Performance of a Berlinoise wall in a marly formation

Performance d'une paroi Berlinoise dans une formation marneuse

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ABSTRACT: The paper deals with design sensitivity analyses, ground-wall interaction and monitoring of a 16m deep Berlinoise wall that supports a marly formation. The detected small wall crest displacements and anchor load fluctuations assured a satisfactory performance of the retaining system. Although the sophisticated FEM gave results that were in general agreement with monitored values, the simpler one-dimensional Winkler model is considered promising at the preliminary stage of design, where a rough estimate of the soil-structure interaction is needed; the limit equilibrium method can still be used as a tool to conduct less costly sensitivity analyses, as well as to check that failure does not prevail.

RÉSUMÉ: Le travail resumé dans cet article s' occupe du calcul paramétrique de l' interaction terrain-structure et des mesures en vraie grandeur effectueès sur une paroi Berlinoise de 16m de profondeur dans une formation marneuse. La détection de petits déplacements an sommet de la paroi et de faibles variations de charge aux tirants temoignent de la bonne performance du système de soutènement adopté. Bien que la méthode sophistiquée des éléments finis a produit de résultats en accord avec les mesures in situ, le plus simple modèle unidimensionel de Winkler s'avare intéressant à la phase prélimimaire du calcul où on désire avoir une idée grossière de l' intéraction terrain-structure; la méthode de l' équilibre limite reste cépendaut un outil précieux tant pour l'exécution de calculs paramétriques de coût moderé que pour le contrôle du danger envers la rupture.

1 INTRODUCTION

The construction of an underground 3-storey car parking that would cover a nearly 4,700m^2 block area of polygonal shape in the littoral zone of Athens, Greece, necessitated the design of a rigid retaining system to control ground mouvements. The vertical faces of the excavation were nearly 16m deep and were to be cut inside Pliocene marly formations. An earlier geotechnical investigation (1990) comprising eight boreholes, a deep trench, continuous sampling, in situ permeability and penetration testing, and conventional laboratory tests revealed the presence of nearly continuous, subhorizontal strata consisting of a weathered marly crust in the form of stiff silty clay (CL), with varying amounts of sand and gravel down to a depth of 7m, followed by an argillaceous sedimentary rock in the form of mudstone with a high percentage of calcareous material (locally>50%), well known as the Piraeus marl; the stratigraphic columm was occasionally interbedded with loose conglomerates and it was overlain by a 2m thick fill, while the ground water level was located at an average depth of 6.5m from the ground surface.

Given the extent of the retaining system to be used (3,500m^2) it was felt from the very beginning (1998) that the main impetus to lowering the total cost of the project was to dwell upon the selection of a well performing wall, as sensitive structures including a high-rise building, a heavy-traffic avenue and a flyover founded on piles were surrounding the facility. It was thus decided to conduct design sensitivity analyses of the retaining system, focusing on a favorite scheme that consists of an anchored soldier pile wall (Berlinoise wall) with varying wall geometry; in that scope, a range of values of the geotechnical design parameters were first introduced into a Limit Equilibrium model of analysis, followed by ground-wall interaction analyses with the aid of a Winkler model and a Finite Element model. On the other hand, given the growing need of field evaluating the design criteria, an instrumentation programme was implemented to observe wall movements and anchor load fluctuations during construction of the excavation faces; contingency plans could then be triggered in case of emergency.

The paper deals with design and instrumentation aspects of the project, it evaluates the results of the performed sensitivity analyses on the basis of an economic criterion, while it critically presents the analytical tools used by comparing the output obtained from the ground-wall interaction analyses with the monitored data.

2 DESIGN AND INSTRUMENTATION ASPECTS

The Berlinoise wall examined in the analyses consisted of bored piles with a nominal diameter of 800mm, spaced between 1.0 and 2.8m center to center. Each soldier pile was laterally supported by one to four rows of prestressed anchors at an angle of 15° to the horizontal; the bonded length varied between 6 and 10m, while the free anchor length between 4 and 10m decreasing from the top downwards. Anchor head distance of the consecutive rows from the wall crest was taken to vary ±10% the fixed distance of 18,40,64 and 85% the excavation depth. In the special case of one row of anchors, the presence of a reinforced concrete continuous pile cap (1000x800mm^2) would act as bearing element for the anchors. A shotcrete layer of 80 to 150mm of thickness (reinforced with a wire mesh) was to be cast between the piles to transfer earth pressures onto the piles.

As the ground water is present in the major part of the retaining faces, risk of transient flow towards the excavation and subsequent softening of the marl due to solution of the carbonate content is apparent; emphasis was then shifted to the design of a drainage system that should provide continuous management of the underground water. Given the permeability of the ground ($3x10^{-5}$ to $1x10^{-9}$ m/sec) it was decided to install a ground water lowering system consisting of deep (20m) drainage wells (Φ 800 mm) outside the excavation, one at each side, equiped with a pump at their bottom; vertical strip drains are also to be placed between the piles, in contact with the excavated soil face and below the shotcrete layer, while weepholes will be included in the shotcrete. With the aid of flow net techniques it was estimated that partial drawdown caused by drainage would form a cone of depression extending horizontally for a distance approximately equal to the depth of excavation; no risk of incipient settlements of the surrounding structures should then be anticipated.

Assuming isotropic permeability, water pressures were estimated approximately one-half of the hydrostatic pressure starting from the water table, although at the most critical depths, near the bottom, the pressure was about 40% of the hydrostatic. In addition, it was assumed that at any intermediate stages of excavation reduction of water pressure lags at least 2m behind the excavation; hence, the assumed water pressure was from above the table or 2m above the interim excavation level, whichever is greater.

According to the SI Report the geotechnical parameters ranged as follows:

Table 1. Geotechnical parameters (SI Report)

Parameter	Silty Clay	Piraeus Marl
γ (KN.m^{-3})	19-21	20-23
w_L (%)	25-48	22-49
PI (%)	8-44	9-28
w (%)	8-25	11-21
CF (%)	5-28	8-33
c (Kpa)	10-80	0-169
Φ (°)	19-42	13-56

where γ:bulk modulus, w_L: liquid limit, PI:plasticity index, w:water content, CF: clay fraction (<0.002mm), c: cohesion intersept, Φ:angle of shearing resistance (from UU and CU Direct Shear tests and UU Triaxial Compression tests).

Considerable scatter of the strength parameter values seems to result in the marly formations (weathered crust, Piraeus marl). Due to this well known phenomenon controversy had existed for many years between local engineers over the design values to be introduced into analysis; the main questions resided on the 'soil-like' or 'rock-like' behaviour of the marl. Regional praxis suggests that this formation is non-homogeneous, anistotropic, easily weatherable, overconsolidated and/or cemented, usually inactive and locally very sensitive, fissured and slickensided (Costopoulos, 1989); RQD index normally varies between 15% and 55% indicating an average discontinuity spacing as close as 50 to 80mm. As a consequence, the release of horizontal and vertical stresses by the excavation process initiates, by relaxation and opening of fissures, time-dependent softening and leads, in a short time, to drained conditions which, however, are partially restrained by shotcreting the face. On the other hand, the rate at which the clay softens in front of the wall should be much slower than behind, because of the increase in horizontal stress acting on the passive soil wedge from the wall loading; undrained strength in extension might then be similar to that in compression or it might be as low as half of that in compression. Cairncross & James (1977) showed that stiffnesses in 'active' shear can be as large as 10 times greater than those in 'passive' shear. Moreover, under conditions of lateral unloading and partial drainage, the in situ marl is expected to mobilize lower strength and higher deformation with respect to the undrained compression loading conditions in the triaxial cell; parameters derived from UU compression tests could thus lead to an unconservative design and greater than expected displacements. In fact, induced stress anisotropy, especially under undrained conditions, is important when determining the global stability, basal heave stability and deformation of a deep excavation; the computed factor of safety in a material with a likely ratio of extensive to compressive strength of 0.5 might be about 20% less than that using undrained compressive strengths only; the typical recommended value of 1.3 could thus be more like 1.0 to 1.1. Therefore, it seems unrealistic to design temporary tie-backs, especially Berlinoise walls in hard soil/soft rock formations, using fully-drained effective stress analysis or conventional undrained analysis. It is also to be noted that, as the above marly formations are expected to behave at least as overconsolidated hard soils, with undrained strength behaviour and stiffness greatly affected by the maximum past pressure ever experienced, values of the angle of shearing resistance might be of the same order of magnitude in terms total or effective stress, while cohesion in undrained analysis should undoubtely be credited with greater values in the first than in the second case. According to Ladd (1967) the undrained shear strength determined by Direct Shear tests (DSS) is approximately equal to the average of the strength measured in Triaxial Compression and Triaxial Extension tests; on the other hand, the average strength acting along a typical failure surface is similar to the DSS strength (with a few corrections to adjust for strain compatibility and differences between triaxial and plane strain conditions). The correct undrained strength is ultimately determined by appeal to large scale behaviour (Morgenstern, 1967).

Based on the above considerations, personal experience of the writer in the design and monitoring the performance of temporary tie-backs in these formations (Costopoulos et al, 1981, 1985, 1993, 1997), as well as the evaluation of prior geotechnical testing on these materials, suggest the use of undrained stability analysis with 'calibrated' parameters to reflect the above phenomena, as follows:

Table 2. Calibrated parameters for stability analyses

Formation	c (KPa)	$\Phi^{(o)}$	E(MPa)
Silty clay	15-25	23-36	15-30
Piraeus Marl	30-80	23-40	30-50

where E: deformation modulus. Poisson's ratio is usually considered constant (v=0.3).

In the present case, the bulk modulus was given a constant value of 22 KN·m^{-3}, while the fill was attributed the following design parameters: γ=20KN·m^{-3}, c=0, Φ=25°, E=10MPa, v=0.3.

The analytical tools used were the computer Codes CWALSHT (1990) for the Limit Equilibrium analysis (fixed earth support), DENEBOLA (1982) for the Winkler elastoplastic analysis and PLAXIS V6.31 (1996) for the Finite Element analysis (plain strain, triangular elements, elastoplastic hyperbolic behaviour of the ground, elastic behaviour of the steel and concrete members). In all cases a uniform surficial surcharge load of 20 KN.m^{-2} was taken into account.

Friction between the marly formations and the piles was taken equal to 10°, while default safety factors were given the values of 1.5 for the passive earth pressure in front of the wall, 1.3 for failure along any potential surface and 2.0 for the bond between the anchor and the marly formations.

Monitoring consisted in the detection of horizontal and vertical wall crest displacements, as well as the measurement of load fluctuations on six control anchors. In the first case, survey bolts were attached at eight distinct points on the wall and readings were made with the aid of a precision theodolite and an INVAR tape; the instrument was set successfully on four reinforced concrete pedestals located at the corners of a quadrilateral outside the excavation area and each measurement point was aimed. Direction measurements were made in four periods and the mean values were finally retained. Data were then processed with the aid of a computer programme based on a mathematical model of least squares adjustment with least constraints (Costopoulos, 1985). Results were given in the form of displacement vectors and of typical absolute error ellipses at all points; accuracy of the measurements was 5x10^{-4}m. Anchor load fluctuations were monitored using flat hydraulic cells with a nominal precision of 10KN; however, air temperatures were recorded throughout, as there would appear to be a thermal effect on the apparent load due to susceptibility of the load cells to temperature variations.

3 RESULTS OF THE ANALYTICAL COMPUTATIONS

Several hundreds of stability analyses were conducted on different wall geometries using the Limit Equilibrium method (CWALSHT Code); the purpose was to examine the sensitivity of the factor of safety against local failure or deep seated movement to changes of strength parameters, due to variations in soil/rock quality along the anchorage zone, inside the retained mass and/or below the excavation level in front of the wall.

Interpretation of the computational data corroborated practical evidence; the following remarks are nonetheless instructive:

- increased strength parameters produced lesser anchor loads, anchor lengths, pile bending moments and depth of pile embedment
- lower anchor lengths are needed to avoid failure along cylindrical surfaces than along composite plane surfaces
- the above mentioned trends become less explicit for Φ>25° and c>20KPa, where an almost linear reduction with strength parameters can be sustained
- when c>50 KPa anchor lengths and loads remain practically insensitive to increased values of Φ
- the presence of a water table at the predetermined level nearly doubles the anchor lengths and loads necessary to achieve the same factor of safety, the phenomenon being more pronounced as cohesion decreases
- for a certain value of the safety factor, lowering of the water table greatly reduces the anchor loads, the pile embedment and the pile bending moment; however, less pronounced is the reduction of the anchor lengths
- as excavation proceeds, the length of the anchors contribute but little to the overall stability, while anchor loads and pile bending moments are greatly increased
- a higher factor of safety against deep seated failure results in an almost linear increase of the anchor lengths.

Factual data also indicated that anchor head elevation on the piles had but little influence on the overall quantities; thus, choice of the appropriate anchor levels appears to lie on personal or local experience which definitely reflect sound performance of the erected walls.

Analysis clearly shows that, if the acceptance criterion for wall geometry is the minimization of the pile and anchor quantities, the optimum selected candidates for the 'most probable' values of the 'calibrated' strength parameters are as follows:

Table 3. Optimum design schemes of the Berlinoise wall

No of anchor rows	Pile distance (m)	Service anchor load (KN)	Total anchor length (m)
1	1.00	300	18
3	2.80	420/450/480	18/16/14
4	2.00	240/240/240/240	15/14/13/12

A further attempt to evaluate the above configurations consisted in attributing current (1998) local unit prices to each component that contributes to the overall cost of the retaining structure. Rough estimates of this kind favors the use of a three-row anchored pile wall, where the piles and the anchors contribute almost equally to the overall minimized cost. In all cases the shotcrete layer counts for only 6 to 10% of the total cost.

However, limit equilibrium analyses offers no-sign of ground mouvements, the magnitude of which is the ultimale criterion in selecting wall configurations. It was then more than necessary to conduct further analyses using computational tools that can give an indication of these mouvements. On this goal, two more analytical models were used: the one-dimensional elastoplastic Winkler model and the two dimensional Finite Element model. Analysis of the configurations depicted in Table 3 with the aid of the DENEBOLA and the PLAXIS computer Codes was conducted using the following geotechnical parameters:

DENEBOLA				
Silty clay	c=20KPa	Φ=25°	$E_h=80\ \sigma_o^{0.5}$ MPa	
Piraeus marl	30 KPa	28°	$120\ \sigma_o^{0.5}$ MPa	
PLAXIS				
Silty clay	c=25KPa	Φ=36°	E=20MPa	v=0.3
Piraeus marl	80Kpa	25°	30Mpa	v=0.3

where σ_o the geostatic vertical stress and E_h the horizontal deformation modulus.

Characteristic results of the soil-structure interaction analyses are depicted in the following figures 1,2. In the last configuration (Table 3), the Winkler model produced a maximum wall crest displacement of 16mm, while the Finite Element model gave a corresponding displacement of 17mm. The latter model also indicated maximum displacements of the ground surface (points 1 to 5, fig 2) as depicted in Table 4 below:

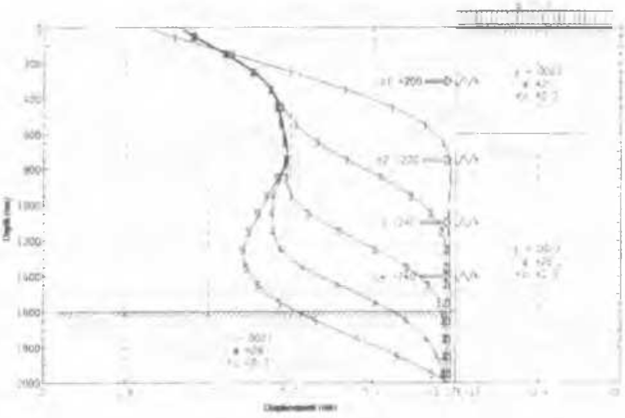

Figure 1. Evolution of wall displacements (DENEBOLA) during the excavation phases

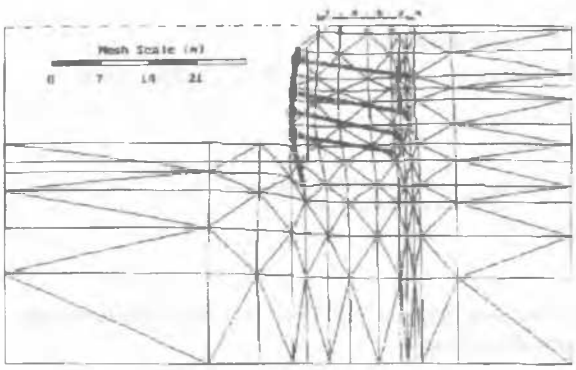

Figure 2. Deformed Finite Element Mesh at final excavation depth (PLAXIS)

Table 4. Analytical results (PLAXIS)

	Max displacement	
No Point	Vertical	Horizontal
1	6.5	17
2	7	17
3	7.5	15.5
4	7	12
5	6.5	12.5

The wall deformation mode during excavation is reproduced in a consistent manner by both models, although some dissimilarities may be discerned at the final excavation stage. On the other hand, the distribution of earth pressures on the wall changed continuously with construction processes and these phenomena were well simulated by both the Winkler and the Finite Element model. The latter lends a small deformation to the marl around the anchor zone, while the retained mass is seen to deform almost uniformely between the wall and the bonded anchor length, the deformation fading thereafter in both directions (horizontal, vertical) up to a distance from the wall crest of approximately 1.0 to 1.3 times the excavation depth; this behaviour should be attributed to the anchor prestressing which was fixed to 100% the design load. However, the beneficial effect of prestressing on ground mouvements is significantly reduced in the case of one row of anchors, where much greater deformations occurred. Earlier work of the author on a similar scheme (Costopoulos, 1988) showed that wall crest displacements from a resembling Winkler model fitted reasonably well the displacements measured on a physical model of identical geometry. Anchor load fluctuations produced by both models all along the construction procedure reflected significantly well the monitored wall-anchor interaction.

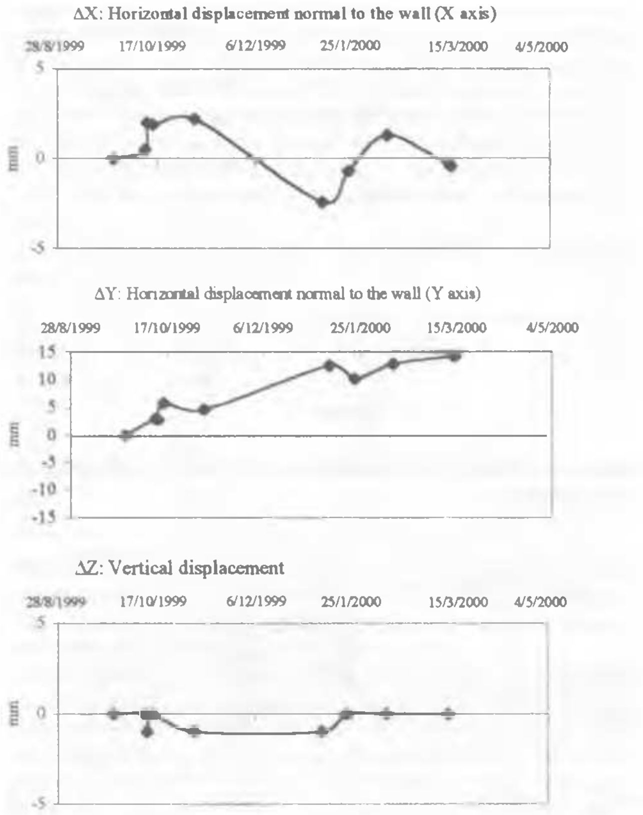

Figure 3. Evolution of crest displacements on a typical soldier pile during the excavation phases

4 MONITORING DATA

The complete movement history of the wall crest, as recorded by the instrumentation set-up, is shown in figure 3. It can be seen that in the first excavation stage (0.5m below the level of the first row of anchors), the wall crest underwent horizontal movements towards the excavation not exceeding 3mm, while practically insignificant settlements were recorded; the drilling operation and the injection procedure for the anchors had altered but little this displacement pattern. On the contrary, subsequent stressing of the tendons (4Φ0.6" S160/180) had resulted in a small inward horizontal displacement towards the retained mass and a small increase in settlements. Further excavation produced ever increasing movements, both horizontal and vertical, while after the installation and prestressing of the subsequent rows of anchors similar deformational trends could be discerned. By the end of the construction period (14/2/2000) movements appeared to have reached values of about 14mm towards the excavation, while settlements have ceased. Almost two months later (4/4/2000) wall crest horizontal displacements exhibited a value of nearly 15mm, which is equivalent to 0.01% the total wall height.

During prestressing all production anchors experienced a nearly linear-elastic response up to 120% the design load. On the other hand, load fluctuations of the control anchors during construction exhibited a likely response, as follows (fig 4) : very little variation in load at the time adjacent anchors were stressed, a sharp load increase with excavation deepening, small fluctuations during construction of the next row and a subsequent load decrease at the time the latter were stressed. Further excavation resulted again in load increase followed by a nearly constant load until the end of the measurement period. Load variations were fluctuating between 11 and 21% the initial design load; however, lower rows of anchors displayed minor differences in load not exceeding 13% the design value.

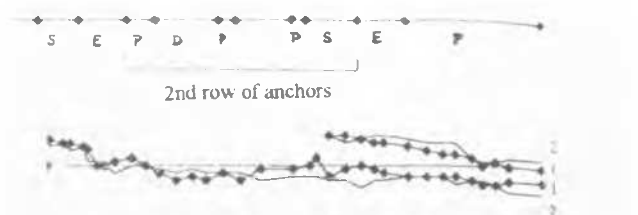

Figure 4. Evolution of prestress load on a typical contorl anchor during the excavation phases (S: stressing, E: excavation, P: pause, D: drilling, 1: 1st row, 2: 2nd row from the top downwards)

5 CONCLUSIONS

Synthetic interpretation of the computational and monitoring results assures the satisfactory performance of the 16m deep Berlinoise wall supported by four rows of anchors which were prestressed at their design values. Wall crest displacements remained lower than the limit of 0.25% the wall height, usually adopted in praxis for tie-backs, while load fluctuations were fairly low; a load carrying capacity of almost 60KN/m can then be attributed safely to injected anchors in the Piraeus marl. The numerical tools used in the analyses proved reliable in assessing the soil-structure interaction; the one-dimensional Winkler model is considered promising at the preliminary stage of design, while the more sophisticated Finite Element model can be used in the final design of the structure. The limit equilibrium method is still regarded as a useful computational procedure to conduct less costly sensitivity analyses of the wall geometry and to check again imminent failure.

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