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Analysis, design, and construction of mixed stability solutions for a 19 m depth excavation

Análisis, diseño y construcción de soluciones de estabilidad mixtas para una excavación a 19 m de profundidad

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ABSTRACT: In the metropolitan area of Monterrey, there are current projects that require the design and construction of basements that can be located in alluvial soils and shale rock, and also these basements are projected below the water table. Under these geotechnical conditions, different construction techniques are required to ensure excavation stability and groundwater control. This work describes the analysis, design and construction of a 19 m excavation located in San Pedro Garza Garcia. Different stability solutions were used due to the geotechnical particularities and the needs of the project. Thus, to achieve the maximum excavation level, a diaphragm wall with active anchors, tangent piles in cantilever, diaphragm wall supported by struts, micropiles accompanied by Soil-Nailing, plastic wall, slope with passive anchors and shotcrete wall with active anchors, were performed. The performance of the different systems of stability was verified by the instrumentation installed in the project.

KEYWORDS: Excavation stability, Groundwater control, Diaphragm wall, Active anchors, Geotechnical conditions

1 INTRODUCTION

In the metropolitan area of Monterrey, there are currently building projects with a significant number of superstructure levels, for which the design and construction of parking basements are necessary.

In this area of the country, it is common for parking basements to be located in alluvial soils and shale rock, in addition, they will be below the groundwater level (NAF). Under these conditions, it is necessary to use different construction techniques to ensure the stability of the excavation and the control of groundwater.

This work describes the analysis, design, and construction of an excavation at a depth of 19 m located in San Pedro Garza García, Mexico.

Different stability solutions were used due to the geotechnical conditions and the needs of the project. Regarding the geotechnical conditions, it is highlighted that there is an important change in the location of the level of the shale rock. Respect to the needs of the project, it was important to consider that in one of the boundaries there was a restriction for active anchors.

In this way, to achieve the maximum level of excavation, the following were built: a diaphragm wall with active anchors, tangent cantilever piles, a diaphragm wall supported by struts, micropiles accompanied by Soil-Nailing, a plastic wall, a slope with passive anchors and a shotcrete wall with active anchors.

The behavior of some of these stability systems were verified through the instrumentation installed in the project.

1.1 *Project description*

The project is part of an architectural complex of different buildings on nearby properties, for which 4 construction phases were considered. The project described in this work refers to phase 3 of construction which is a set of 2 mixed-use towers that share 4 parking basements, Figure 1.

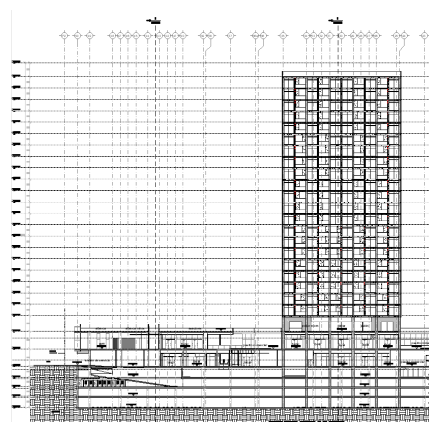


Figure 1. Schematic section of the project.

The property has an approximate area of 9,800 m^2, in which it was excavated up to 19 m deep to achieve the construction of the project basements.

It should be noted that the project is next to the fourth construction phase, which was not contemplated to be built at that time, therefore, it required that some stability solution be applied to divide both phases of the project.

2 GEOTECHNICAL CONDITIONS

Initially, a geotechnical model was defined according to the soil mechanics studies carried out on the property under study. However, once the excavation works began, as well as with the drilling of some diaphragm wall panels, the formation of the shale was found 6 meters above what was reported in the studies for the northern boundary. It is worth noting that in the same direction, the property had a natural ground slope of approximately 7 meters.

With the information found during the execution of the works, the geotechnical characterization was adjusted, as well as some stability solutions considered, according to the profile of the shale found in the north-south direction, Figure 2.

Figure 2. Schematic section of the project.

In this way, the geotechnical model (Table 1) was defined for the southern boundary, and according to the different areas of the project, the level of the shale was adjusted to carry out the different stability analyses.

Table 1. Geotechnical model.

Geotechnica l unit	Depth From (m)	To (m)	*γ (t/m³)*	*c (t/m²)*	*φ (°)*	*Ks (t/m³)*
U1	0.0	-5.0	*1.8*	3.5	20	1250
U2	-5.0	-11.5	1.6	5.0	32	4800
U3	-11.5	-16.5	1.7	6.0	34	5800
U4	-16.5	-	2.2	15.0	20	4500

Where:
U1 Sandy clay
U2 Gravels packed in sandy clay of medium compactness.
U3 Sandy clay with small shale fragments
U4 Shale
γ Volumetric weight
c Soil cohesion
φ Friction angle
Ks Horizontal ground reaction module

The mechanical parameters of the materials were obtained based on correlations and the experience obtained in the construction of the previous phases of the entire architectural complex.

Furthermore, the geotechnical studies detected the groundwater level at an average depth of 12 m; however, during the construction of the previous phases of this architectural complex, in some months of the year the level tended to rise to 10 m depth.

3 CALCULATION METHODOLOGIES

This chapter generally describes the calculation methodology for each of the stability systems that were built in the project.

3.1 *Stability analysis*

3.1.1 *Reaction modulus method*

The stability analyses for some areas of the project were carried out following a construction sequence that can be calculated using the PARIS program developed by Soletanche-Bachy. The program is based on the Reaction Module Method, allowing the simulation of soil-structure interaction.

The program models soils as an elasto-plastic material, following the Winkler hypotheses (Winkler, 1867), that is, the existence of a reaction module of the ground to a horizontal displacement, Figure 3.

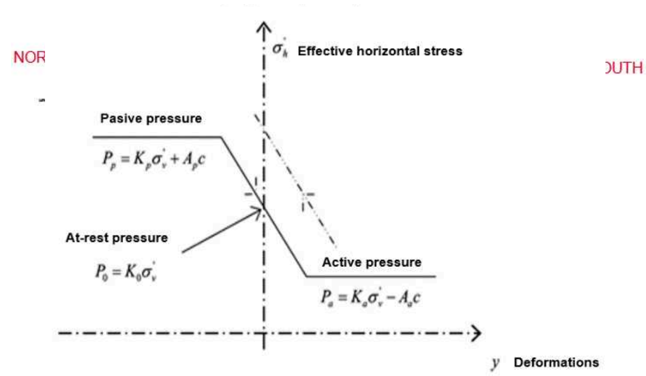


Figure 3. Stress-strain behavior used in the analysis model.

The thrust coefficient at rest is calculated as (Jaky, 1944):

$$K_0 = 1 - \sin sin\,\theta \quad (1)$$

Where:
K0 coefficient of lateral earth pressure

The values of the active-passive coefficients consider the force obliquity equal to zero in active and equal to -2/3φ in passive (Caquot and Kérisel, 1948). In this way, the program obtains the active and passive lateral earth pressure, based on the horizontal displacements of the wall, as well as the deformations and mechanical elements in the wall for each phase.

3.1.2 Fine Element Method

The stability analyzes for other areas of the project were carried out by the finite element method, using the PLAXIS 2D program. The model was carried out in PLAXIS 2d because it has constitutive models that allowed the behavior of the soils and the structures that were built to be adequately simulated.

The program allows the analysis to be carried out considering construction stages, where the state of stress at the end of a stage is the one used for the next stage of analysis.

The behavior of the soil strata was simulated using the Hardening Soil constitutive model. Likewise, regarding the deformability parameters, in general, it considers three moduli which are a function of the conditions to which the soil will be subjected, Table 2.

Table 1. Moduli for each Geotechnical Unit

Geotechnical unit	*E_{50} (t/m²)*	*E_{oed} (t/m²)*	*E_{ur} (t/m²)*
U1	4,000	3,200	12,000
U2	9,000	7,200	36,000
U3	12,500	10,000	50,000
U4	75,000	60,000	300,000

Where:
E_{50} secant modulus corresponding to 50% of deviatoric stress at failure
E_{oed} tangent oedometric modulus

E_{ur} unloading-reloading sitfness

It was considered to perform an appropriate finite element mesh to ensure that the results were reliable. In addition, the lower borders were restricted horizontally and vertically, while the lateral borders were restricted only horizontally.

The behavior of the structural elements was simulated using a linear elastic constitutive model, which only requires the elastic modulus (E) and the Poisson's ratio (v), which are elastic stiffness parameters.

3.2 *Active anchor design*

The calculation of the bulb length is based on the Bustamante method (Bustamante, 1985, 1994). This method calculates the ultimate tension capacity of the anchor (Tu) as:

$$T_u = \pi \alpha D_d L_s q_s \quad (2)$$

Where:
α coefficient whose value depends on the soil and the injection technique.
Dd drilling diameter.
Ls bulb length
qs soil limit adhesion

Applying this method, the capacity at tension service is obtained by applying a Safety Factor equal to 2.

Regarding the free length, it is mainly designed considering that it is sufficient for the anchor bulb to be behind the sliding surface, which can be calculated by means of traditional limit equilibrium methods that ensure the stability of the slope.

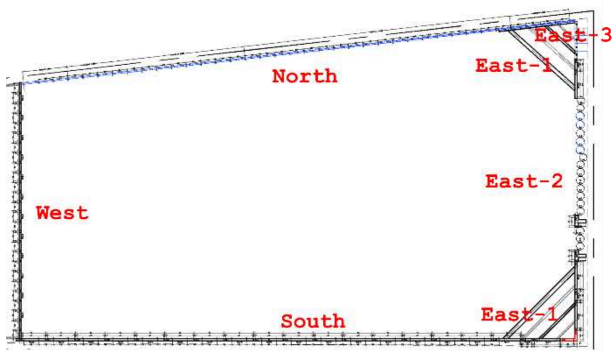
In addition, a minimum length must be considered that allows the initial blocking of the tension force, considering the losses due to a deformation recovery of the strand steel.

The length of 5.0 m ensures that there will be no loss of the blocking load due to recovery of the elongation of the structural elements of the anchor.

Finally, the design of the number of strand cables to be used in the anchors considers that the service load (CS) does not exceed 75% of the yield stress of the strands (P).

4 EXCAVATION STABILITY SOLUTIONS

In terms of value engineering and with the objective of having the same Working Platform Level (WPL) for the execution of the excavation stability work, a limit equilibrium analysis was carried out to know the maximum height for a first excavation throughout the property, so the results permitted a maximum slope of 80°. The results obtained from these analyzes showed that a cut of up to 7.5 m could be made, which would only be protected with shotcrete.



Based on the Working Platform Level, different stability solutions were proposed according to the needs of the project and the geotechnical conditions.

Approximately considering the cardinal points, Figure 4 shows the different areas to which a certain excavation stability solution was applied.

Figure 4. Project zones for different stability solutions.

4.1 *Northern Zone*

As previously mentioned, soil mechanics studies indicated that the shale level were shallower, which is why initially a solution had been proposed using a diaphragm wall and the construction of anchors with a higher blocking load. However, the shale level was found at a level close to the WPL, therefore, the stability system was adjusted. The initial solution using a diaphragm wall.

However, the stability of the excavation was carried out using a 0.25 m thick of Shotcrete Wall, accompanied by 3 levels of anchors. It should be noted that the first level of anchors was built on the slope with an inclination close to 80°. This first level of anchors was supported on a reinforced concrete reaction footing.

The stability check was carried out using traditional limit equilibrium methods and, in addition, verification was carried out using finite element models, Figure 5.

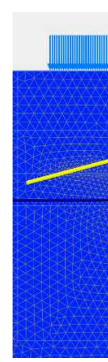


Figure 5. Calculation of Safety Factor in the last stage of excavation.

The first anchor level was built on alluvial soil with blocking tension up to 1500 kN, while the other anchor levels were built on shale with blocking load of 700 kN, Table 3.

Table 3. Characteristics of anchoring systems, northern zone.

Anchor level	Strands	Slope (°)	Blocking Tension (kN)
1	9T15	15	1500
2	5T15	15	700
3	5T15	15	700

In this way, the northern boundary managed to stabilize to reach 19 m depth, Figure 6.

Figure 6. Shotcrete wall and active anchors in the northern area of the project.

4.2 *East Zone*

Anchors could not be built in this area, and in addition, the shale level has a descending profile in a north-south direction, therefore, the following stability solutions were proposed.

4.2.1 *East-1*

In these corners it was decided to support the wall with struts. In the south corner, the struts were supported by a 0.5 m thick diaphragm wall, while in the north corner the struts were supported by the 0.25 m thick Shotcrete Wall.

The analysis of the construction sequence was carried out using the PARIS program developed by the Soletanche-Bachy Group.

With this analysis, Figure 7, a safety factor was obtained to ensure stability of the excavation.

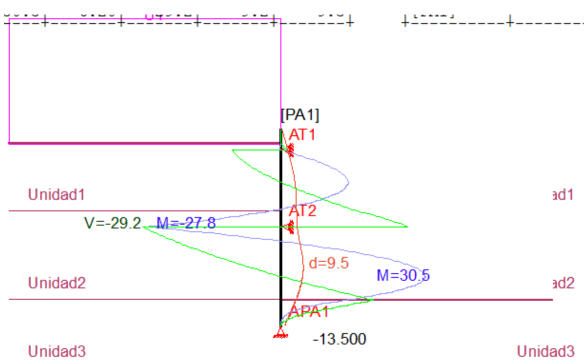


Figure 7. Deformations (mm) and mechanical elements (Moment in t-m and Shear in t) in the construction stage of the Milan wall supported on struts.

In the corners of eastern boundary, 4 struts were placed per level, where the longest strut was 22 m, Figure 8. The sections of the struts varied depending on the length of the corners. buckling and the acting thrust, where the largest diameter die was 36”.

Figure 8. Shotcrete wall and struts in the northeast area of the project.

4.2.2 *East-2*

Prior to adjusting the shale level, a solution using a diaphragm wall with a buttress (“T” section) had initially been proposed, in such a way that the wall will function as a cantilever element. However, for the resistance that the shale has, the wall drilling equipment is not adequate, therefore, the stability solution was adjusted by constructing a screen of tangent piles of 1.8 m in diameter.

These piles were also designed as a cantilever element, that is, it will not have any other element accompanying it for stability, therefore, they were planted 12 m below the Maximum Excavation Level.

The analysis of the construction sequence was also carried out using the PARIS program, in which it was analyzed as a continuous wall of rigidity equivalent to the pile screen.

The flexural reinforcement of the piles was analyzed using an interaction diagram (Figure 9) for which it was verified that the calculated resistant moment was greater than the acting moment calculated using the PARIS program.

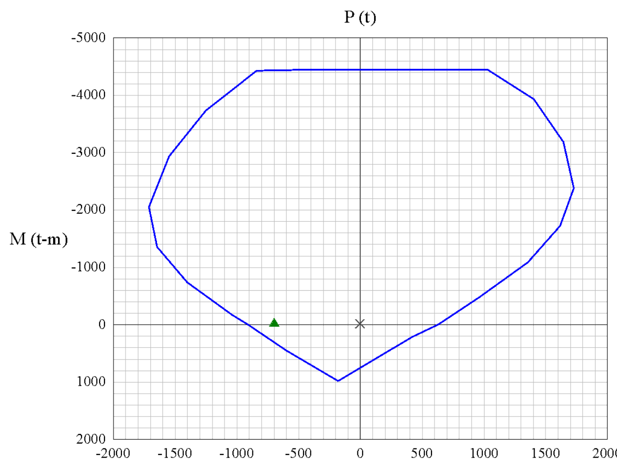


Figure 9. Interaction diagram for flexural reinforcement of tangent piles.

Figure 10 shows the tangent piles built when the excavation was 14 m deep.

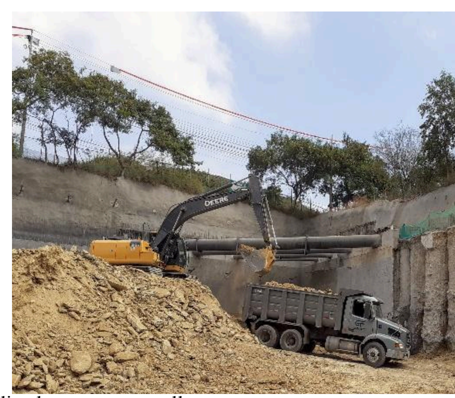


Figure 10. Screen of tangent piles and die-shot concrete wall.

4.2.3 *East-3*

In this area the shale level was very shallow, and the stability of the excavation was given by the struts to be placed in that area.

However, knowing that shale is usually formed by blocks that can slide mainly when in contact with water, it was decided to place micropiles and short passive anchors (Soil Nailing) with the aim of avoiding these possible slides. The anchors were 3 m long, which were within the project site.

In addition, a beam of concrete was built to join the Shotcrete Wall and the micropiles. In this way, the micropiles could resist the own weight of the wall and the passive anchors prevented possible slippage of shale blocks, Figure 11.

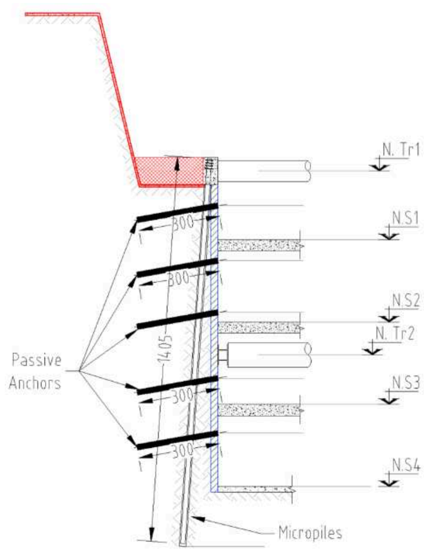


Figure 11. Soil-Nailing and Micropiles Scheme.

It should be noted that the micropiles had an inclination with respect to the vertical with the objective of, due to the possible deviation of the drilling, ensuring that the micropiles were outside the excavation area.

4.3 *South Zone*

In this boundary, the stability of the excavation was analyzed using a 0.5 m thick diaphragm wall, with 2 levels of active anchors. There were two types of anchors, which depended entirely on the overload generated by the ground behind the wall. That is, due to topographic conditions, the soil that existed above the WPL had an upward slope starting from the corner of the project towards the west side.

In this way, the type of anchors in this project area is shown in Table 4.

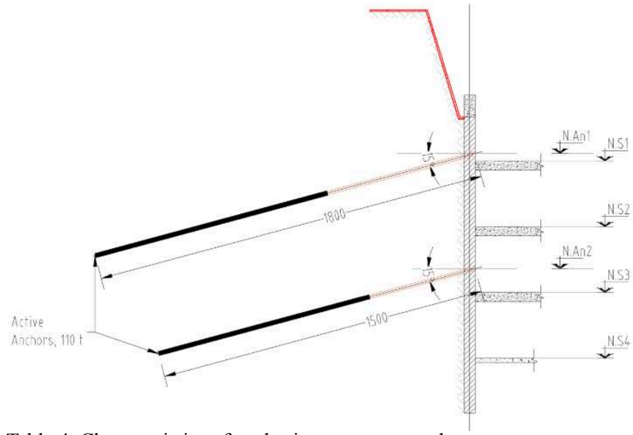


Table 4. Characteristics of anchoring systems, southern zone.

Anchor level	Strands	Inclination (°)	Tension load (t)
3 level anchors zones			
1	5T15	15	80
2	5T15	15	80
2 level anchors zones			
1	7T15	15	110
2	7T15	15	110

To illustrate the 110-t anchor system, Figure 12 presents a schematic section. Figure 13 then shows the maximum values of horizontal deformation, shear force, and bending moment at a specific construction stage.

Figure 12. 110 t anchor system, South Zone.

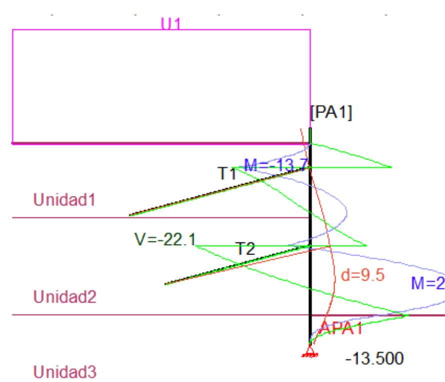


Figure 13. Deformations (mm) and mechanical elements (Moment in t-m and Shear in t) in the construction stage for diaphragm wall with 110 t active anchors.

With the analysis carried out using the PARIS program, a safety factor against kick failure of 1.7 was obtained, which is satisfactory to ensure stability.

4.4 *West Zone*

In this area, the construction of Phase 4 of the architectural project is planned, therefore, only the construction of a plastic screen was considered, which has the function of preventing groundwater leaks into the basements of the Phase 3.

Furthermore, to give stability to the plastic screen, a berm-slope was formed, which was analyzed using limit equilibrium. The angle of the slope to ensure the stability of the excavation was 60°.

It should be noted that the level of displacement of this plastic screen was defined at the contact of the healthy shale, knowing that this material has low permeability.

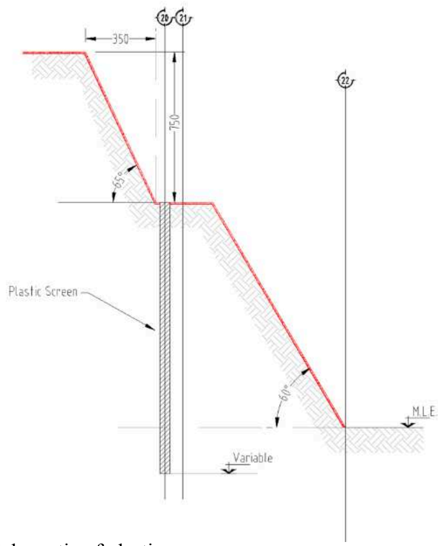


Figure 14. Schematic of plastic screen.

Figure 14 shows a schematic section of the plastic screen and the berm-slope that achieved stability to excavate at the Maximum Excavation Level.

The plastic screen was built using a cement-bentonite-water mixture for which a simple compressive strength of 2 kg/cm^2 at 90 days and a permeability of 1x10^{-5} cm/s was obtained.

Furthermore, Figure 15 shows the South zone with anchors of up to 110 t and the West zone with the slope and berm that protected and gave stability to the plastic screen.

Figure 15. Southwest zone, in location of active anchors of 110 t and berm-slope for plastic screen.

4 INSTRUMENTATIONS

Recognizing that an excavation would be carried out at a depth of 19 m in an important area of San Pedro Garza García, it was decided to place inclinometers to review the behavior of some boundaries throughout the excavation process.

Four inclinometers were installed to monitor movement. Two were placed in the tangent piles, and the remaining two were installed in the diaphragm wall panels (Figure 16).

Inclinometers were installed to monitor for any horizontal wall deformations and ensure the behavior remained within the parameters established by the interaction analyses.

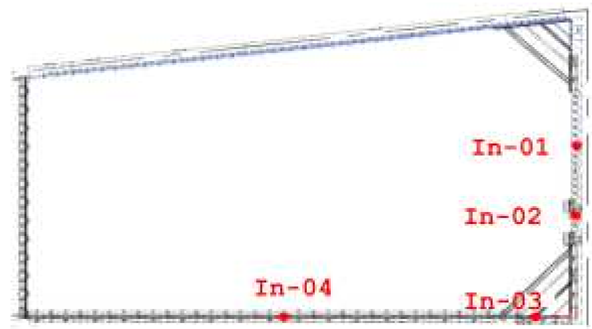


Figure 16. Inclinometer's location.

Once the inclinometers were installed, a first reading was taken, which would be taken as a basis for subsequent measurements. The measurements were made weekly, but additional measurements were made in case of excavation or applying tension loads to the active anchors.

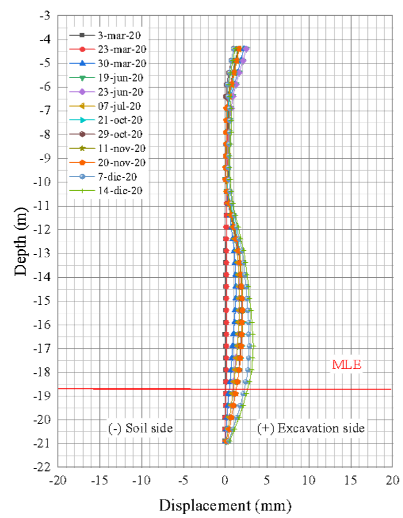


Figure 17. Measured displacements for In-04.

The monitoring program lasted approximately 4 months. The results showed that the diaphragm wall deformations matched the interaction analysis predictions, while the tangent pile deformations were lower. Table 6 summarizes the maximum displacements recorded by each inclinometer compared to the analytical values.

Table 4. Obtained results for each type of containment system.

Inclinometer	Type	dx (mm)	d-real (mm)
IN-01	Tangent piles	28.0	7.0
IN-02	Tangent piles	28.0	7.0
IN-03	MM and struts	7.5	4.0

IN-04	MM and anchors	9.0	3.0

Where:
dreal refers to the maximum horizontal deformation measured in inclinometer.

To illustrate, Figure 17 shows the displacement record for inclinometer In-04. This inclinometer exhibited a maximum displacement of 3.0 mm, which remained comfortably within the acceptable range for maintaining stability.

The successful instrumentation program confirmed the positive performance of the constructed containment systems. This allowed for safe excavation to the project's designated maximum excavation level, as detailed in Figures 18 and 19.

Figure 18. Maximum Excavation Level from the West area of the project.

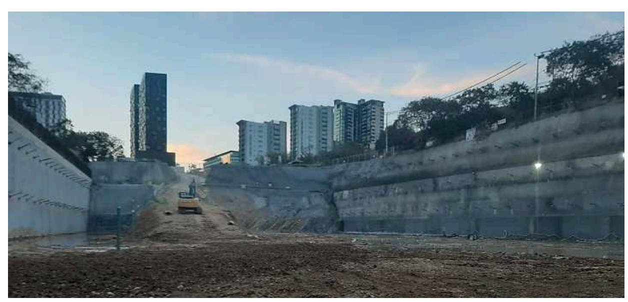


Figure 19. Maximum Excavation Level from the East area of the project.

6 CONCLUSIONS

In the metropolitan area of Monterrey, there are currently building projects with a significant number of superstructure levels, for which it is necessary the design and construction of parking basements that are regularly housed in alluvial soils and shale rock, and, below of the groundwater.

Under these conditions, the present work described the analysis, design, and construction of a 19 m depth excavation located in San Pedro Garza García, which under certain particularities of the project and geotechnical conditions required the use of different stability solutions.

In accordance with the geotechnical studies carried out on the property, a design of the stability of the excavation was carried out, which had to be adjusted according to the actual levels of shale that were found during the execution of the first excavation works.

In this way, to achieve the Maximum Excavation Level, diaphragm wall was used with active anchors, tangent cantilever piles, diaphragm wall supported on struts, micropiles accompanied by Soil-Nailing, plastic screen, slope with passive anchors and shotcrete wall with active anchors.

Project stability could be achieved through diverse solutions, even with variations in the stratigraphy. This flexibility allowed for excavation to the maximum project level while maintaining watertight basements. Instrumentation verified the behavior of specific stability systems by monitoring deformations. The measured deformations closely matched the predictions from the pre-construction analyses.

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