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A Case History of Deep Excavation in Doha

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ABSTRACT

The Doha high-rise tower consists of a 4 level basement which required an excavation depth of 16m along the site perimeter. Minimization of impact of excavation-induced ground movements on the adjacent structures and underground utilities was one of the major considerations in the design and construction of the excavation and its temporary retaining system. The temporary retaining system initially consisted of anchored secant pile walls with toe grouting along the entire site perimeter. After excessive movement was detected on one section of the southern wall, inclined steel bracing was installed inside the excavation as a remedial measure to provide additional lateral support to the wall. This paper discusses the geotechnical aspects of the design and construction of the excavation and its retaining system, the analysis approaches employed to evaluate the conditions of the moving wall, and the remedial measures taken to prevent further wall movement.

INTRODUCTION

The Doha high-rise tower has 45 stories and stands 232m above the ground. It consists of a 4 level basement with a footprint area of approximately 16,000 m^2. The maximum excavation depth was about 16m along the site perimeter.

The pentagonal shaped project site is located at the east of Doha and is approximately 100m away from the Arabian Gulf shoreline. The southern side of the site runs along Al Corniche Street which is a major road in Doha. There are also other office buildings located adjacent to the project site. Minimization of impacts of excavation-induced ground movements on the adjacent structures, Al Corniche Street and the various utilities underneath it was one of the major issues needed to be considered in the design and construction of the excavation and its temporary retaining system.

The temporary retaining system initially consisted of anchored secant pile walls with toe grouting along the entire site perimeter. After excessive lateral movement was detected on one section of the southern wall, inclined steel bracing was installed inside the excavation as a remedial measure to provide additional lateral support to the wall.

This paper discusses the geotechnical aspects of the design and construction of the excavation and its retaining system, the analysis approaches employed to evaluate the conditions of the moving wall, and the remedial measures taken to prevent further wall movement.

GROUND AND GROUNDWATER CONDITIONS

A comprehensive ground investigation program was carried out on the site, including boreholes, various in-situ and laboratory tests. The ground conditions were assessed based on the investigation results and are discussed in this section.

Site Geology

The Qatar Peninsula geologically represents a part of the Arabian Gulf Basin. This basin is mainly composed of extensive carbonate sediments with different ages overlying the basement rocks and may reach up to 10 km in thickness. The outcropping rocks in Qatar are mainly referred to as Quaternary and Tertiary Ages. The geology of the site is mainly of the recent and Tertiary sediments Simsima Limestone and Rus Formation.

The boreholes carried out for the site showed that there are general similarities and continuities of the subsurface materials which consist of Hydraulic Fill, Marine Sand/Caprock, Simsima Limestone, Dukhan Alvelina, Midra Shale and Rus formation. The Midra Shale layer was encountered in all boreholes and was about 4.7m thick. This layer was known as impermeable.

Groundwater Level

Groundwater was encountered in all boreholes at different depths ranging between 2.25m and 4m below the existing ground level, which might vary due to the proximity of the sea shoreline to the site and the tidal fluctuation effect.

Permeability of Ground

Since excavation was to terminate in the Simsima limestone, several packer tests were carried out in this layer to determine its permeability. The packer test results indicated that the permeability of this layer was in the order of $5x10^{-5}$ m/s.

A pumping test was also carried out to assess the permeability of the Simsima limestone layer and the materials overlying it, the results of which could be used to determine the effect of dewatering on adjacent areas and to assess dewatering requirements during excavation.

The test included one 25m deep pumping well, 300mm in diameter, and eight 125mm diameter observation wells, four of which being 20m deep and the other four being 18m deep. The observation wells were installed at specified distances from the test well in four directions set at 90º to the test well (two observation wells at each direction).

The pumping rate selected for the test was approximately $64m^3$/hr. The discharge rate and the ground water levels in all the wells were recorded.

The test results showed that the permeability of the test depth was in the order of about 5.5×10^{-5}m/s, similar to that obtained from the packer tests, indicating a very permeable ground.

EXCAVATION RETAINING SYSTEM

The retaining system comprising of secant piles with toe grouting is discussed in this section.

Anchored Secant Pile Wall

The excavation depth of the basement area was 15.65m close to the basement outer boundaries. At the tower area and a few lift pit locations which are located away from the basement outer walls, the excavation was a few meters deeper.

The following points were considered in the selection of an excavation retaining system:

1) The ground above the impermeable Midra Shale layer is very permeable and recharge of groundwater from the adjacent sea is expected to be rapid;

2) Excavation induced ground movements need to be tightly controlled to minimize adverse impact on the adjacent structures, underground utilities and roads especially Al Corniche Street.

After evaluation of several options, an anchored secant pile wall option was considered appropriate and was adopted along the entire excavation perimeter. Due to piling rig limitations, the depth of the secant piles was limited to 20m which was found to be inadequate to cut off groundwater inflow, as will be discussed later in the paper. Consequently, a grout curtain below the toe of the secant piles was adopted as a groundwater cut off barrier. Details of the anchored secant piles with toe grouting are shown in Figure 1.

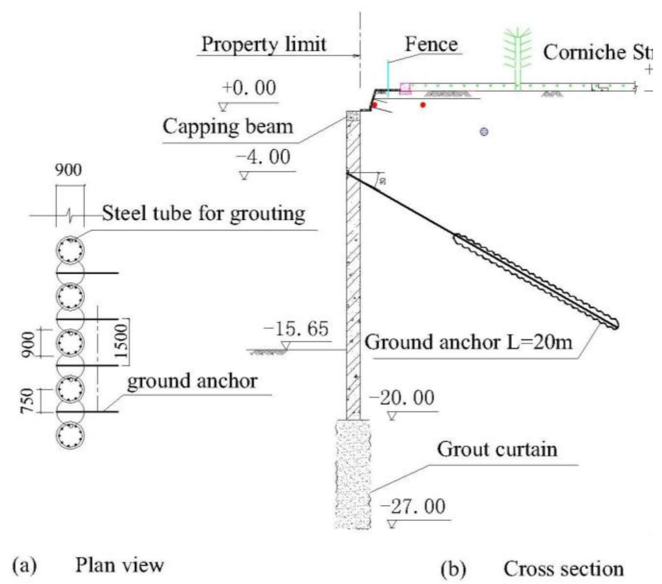

Fig. 1. Anchored secant pile wall with toe grouting

The anchored secant pile wall consisted of primary and secondary bored piles connected by a capping beam on the top. The primary bored piles were constructed of plain concrete, while the secondary bored piles were constructed of reinforced concrete. Both the primary and secondary piles were 900mm in diameter and 20m long. The neighbouring piles overlapped by a maximum of 150mm. The secant pile wall was supported laterally by one row of ground anchors connected by a wailer beam. The level of the anchor heads was at -4m, about 0.5m above the groundwater level expected at the time of anchor installation.

Design of Piles

The geotechnical stability of the anchored secant pile wall was analyzed using Plaxis 2D. The ground model was simplified as a loose sand layer overlying a Simsima limestone layer, with the adopted parameters being presented in Table 1.

Table 1. Adopted Geotechnical Parameters

Soil type	Layer thickness (m)	c' (kPa)	ϕ' (deg)	E_s (MPa)
Loose sand	8	0	30	10
Simsima limestone	17	100	38	500

The calculated maximum working bending moment and shear force of each secondary pile (i.e. reinforced pile) are presented in Table 2, together with the calculated maximum wall deflection. These forces were used for the structural design of the reinforced piles.

Table 2. Calculated Maximum Pile Responses

Bending moment (kN.m)	Shear force (kN)	Deflection (mm)
1,035	615	24

Correspondingly each reinforced pile was designed to have an ultimate bending moment capacity of about 1,500 kN.m.

The calculated maximum working anchor force was 697 kN per anchor and each anchor was designed to have an ultimate load capacity of about 1,465 kN.

Toe Grouting

Seepage analysis results, later confirmed by results of trial dewatering after installation of the secant pile walls, indicated that the 20m deep pile walls were not deep enough to cut off expected groundwater ingress into the excavation. Therefore, it was decided to install a grout curtain to form a groundwater cut off barrier between the toe of the secant piles and the underlying impermeable Midra Shale. The grout was to be injected through the vertical tubes pre-installed inside the secondary bored piles. The grouting tubes were spaced at 1.5m.

Grout injection was carried out in three steps at different spacings, 6m at first step, reducing to 3m at second step and 1.5m at the final step. This sequence was adopted to ensure that sufficient overlap between successive grout bodies could be achieved.

The amount of cement consumption was used to judge the grouting effect. A decrease in cement consumption with decreasing grouting spacing would indicate proper overlap between successive gout bodies. The adopted criterion was that, if the amount of cement consumption at the current grouting step was less than 75% of that at the preceding step, the overlap would be considered to be adequate.

Subsequent dewatering proved that this system of secant piles with toe grouting was effective in cutting off groundwater inflow.

EXCAVATION

The retaining system was working successfully during excavation except for a small portion of the southern wall which had experienced excessive movement, as discussed in this section.

Excavation Sequence

The excavation was carried out in the following sequences:

a. construct secant piles and capping beam;
b. carry out toe grouting;
c. dewater and excavate to -4.5m;
d. install wailer beam and ground anchors at -4m; and
e. dewater and excavate progressively to formation level.

Plate 1. Excavation to Level of -12m

Lateral and vertical ground/wall movements were monitored by inclinometers and settlement markers, respectively. Plate 1 shows the site condition when the excavation reached the level of about -12m.

Excessive Wall Movement

When the excavation reached the level of about -13.5m, excessive lateral wall movement was detected on part of the southern wall. Within a couple of days, the measured wall deflection had reached about 80mm which was significantly greater than the design limit of 25mm. Plate 2 shows the ground depression behind the wall.

Plate 2. Ground Depression behind Secant Pile Wall

A subsequent investigation found that the excessive wall movement had been caused by some loosened ground anchors.

Assessment of Moving Wall Conditions

In order to check the structural condition of the moving piles, it was necessary to estimate the maximum pile responses induced by the measured wall movement. Such a check may be undertaken using either numerical methods or published design charts as discussed in Chen & Poulos (1997, 1999, 2001) and Poulos & Chen (1996a, 1996b).

As a first step reacting to the emergency situation, the simple design charts proposed by Chen & Poulos (1997) were used to back calculate the maximum bending moment induced in the pile due to the measured wall movement. The procedure is illustrated below.

The input pile and soil parameters required for these design charts include pile diameter (d), pile bending rigidity (E_pI_p), pile length (L), soil movement at surface level (s_o), thickness of moving soil layer (z_s) and soil Young's modulus.

Based on the measured inclinometer data, the lateral soil movement profile was closely simplified as a linear profile, reducing from 80mm at the pile top level to zero at the level of -8m. A uniform soil was assumed.

For each reinforced pile, d = 0.9m, L = 20m and $E_pI_p = 1.9 \times 10^6$kN.m^2. For L/d = 20/0.9 = 22.2, $K_R = E_pI_p/E_sL^4 = 1.2 \times 10^{-3}$ and $z_s/L = 8/20 = 0.4$, the maximum bending moment can be calculated as $M_{max} = 0.26 \times 10{,}000 \times 0.9^2 \times 8 \times 0.08 = 1348$ kN.m which is close to its ultimate capacity of 1,500 kN.m, indicating that the concerned piles had reached a critical condition.

The above simple estimation was subsequently verified by a more detailed numerical analysis via Plaxis 2D. By matching the measured wall movement profile, the calculated maximum pile bending moment is about 1400 kN.m.

Remedial measures

The excavation was immediately stopped and the concerned wall section backfilled to the level of about -10m as an emergency measure to prevent wall collapse.

The loosened ground anchors were subsequently re-stressed to their design load level. Furthermore, it was considered necessary to strengthen the concerned wall section by additional lateral support prior to resumption of excavation works, in order to prevent further wall movement.

After option evaluation, it was decided to adopt a strutting system consisting of inclined steel tubular pipes having an external diameter of 600mm and a wall thickness of 10mm, installed at an inclination angle of 30^o to the horizontal and at a spacing of 4.3m, see Plate 3. The top of these bracing pipes were connected by a steel wailer beam fixed to the wall at the level of -10m. The bottom of the bracing pipes was supported by a foundation consisting of inclined steel sections and the 750mm diameter bored piles which had been installed as tension piles to support the basement.

Plate 3. Installed Strutting System

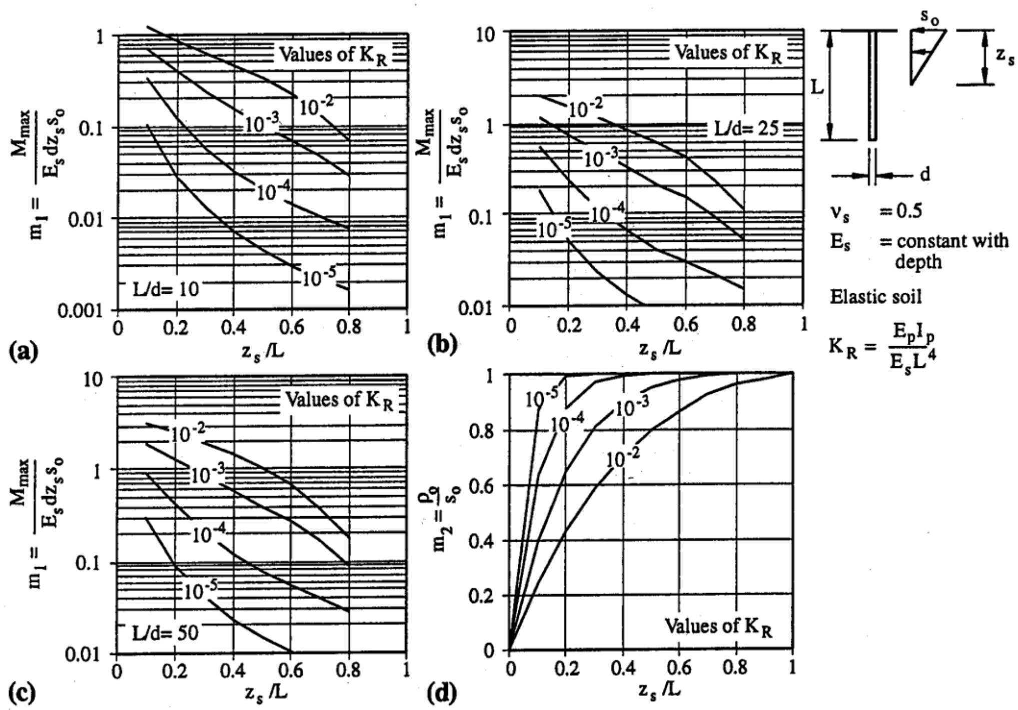

Fig. 2. Elastic solutions for unrestrained free-head pile in uniform soil (linear soil movement profile) (after Chen & Poulos, 1997)

Although this strutting system had caused inconvenience to the subsequent excavation and construction of the basement slab, it played a critical role in safeguarding the retaining system.

Following installation of the additional strutting system, the excavation was continued successfully to completion.

CONCLUSION

The temporary retaining system consisting of anchored secant pile walls with toe grouting was proved to be effective in not only retaining the excavated ground but also cutting off groundwater inflow.

However, due to careless installation of ground anchors, excessive wall movement was detected on one of the wall sections. This incident shows that it is imperative to implement proper site supervision and monitoring for excavation works.

The published design charts were found to be useful in providing a prompt evaluation of the condition of the moving wall, especially considering the emergency situation whereby a detailed numerical analysis was not immediately available due to time constraints.

An additional strutting system was adopted to provide additional lateral support to the moving wall prior to resumption of excavation works, in order to minimize further wall movement. Subsequently, the excavation was continued successfully to completion.

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