whole projecting process, continuously feed-backing each other. This setting usually gives the most optimal results, and it's been described here as a contrast paradigm for the other case studies.

2.3 *9-meter high water tank on soft plastic clays in Burgos*

A leading tire manufacturing company commissioned for its plant in Burgos the geotechnical research of the future ground for a 9-meter high water tank and its pump station. Two rotary drills were conducted for the task.

The soil profile, from top to bottom layers, consisted on gravels (0 to -2.40 m) and soft silty clays (-2.40 to -10.80 m) laying on the locally well-known stiff tertiary loamy clays that extend across most of the city of Burgos subsoil.

Oedometrical tests revealed a significant modulus of volume compressibility ($m_v > 0.25$ m^2/MN), so settlement estimation soon became the main concern for both the geotechnical and structural design teams. Using an elastic model for the mat foundation simulated by a beam grid on a Winkler soil (see Fig.6), it was determined that an instant settlement of 8 cm should be expected on the center of the foundation:

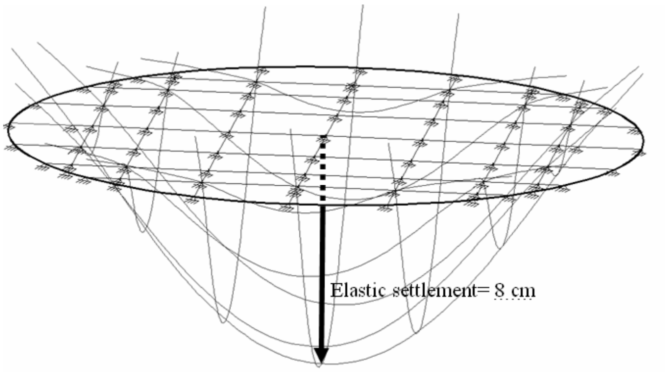

Figure 6. Model grid foundation on an elastic soil. Deformed shape has been exaggerated for clarity purposes .

Once obtained the elastic fraction of the settlement, then the consolidation deformation was then added up. According to classical one-dimensional consolidation theory, this time dependent long-term settlement could reach 8.5 cm, so definitive deformation would mount up to 16.5 cm below the center of the slab.

Asked by the client, the authors then worked on the simulation of an interactive elastic soil model with a mat on piles foundation to evaluate if this solution could be arranged to limit excessive settlement. Several models were approached, calculating axial pile stiffness after the expressions by Spanish Ministry of Public Works (2009), as the one shown on figure 7:

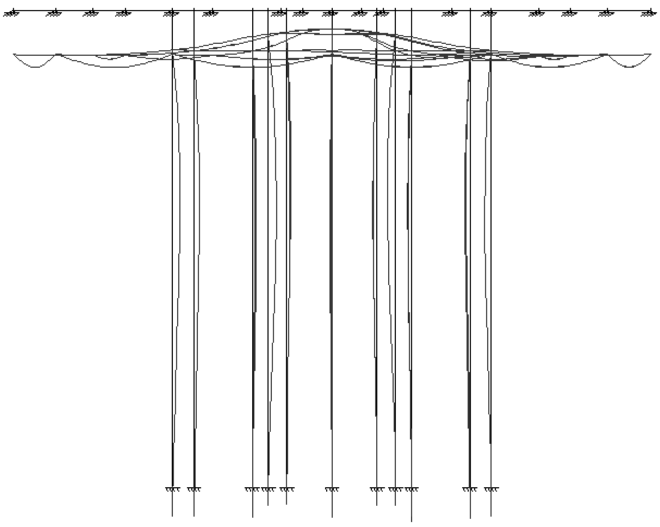

Figure 7. Vertical section of the deformed three-dimensional grid simulating the pile underpinned slab.

The Owner then expressed that this solution, while geotechnically acceptable, outranged any previous budget predictions, so another solution should be explored: two other spots across the plant premises were also suitable to ensure the tank functionality, so the Owner commissioned one additional rotary drill in each site. The target was to explore if underground parameters might show a better aptitude for a shallow foundation on any of them. Fortunately, one of the new spots revealed a thinner soft silty clays layer, so maximum settlement for the foundation would reach just 5 cm. This solution was the one adopted.

By extending the geotechnical research, the Owner saved somewhere around 100,000 € for the foundation of the tank.

2.4 *Uplifting due to expansion of low plasticity schistose clays in Guadalajara (Castilla La Mancha – Spain)*

In 2009, an electrical company reported the cracks that had been appearing at an electrical substation building both across the walls and over the concrete bases around the building. To illustrate the problem, the schematic distribution of the main cracks on the front façade is shown in figure 8 (here they are represented both the preexisting cracks –before the authors' intervention- and the ones that continued appearing during the investigation):

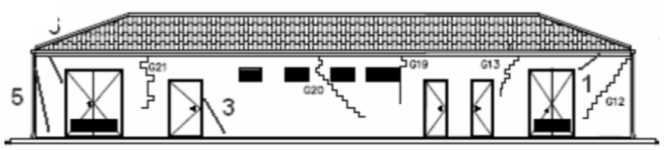

Figure 8. Map of cracks on the front façade.

After this crack mapping was complete, tensional isostatic lines compatible with crack directions were inferred, so it was hypothesized that an uplift stress was occurring under the substation. Figure 9 shows the isostatic tension lines over the crack map on the front façade:

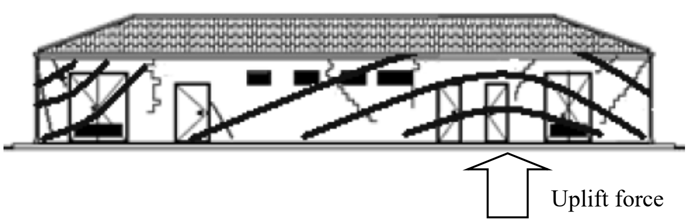

Figure 9. Inferred isostatic tension lines and upward force due to soil expansion.

Then, the previous geotechnical reports and project plans provided by the Owner were analyzed. This way, these facts were discovered:

- The building site was on slope, so a 0 to 4 meter-deep cut was performed to reach the foundation plane.
- The foundation of the very light building consisted on isolated square footings under concrete pillars.
- The soil profile under the footings showed a thick layer of schistose, non-plastic clays.
- Average plastic limits ranged between 25-28, while average liquid limits ranged between 42-47.

As soil swelling was the most likely cause for the pathology, but the soil showed little potential swelling qualities due to its low plasticity, new probing had to be performed: test pits were excavated next to some of the footings to take new soil samples. After lab tests were carried out, it was discovered that the water content was about 9 to 14 % below the plastic limit. At this point it was easy to make new deductions: the previous geotechnical report showed natural moisture contents very similar to the plastic limit before the slope cut was made, but afterwards, after excavation took place and during the summer the building was constructed, desiccation and, therefore, shrinkage took place. Then, as the building surroundings had not been properly waterproofed, seepage and superficial run-off began to cause progressive swelling to the previously shrinked clays. To study this matter, the author compiled previous data on dry clay swelling, determining that low-plasticity clays could show high swelling indexes if their water contents were below their plastic limit. Some of the data is shown in figure 10 (in the horizontal axis the difference between the plastic limit and the real water content is represented):

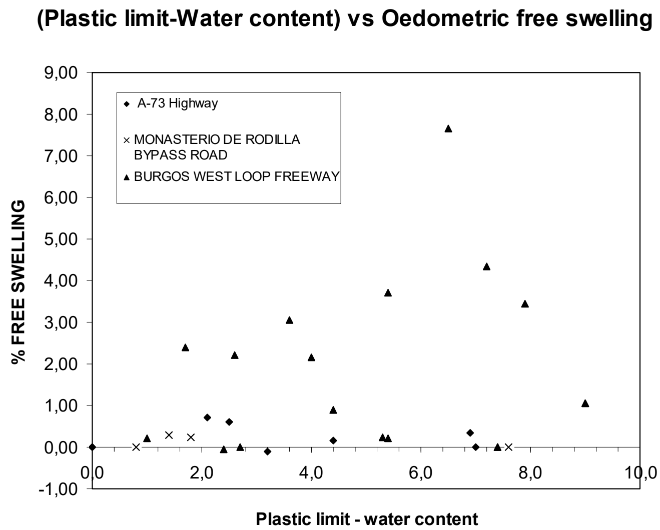

Figure 10. Free oedometric swelling of Burgos soils.

In Figure 10, values with less than 0.5 % swelling were obtained in samples with less than 35 % of clay particles, while clayey soils showed high expansive properties when dried.

It was then clear that the more desiccated the soils were, the more swelling would be produced. In the building in Guadalajara, the induced heave deformed the building. This hypothesis had been already stated in previous works, as, for instance, in Day (2011).

To solve the pathologies, new drainage and perimeter waterproofing solutions were then designed by the authors and, as many of the crack openings were being monitored, these solutions prove to stop the swelling-shrinking process once the seepage below the building ceased to exist.

This example shows how a thorough initial interpretation of crack directions and their most plausible origin led to design a non-expensive soil surveying campaign, as it was already known lab tests would have to deal with swelling processes and not with other mechanical or chemical properties.

2.5 *The foundation for a new bridge inside the medieval Costana Bridge in Salas de los Infantes (Castile and Leon – Spain)*

60 kms southeast from the City of Burgos, we discover the town of Salas de los Infantes, known, among other things, for its medieval bridge located on the so-called Costana District. Due to its strategic position, it had always been used as the crossing path over the Arlanza River on the main road N-234 connecting Burgos and Soria, suffering a high intensity of heavy traffic. It is a five arches masonry bridge built from 1641 to 1646 by Architect Tomás Rivera (see Fig.11), and transversally widen in 1944 in an unfortunate reinforced concrete shape which didn’t respect the original geometry for the arches (see Fig. 12 for details on this matter).

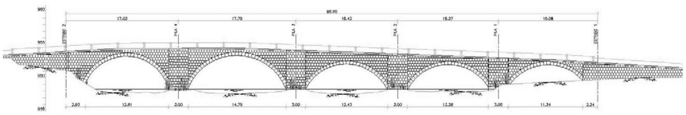

Figure 11. Downstream vertical view of the masonry bridge

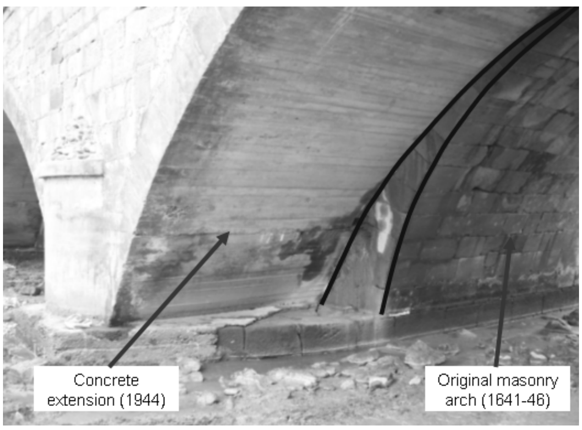

Figure 12. Detail of 1944 and 17th century arches. Differences between geometries are highlighted.

In May, 2007, the already damaged bridge saw one of its arches partially collapse, so the road had to be closed to traffic, as can be observed in Fig. 13:

Figure 13. Collapse of the key section of the arch.

Because of the architectural and historical relevance the bridge, the Ministry of Public Works – responsible for this infrastructure- commissioned a project for a new bridge that would preserve the preexisting one. Thus, an integral bridge was designed to comply with the Owner’s expressed requirements. The bridge would consist on a post-stressed thin deck monolithically connected to micropiled piers and abutments. These micropiles would go through the preexisting structure without connecting with it at all: as the old bridge would be repaired, its future task would be no other than sustaining itself, so no external loads would be allowed to be transmitted to it (see Ortiz (2010) for further details on the new bridge design and its construction process).

To gather the essential information to design the micropiles, several rotary drills were performed in each pier, right from above the bridge. The nature of the backfill of the arches and the layers that supported the old bridge and had to support the new integral structure were the main subjects of the geotechnical surveying.

The profile revealed by the drills is shown both in Figs. 14 and 15: down below the heterogeneous backfill and the stone base (on wooden piles under the 17th century bridge and directly on the river bed under the 20th century extension) 3.50 meter-thick alluvial gravels were found. Below this layer, tertiary stiff clays were observed, extending down to the maximum probed depth.

Then, the definitive designing stage took place: the backfill would be improved and properly compacted. Over it, planks of expanded polystyrene would serve as the formwork for the post-stressed deck (which would detach itself from the old bridge after post-stressing was fulfilled).

The micropiles were designed to go through the backfill and the foundation of the old bridge with full detachment from it. Also, as injection through the river gravels posed some technical and hydrological challenges, it was decided that the injection wouldn't touch this alluvial layer, either. The final draft for the micropile is shown below in figure 14:

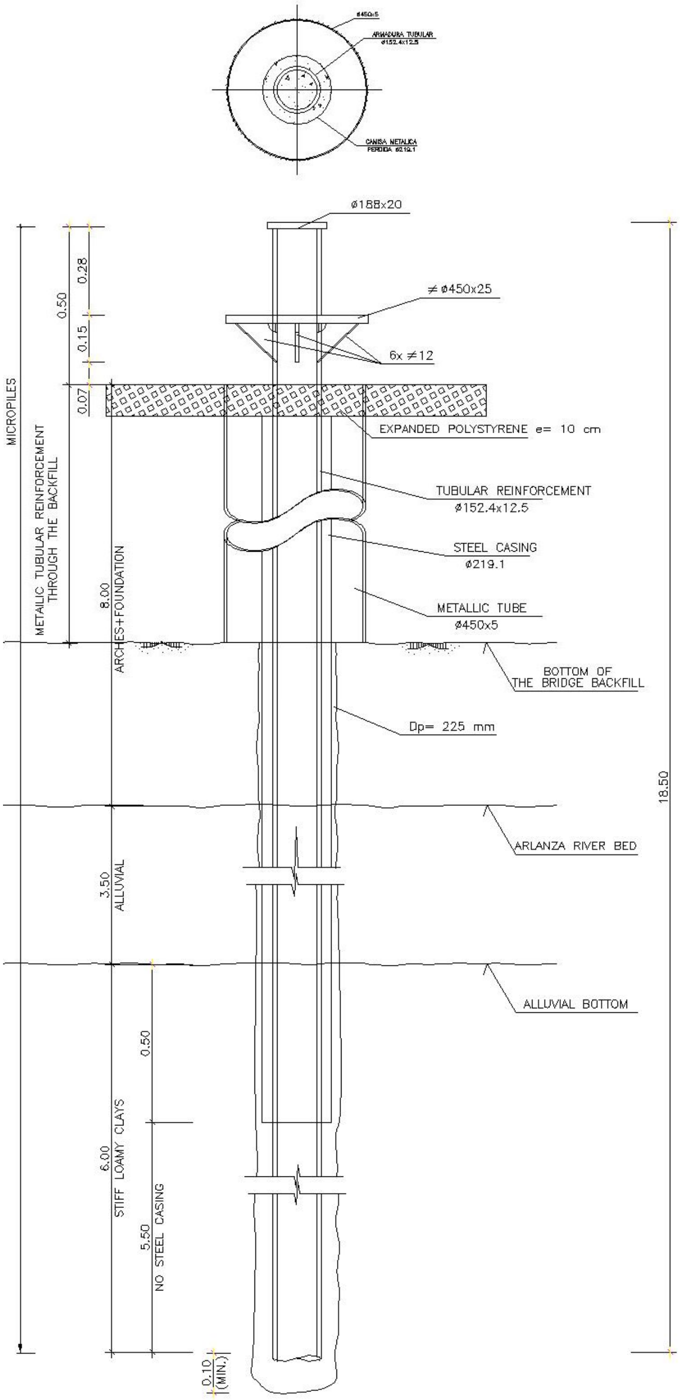

Figure 14. Vertical and plant cross section of the micropile. Note how the steel casing penetrates the stiff loamy clays layer, avoiding any injection reachin neither the backfill of the old bridge nor the alluvial gravels below its foundation.

The unrecoverable steel casing socketed itself into the stiff loamy clays in order to avoid any injection reaching the bridge or its foundation. The expected stresses if this precaution hadn't been adopted would have been, schematically, the ones shown in Fig. 15:

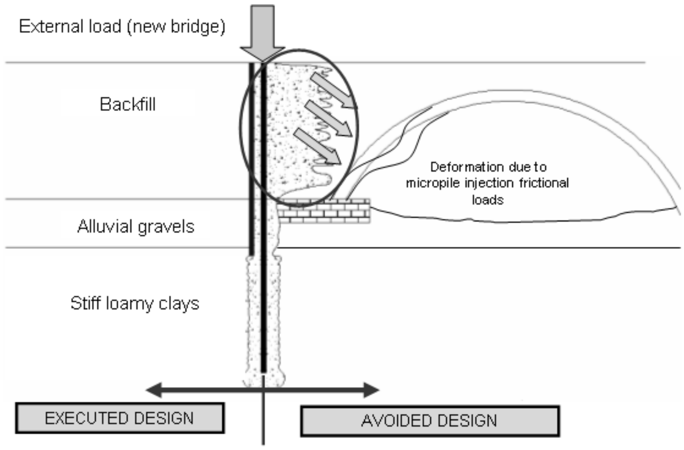

Figure 15. Parasite stresses induced by the micropile frictional forces if a steel casing hadn't been socketed into the loamy clay layer.

On the left side, it is been represented the executed design, with the steel casing going through the bridge backfill, foundation and alluvial gravels, thus avoiding any injection inducing frictional stresses between the micropiles and the old bridge, while on the right side the undesired situation is schematized.

3 CONCLUSIONS

Budgetary restrictions are commonplace in geotechnical surveying, so when a soil reveals itself as a complex one, the specialist must deal with intuition, using any tool at hand to build a complete a safe report on the soil properties and recommendations for foundation, repairing pathologies, etc. Sometimes symptoms from a soil behavior will reveal much more information than an expensive, extensive sampling campaign, while other times the available data will be so scarce, that applying high safety levels to delimitation analysis will be the only solution available. In any case, developing new, more accurate and ever less expensive methods for soil probing is a need we must not detour from at the present and in the near future.

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