Feasibility of Diaphragm Wall with post-tensioned Anchors in Non-Controlled Fill Material

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ABSTRACT

Deep excavations associated with the construction of underground parking garages for major corporate buildings create significant challenges for geotechnical engineers, especially when the subsurface material consist of non-controlled fill. This paper provides a detailed description of a project located in Santa Fe, Mexico City, Mexico, where two corporate towers with 26 and 12 stories and 8 basement levels were built on a 1970s landfill site. Based on the maximum depth of excavation and local restrictions on deformations for major road arteries located in close proximity to the project site, the retaining system selected for the project consisted of a diaphragm wall reinforced with post-tension anchors. In addition to geotechnical drilling and sampling, results of three full scale pull-out test completed at the north end of the project site are discussed. From the load tests, cable elongations were recorded to be less than 1\% of the bonded length of the anchors.

INTRODUCTION

Deep excavations in Mexico City are often carried out in densely populated and congested urban areas. The sites are usually surrounded by buildings or principal road arteries. Despite construction precision and monitoring efforts, the excavation process generates ground settlement and horizontal displacement which could result in undesirable damage to the adjacent structures (Peck 1969). Poor subsurface condition at the excavation site is another factor to be considered for the selection of the retaining system for deep excavations. The project described in this paper represents a case where a 27.0m (89ft) excavation is carried out in a non-controlled fill material with the presence of a major road artery located in close proximity to the project site. Case histories in Mexico City and local restrictions on horizontal deformations have shown that the use of diaphragm walls is appropriate (Tamez 2001) for deep excavations and poor subsurface condition. However, horizontal pressures acting on the diaphragm wall
can exceed the structural capacity and the stability of the retaining structure. In such case, the diaphragm wall can be reinforced using post-tensioned anchors.

This paper further presents a brief background of the project site and the subsurface conditions determined from geotechnical exploration followed by the analysis of the horizontal pressures acting on the diaphragm wall and the design of the post-tensioned anchors implemented as a reinforcement for the diaphragm wall. In addition, results obtained from three full scale load tests are presented and discussed.

SITE AND PROJECT CHARACTERISTICS

The “Centro Corporativo” project consisted of two towers for general office spaces, referred to as Tower A and B with 26 and 12 floors, respectively. Both structures were designed with eight levels of basement for underground parking space with depths varying from 21.0m (69ft) to 27.0m (89ft) measured from the sidewalk level, Figure 1 and 3. The structural element for the super and substructures consisted of reinforced concrete beams and columns with slabs and peripheral walls. Considering structural load combinations, Tower A was built on deep foundations consisting of drilled shafts with maximum depth of 20.0m (65ft) measured from the finish floor level of the basement and diameters ranging from 0.8m to 1.5m (2.5 and 5ft). Tower B, was designed to be constructed on spread footings and continuous footings for columns and walls, respectively, Figure 1.

Figure 1. Plan view for Towers A and B distribution and Diaphragm walls
SUBSURFACE CONDITIONS

To better understand the subsurface condition and the site characterization associated with the project site, aerial photos of the region were reviewed to grasp a feeling about the historical development of the site. In addition to aerial photos, geotechnical explorations were executed at the project site. From photo interpretation and results of exploration the subsurface condition and stratigraphy are described below.

Aerial photos

The project site, is located in Santa Fe, West of Mexico City. According to existing archive and aerial photos, during 1970s the site was part of a sand mine exploitation project. The sand mining process was carried out creating open pits which resulted in deep artificial valley. From 1970s to 1990s this valley was filled with non-controlled material for the construction of a State Highway (SH) 150 connecting Mexico City to Toluca. Based on the photo interpretation, the project site is located to the north of the SH150 and directly on top of the non-controlled material used to fill the valley. This development and site modification can be observed in Photo 1.

![Photo 1a](image)

![Photo 1b](image)

**Photo 1.** a) Project site during 1970s b) Project site 1990 (Tamez-INEGI, 2000)
**Geotechnical Exploration**

The field exploration consisted of eight standard penetration test (SPT) borings and four 1.20m (4ft) diameter vertical shafts completed to maximum depths of 10.0 and 30.0m (33 and 100ft). To better delineate the transition from non-controlled fill material to natural blue sands, 50 flight auger borings were completed at the east side of the project following the center line of the proposed diaphragm wall, Figure 2.

![Diagram](image)

**Figure 2.** Approximate Boring locations at the site
From the field exploration the subsurface condition at the site can be described as a non-controlled fill material extending to depths of 7.0 and 27.0m (23 and 89ft) where the thickest layer is encountered towards the north of the project site. This material consist of a mixture of a low plasticity sandy silt (ML) and silty sand (SM) with gravel and construction debris. In some borings, plastic bottles and wood fragments were encountered which is considered to be less than 3% of the fill deposit. Results of borings indicated that these fragments were not forming continuous layers which makes them different from a typical landfill formations. The water content of this fill ranged from dry to 20%. The water table was encountered at 18.8m (62ft) at the time of drilling which is considered to be a perched water table product of filtrations during rain season. The number of blow counts from the SPT borings ($N_{SPT}$) varies from 5 to 10 blows per foot close to the surface and increases to an average of 20 blows per foot at higher depths.

Beneath the non-controlled and heterogeneous fill material, a deposit of dense blue silty sands (SM) with less than 5% water content and $N_{SPT}$ of more than 50 blows per foot is located. Figure 3 illustrates a general section view of the project and the variation of the subsurface layers.

Figure 3. General Section view of the project and subsurface condition
EXCAVATION DESIGN

Due to the proximity of the state highway Mexico-Toluca and local restrictions on horizontal displacements associated with this principal road artery, the consistency of the non-controlled fill material, and the maximum depth of excavation, the sloping and benching method for the excavation process was not a viable solution. Considering these factors, the retention system was developed based on a diaphragm wall reinforced with post-tensioned anchors. The excavation was carried out in two main phases. Phase I was the construction of the diaphragm wall whereas phase II was when the excavation and installation of anchors took place.

*Horizontal Pressures*

Horizontal pressures were calculated by using equation (1) proposed by Rankine for active pressure conditions and considering a unit weight of 1.75ton/m$^3$ (110pcf) and an internal friction angle of 30° obtained from laboratory tests. In order to account the effect of the heavy equipment and tools during the construction process, a surcharge of 0.75 ton/m$^2$ (150psf) was considered for the calculations. To determine the required anchor tension force, the Rankine horizontal pressures were redistributed according to Terzaghi-Peck (1996) apparent horizontal pressure envelopes, Figure 4.

\[
P_h = (\gamma_m H + q) k_a - 2c \sqrt{k_a}
\]

Where $P_h$ is Horizontal active pressure in ton/m$^2$, $K_a$ is Rankine active pressure coefficient, $\gamma_m$ is the soil Unit Weight in ton/m$^3$, $H$ is the depth of the excavation in meters, and $q$ is the surcharge due to excavated soil in ton/m$^2$.

*Figure 4. Apparent horizontal pressure distribution envelope*
Diaphragm Wall

The construction of the diaphragm wall was carried out using guided equipment and proper instrumentation to ensure wall verticality. During construction, the stability of the diaphragm wall trenches was guaranteed by using a viscous fluid containing 60kg of bentonite per each cubic meter of water.

From construction axis 2 to 9, Figure 3, where the non-controlled fill extends beyond the maximum depth of the diaphragm wall, a reinforced extension wall was constructed to support the fill material. Passing construction axis 9 and moving towards the south of the project site, a confining reaction plate made of reinforced concrete was installed with two levels of post-tensioned anchors. This reaction plates guaranteed the movements presented at the tip of the diaphragm wall during the excavation of blue sands, Figure 5 and 6.

Figure 5. Front view of the diaphragm wall and confining wall with anchors
Figure 6. Diaphragm wall and confining wall with anchors

Post-Tension Anchors

Rankine critical plane failure was used for the analysis and design of the anchors. Considering the magnitude of the horizontal pressures acting on the retaining structure, the diaphragm wall was reinforced with post-tensioned anchors installed in 10.16cm (4in) diameter bores with 15° inclination from the horizontal. Each anchor was installed with 6 grade 270 steel cables working at 0.65f′_y. After installing the anchors, grout injections were carried out to fill the bonded length using a grout mixture with f′_c = 150 kg/cm^2 (2100psi). The bonded length of the anchors was obtained from equation (2) using strength properties for the non-controlled fill material obtained from laboratory test and grout injection pressure of 100 ton/m^2 (140psi).

\[
L_b = \frac{T_f}{\pi D N \tan \phi}
\]  

(2)
Where $T_f$ is the allowable anchor tension force of 275 Ton (550kips) with a FS=2, D is the bore diameter equal to 0.1016m (4in), N is the injection pressure, and $\varphi$ is the internal angle of friction 30° obtained from laboratory test.

In addition to the bonded length calculations, the soil-grout and grout-fiber adhesion was evaluated using equations (3) and (4) taken from ACI (2005). The lengths resulted from these equations were shorter than the results obtained from equation (1) meaning that the bonded length is developed enough to support the tension force and the anchor will not fail by soil-grout nor grout-fiber adhesion.

\[
L_{s-g} = \frac{T_f}{\pi D V} \quad (3)
\]
\[
L_{s-g} = \frac{T_f}{n \pi D_e U_p} \quad (4)
\]
\[
U_p = \frac{3.2 \sqrt{f'_c}}{D_e} \quad (4')
\]

Where $T_f$ is the allowable anchor tension force of 275 Ton (550kips) determined using a FS=2, D is the bore diameter equal to 0.1016m (4in), $V$ is concrete shear strength taken as 0.06$f'_c$, $D_e$ is the nominal diameter of the cables used in the anchor 6in, and $U_p$ is the equivalent shear stress at the grout-cable interaction.

Bonded lengths were determined to be behind the Rankine critical failure line and the un-bonded length is determined to be the required length to extend the anchors to the front of the diaphragm wall, Figure 7. Lengths and anchor levels are shown in Table1.

Figure 7. Bonded and un-bonded length of anchors
Table 1. Anchor Levels and Lengths shown per Axis

<table>
<thead>
<tr>
<th>Anchor Level</th>
<th>Bonded Length (m)</th>
<th>Total Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Axis 1 to 11</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Axis 11 to 21</td>
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<tr>
<td>NA-9</td>
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FULL SCALE PULLOUT TEST AND RESULTS

Prior to the construction of the diaphragm wall, due to uncertainties associated with the strength parameter obtained from laboratory tests performed on the non-controlled fill samples and the heterogeneity of the fill deposit, three full scale pullout load test were performed at the site. The anchors consisted of 6 steel cables grade 270 with a nominal diameter of 1.52cm (0.6in) installed in 10.2cm (4in) bore with 15° of inclination from the horizontal. The bonding length and the injection pressure were 15.0m (50ft) and 10kg/cm² (140psi), respectively.

All load tests were completed at the north end of the project site where the maximum depth of the non-controlled fill material was encountered, Figure 8. Each load test was completed in two phases. During Phase I, the anchor system was loaded (tensed) with two loading cycles of 25 and 40 tons (50 and 80kips) with increments of 5.0 and 10.0 ton (10 and 20kips), respectively. The main objective of Phase I was to arrange and align the steel cables as well as making the bonding length be exposed to initial stress, Figure 7. In Phase II, the anchor was exposed to full design load of 140ton (280kips) with load increments of 20ton (40kips). Maximum elongations registered from the pullout test were between 9 and 14cm (3.5 to 5.5in) with an average of 11.5cm (4.5in), which does not exceed the 1% required elongation of the total bonded length, Figure 9.
Figure 8. Loading and Unloading phase, Anchor Test 1

First Cycle Maximum 25tons

Second Cycle Maximum 40tons

Figure 9. Load-Elongation Curve for Anchor Tests 1 to 3
SUMMARY AND CONCLUSIONS

The project presented in this paper is the construction of two towers with 26 and 12 floors on top of 8 basement levels. The maximum depth of excavation varied from 21.0 to 27.0 m. The subsurface material consisted of three main layers of non-controlled fill material from the surface to the depth of 7.0 and 27.0m followed by a deposit of dense blue sands and very dense sands.

Due to space availability and restrictions on horizontal deformations for major road arteries, the retaining structure selected for the deep excavation was a diaphragm wall reinforced with post-tensioned anchors. Horizontal pressures for the design of the diaphragm wall and the bonded length of the anchors, were calculated based on index and strength properties of the non-controlled fill material obtained from laboratory testing. However, due to heterogeneity of the fill material and the magnitude of horizontal pressures, three full scale load test (pullout test) were carried out at the north end of the project. The results show that anchors installed in non-controlled fill material can be functional. However, compaction and relative density of the sand play an important role for the design of the anchors. In this case, an average $N_{SPT}$ value of 10 blows/foot for the superficial material and 20 blows/foot for deeper fill material was recorded. This medium granular fill material classified as ML to SM, resisted the tension force of 140 ton (280kips) with elongations between 9cm and 14cm which in average was less than 1% permissible elongation of 15cm (6in).

Approximately 750 anchors were installed for this project where only three of these were tested. However, the performance of the diaphragm wall interacting with the anchors during the deep excavation phase for the basement levels and the construction of the superstructure was the sign of feasibility of reinforced diaphragm wall in non-controlled fill materials such as the deposit encountered in Santa Fe, Mexico City.
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