Field Observation of Soil Displacements Resulting Due Unsupported Excavation and Its Effects on Proposed Adjacent Piles

Dr. Ala Nasir Al-Jorany
Professor
College of Engineering - University of Baghdad
alaljorany@yahoo.com

Ghusoon Sadik Al-Qaisee
Asst. Lecture
Institute of Baghdad Technology
eng_g76@ yahoo.com

ABSTRACT

Soil movement resulting due unsupported excavation nearby axially loaded piles imposes significant structural troubles on geotechnical engineers especially for piles that are not designed to account for loss of lateral confinement. In this study the field excavation works of 7.0 m deep open tunnel was continuously followed up by the authors. The work is related to the project of developing the Army canal in the east of Baghdad city in Iraq. A number of selected points around the field excavation are installed on the ground surface at different horizontal distance. The elevation and coordinates of points are recorded during **23 days** with excavation progress period. The field excavation process was numerically simulated by using the finite element package **PLAXIS 3D** foundation. The obtained analysis results regarding the displacements of the selected points are compared with the field observation for verification purpose. Moreover, finite element analysis of axially loaded piles that are presumed to be existed at the locations of the observation points is carried out to study the effect of excavation on full scale piles behaviors. The field observation monitored an upward movement and positive lateral ground movement for shallow excavation depth. Later on and as the excavation process went deeper, a downward movement and negative lateral ground movement are noticed. The analyses results are in general well agreed with the monitored values of soil displacements at the selected points. It is found also that there are obvious effects of the nearby excavation on the presumed piles in terms of displacements and bending moments.

Key words: excavation, axially loaded pile, deflection, bending moment

الملاحظات الموقعية عن الازاحات في التربة الناتجة عن الحفريات غير المسندة وتاثيرها على ركيزة مجاورة افتراضية

غصون صادق القيسي
مدرس مساعد
معهد تكنلوجيا/بغداد – الجامعة التقنية الوسطى

د. علاء ناصر الجوراني
أستاذ
كلية الهندسة- جامعة بغداد

الخلاصة

ان الازاحة الجانبية للتربة الناشئة عن الحفريات المجاورة غير المسندة ذات تاثير على ركيزة مجاورة راسية محملة ، مولدة مشاكل انشائية كبيرة في مجال الهندسة الجيوتكنيكية وخاصة في حالة الركائز غير المصممة لمقاومة تاثير الازاحة الافقية للتربة . تم متابعة اعمال الحفريات الموقعية بعمق 7.0 م اثناء تنفيذ اعمال الحفريات للنفق المقترح انشاؤه في مشروع تطوير قناة الجيش شرق مدينة بغداد في العراق. تم تثبيت عدد من النقاط المراقبة المختارة على سطح الارض حول الحفريات الموقعية للنفق بمسافات أفقية مختلفة ، سجلت المناسيب والاحداثيات للنقاط خلال 23 يوم بشكل مستمر أثناء عمليات الحفر ، سير عمليات الحفر تم تمثيله باستخدام نظرية العناصر المحددة باستخدام برنامج البلاكسس (PLAXIS) . تم مقارنة نتائج التحليل التي تم الحصول عليها فيما يتعلق بالازاحات للنقاط المحددة مع المراقبة الميدانية لغرض التحقق بالاضافة الى ذلك

أجري التحليل لركائز مقترحة مفردة محملة رأسيا تم وضعها في نفس موقع نقاط المراقبة لدراسة تاثير الحفريات على سلوكية الركائز الفعلية . المراقبة الموقعية رصدت على ان الحركة الصعودية والحركة الجانبية الايجابية (عكس اتجاه الحفر) تحدث في حالة الحفريات الضحلة العمق ، ومن جهة اخرى سجلت الحركة الهبوطية والحركة الجانبية السلبية (بأتجاه الحفر) للحفريات العميقة ، بشكل عام فان المراقبة الموقعية لحركة التربة الجانبية للنقاط المختارة متوافقة مع الجانب العددي. ايضا هناك تاثير واضح للحفريات المجاورة على الركائز الافتراضية من حيث الاراحات وعزوم الانحناء .

الكلمات الرئيسية : الحفريات ، ركيزة عمودية محملة رأسيا ، الهبوط ، عزوم الانحناء

1. INTRODUCTION

Constructing the foundation of a new structure close to existing adjacent ones is a common geotechnical problem that is often encountered in practice. Such a problem becomes more complicated when the new structure requires a deep unsupported excavation such as constructing open tunnels or deep rafts for high rise buildings. Such type of excavation may cause severe damages to the adjacent structures resulting due loss of lateral confinement of the foundation soil. The design of these excavations should include an estimation of the ground movement as well as stability check of the adjacent buildings. For example, a deep foundation pit nearby a subway in Taipei was excavated, which caused the line tunnel damages and great economic losses, **Zhang,** and **Mo, 2014**. **Fig.1** presents case study of collapsed 13-floor building in Shanghi, China, **Ahmed, 2014** that was due to adjacent deep excavation. **Fig.2** shows lateral deformation of sheet pile nearby excavation in **Baghdad, 2015**.

The maintaining structural integrity of the pile foundations require the information of these additional loads, deflections is of great importance. It is also important to study the behavior of the structures during and after failure in order to expand knowledge of engineers after the serviceability limits of the structures, **Poulos, 1997**.

Buildings adjacent to excavation may exhibit several phenomena, **Korff, and Mair, 2013**:

- Pile capacity is reduced as smaller stress levels.
- Soil settlement below the base of pile.
- The variation of skin friction (negative or positive) due to relative movements of the soil and the pile shaft.
- Rearrangement of load between the piles.
- Lateral pile deformations.

Ong, et al., 2004, examined the case study for a building erected on soil strata including soft clay subsequent by stiffer soils. Unsupported 5-m deep slope excavation is executed beside a capped 4-pile group of 0.90m diameter bored piles during the excavation of basement. The piles were provided with strain gauges and inclinometer. Unfortunately, during the course of excavation, the slope excavation failed due to heavy rainfall. **Fig.3** shows the difference of pile deflection, lateral soil movements and maximum bending moment on the pile throughout the excavation. When the lateral movements increased, it causes increasing of induced bending moment and the pile deflection and the amount of pile deflections was significantly lesser than the consistent soil movements at the same depth. The measured bending moment override the ultimate bending moment. Severely damaged of the adjacent piles were observed due to that extreme soil movements as a result from excavation and were replaced by another group. Assessment of the influence of excavation on nearby piles is necessary but full scale tests were considered time consuming, and needed additional cost to perform, therefore, centrifuge modeling technique was adopted to simulate the problem.

Poulos, 2007, exanimated the excavation for new pile cap nearby existing piles in soft to medium clay with 3.0 m and 10m depth and width of pile cap, respectively. No lateral support was available for the excavation. The examination showed the maximum bending moment was significant for the piles adjacent to the excavation. The nearby pile attempted to move upwards slightly as a result of the excavation as there was no surface pressure, while it settleed when there was surface pressure. Thus, it would be observed that the bending moment and shear in the pile were developed due to lateral movement.

The case study for commercial project was carried out on the island of **Java, Indonesia**; it included the erection of three buildings: an organization building, a hotel, and a shopping mall. The investigation details showed the soil strata were soft to very soft silt underneath by firmer silt. The driven cast-in-situ piles of 0.5 m and 20 m diameter and depth respectively. The ninety piles are casted for the organization building. An excavation was progressed nearby to the shopping center where the unbraced excavation closest to the pile group with excavation depth of 4m. Horizontal movement of the soft silty soil in the direction of the excavation was observed, and it was difficult to finish the excavation. Stabilization of the excavation was attempted by steel I-beams, while it was not feasible. Also it was specified that some of the steel I-beams lied closely to the excavation were shafted more than 1m in the direction of the excavation, thereafter the building began to incline slightly. The ultimate pile capacity was significantly exceeded the design capacity due to uncontrolled excavation , **Poulos, 2007**. Displacement of the corner piles produces a transfer of the building load to the close columns of the building and causes additional bending moment in the beam and slab. Thus, cracking of the beams and slab happened, leading to additional redistribution of building loads to closely columns and slabs, and then additional cracking. The rigidity of the structure causes the inclination of building and then an increase of bending moment is caused by the eccentricity of the building load, and the tilting is worsened. Thus, the initial weakness of the piles as a result of the soil movements that caused a gradual failure of the foundation and structure throughout duration of 2 - 3 months approximately is evaluated.

Fig.4a shows the picture of cracked pile group for case study in West Malaysia throughout the construction of pile cap. Bending moment was developed in piles and leaded to crack and destroy of piles. **PLAXIS 3D FOUNDATION** with Hardening-Soil model was adopted to simulate the behavior of these piles throughout the excavation as shown in **Fig. 4b, Kok, et al., 2009.**

In this study, an attempt is made to investigate and evaluate some of the above mentioned side effects that result due to unsupported excavation.

2. FIELD WORK

The field work during the execution of excavation works for proposed tunnel is 7.0 m deep in the project of developing the army canal. The site location is close to east of Zayona district in the east of Baghdad city in Iraq. The project of Zayona tunnel is located between the army canal at one side and parking at Omar bin Al-Katab street in other side as presented in **Fig.5**. The tunnel occupies an approximate area about (45X28) m^2.

Five observations points are located at two sides close to tunnel boundaries at horizontal distance that ranges between 1.25-3.25 m from tunnel excavation edges as present in **Fig.6.**

Fig.7. shows plates of in situ points before excavation. Many difficulties are encountered after the installation and throughout the excavation due to the site activities; vehicles and worker movements**,** in spite of the area of points are surrounded with caution tape. The point coordinates and elevations are measured using total station before and throughout the excavation**. Table 1** shows the point horizontal distance from face of excavation and coordinates before excavation. The excavation works started in **26** June 2013 and continued for 23 days to reach the final excavation depth of 7.0 m below the natural ground level. Point's coordinates are monitored throughout the excavation period. **Fig.8** presents plates of tunnel project after excavation.

3. RESULTS OF FIELD WORK

Fig.9 displays the variation of point's displacements with time until reaching the 7.0 m depth of excavation. All points are exposed to vertical upward (positive) displacement with increasing depth of excavation until reaching depth 4.0 m (at the ten the day) after that downward vertical (negative) displacements are detected. In general the vertical displacement of points whether it is positive or negative increased with decreasing the horizontal distance between points and excavation face.

Figs.10 and 11 indicate the variation of points displacement **x** and **y** with excavation time, all points vary with excavation progress. In the beginning of excavation to excavation depth 4.0 m (at the ten the day), the positive variation is observed of points displacement whether at x or y. After that depth, negative variation of point displacement is noticed (towards the excavation).

4. NUMERICAL MODELING

4.1. Numerical Modeling of Field Work

In first part of numerical analysis, a series of **3D** finite element analyses are performed using **PLAXIS 3D** foundation program to model the ground movement at location of observation points nearby the excavation. Single pile is then assumed to be installed at same location of observation points. The pile deflection and bending moment profile are examined for each pile with excavation depth. The soil profile in the project site is consisting of approximately 20 m of Silty clay low plasticity. The Silty clay layer is modeled with hardening soil model and soil properties are listed in **Table 2** depending on soil tests and investigation report of the project. The numbers of excavation stages are five stages at 1, 2, 3, 5 and 7 m respectively.**Fig.12** displays the top view of outer boundaries of tunnel and the observation points.

4.2. Results of Numerical Modeling of Field Work

Fig.13 presents the distribution of vertical ground movement for each excavation depth. The upward vertical ground surface movement (positive) is observed for all observation points; generally, the positive vertical movement of observation points increase with increasing depth of excavation until 5.0 m deep and then decrease. The upward (positive) vertical movement is range from 15 to 60 mm in central area of tunnel pit while it is range from 3 to 15 mm at the location of observation points.

Fig.14 presents the distribution of lateral ground movement for each excavation depth. Firstly, the lateral grounds surface movements reverse to excavation direction (positive) are detected when the excavation depth is less than or equal 3.0m deep and that lateral grounds surface movements are increased with increasing excavation depth until reach excavation depth is less than or equal 3.0m deep. After that depth of excavation , the lateral grounds surface movements reverse to the excavation (positive) are reduced and the negative lateral grounds movements (towards the excavation) developed below the level of ground surface.

The comparisons are made between the field measurements and numerical analysis regarding the vertical ground movement of observation points that are shown in **Fig. 15**. Good agreements are noticed in distributions and magnitudes for depth about 4-5 m (less than the 10 days); meanwhile less agreement is observed when the depth of excavation exceeded 5 m (more than the 10 days).

4.3 Numerical Modeling of Large Scale Model Pile Performance

The response of proposed full scale axially single pile was studied due to nearby excavation of tunnel. Single pile is installed separately at each location of observation points of length and diameter 13m and 0.28 m respectively of L/deq ratios equal 46. The axial working load is evaluated about 250 KN.

Fig. 16 and 17 present the variation of pile deflection and bending moment along pile length for five excavation depths 1, 2, 3, 5 and 7 m and with 1.25m, 2.25m and 3.25 m horizontal distance from face of excavation. It can be observed that for depth of excavation less than or equal 3.0 m the pile deflection and bending moment are slightly affected compared to that occur of the deeper excavation. When the excavation depth is equal or more than 5.0 m (L/2), the pile deflection is almost changed to be towards the excavation (negative).The pile deflection and bending moment values decreases with increasing the horizontal distance of excavation for all depth of excavation as the comparisons present in **Fig. 18 and 19** with respect to each depth of excavation and different horizontal distance of excavation. The maximum deflection is located at pile head that reached about 8% and 10% of pile diameter for horizontal distance of excavation 3.25m and 1.25m, respectively. The minimum deflection is located at pile tip about 1/3 maximum deflection at pile head. The pile bending moments exhibits double curvature response when the excavation depth is more than 5.0 m (L/2) for all examined horizontal distance of excavation 1.25, 2.25 and 3.25 m.

5. CONCLUSIONS

1. Insignificant effect of excavation on pile head deflection and bending moment as the excavation depth less than half pile length.
2. Noticeable effect of excavation as the excavation depth is equal or more than 5.0 m (L/2) ,the pile deflection is almost changed to be towards the excavation (negative).
3. The pile deflection and bending moment values decreases with increasing the horizontal distance of excavation for all depth of excavation.
4. The pile bending moments are exhibited double curvature response when the excavation depth is more than half pile length (L/2) for all examined horizontal distance of excavation.

REFRENCES

- Ahmed S.A., 2014, *State-of-the-Art Report: Deformations Associated with Deep Excavation and Their Effects on Nearby Structures*, Ain Shams University, Faculty of Engineering, Structural Engineering Dept. Geotechnical Engineering Group DOI: 10.13140/RG.2.1.3966.9284.

- Kok, S. T., Haut, B.K., Noorzaei, j. and Jaafar, M.S., 2009, *Modeling of Passive Piles*,. EJGE, Vo. 16 ,Bund P.
- Korff, M. and Mair, R.J. , 2013, *Response of Piled Buildings to Deep Excavations in Soft Soils*, Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris 2013, 2035-2039.
- Ong, D.E.L. Leung, C.F. and Chow, Y.K., 2004, *Pile Behavior Behind a Failed Excavation*, International Conference on Structural and Foundation Failures, August 2-4, 2004, Singapore.
- Poulos, H. G. and Chen, L. T., 1997, *Pile Response Due to Excavation-Induced Lateral Soil Movement*, ASCE, Journal of Geotechnical and Geoenvironmental Engineering, Vol.123 No.2, pp.94-99.
- Poulos, H. G., 2007, *The Influence of Construction "Side Effects" On Existing Pile Foundations*, Lane Cove West, NSW, Australia, and Emeritus.
- Poulos, H. G., 2007, *Ground Movement- A Hidden Source of Loading on Deep Foundations*, DFI Journal Vol. No.1, 37:54.
- *Plaxis 3D foundation*; Material models manual V1.6 (2006).
- Zhang, A and Mo, H., (2014) , *Analytical Solution for Pile Response Due to Excavation-induced Lateral Soil Movement*, Journal of Information & Computational Science, 1111–1120.

Figure 1. Failure of a building in China in 2009 that was initiated by a nearby deep excavation, **Ahmed, 2014**.

Figure 2. Lateral movement of sheet pile due to nearby deep excavation, Baghdad ,2015.

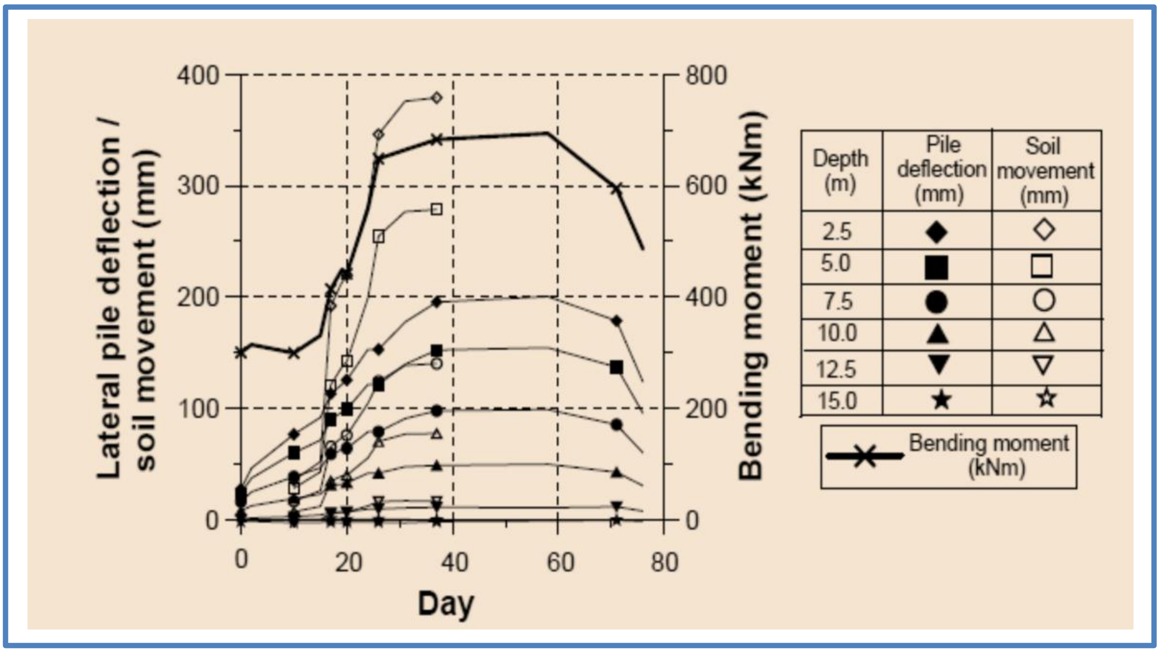

Figure 3. The induced bending moment of pile with depth for sandy soil , Chow et al., 2004.

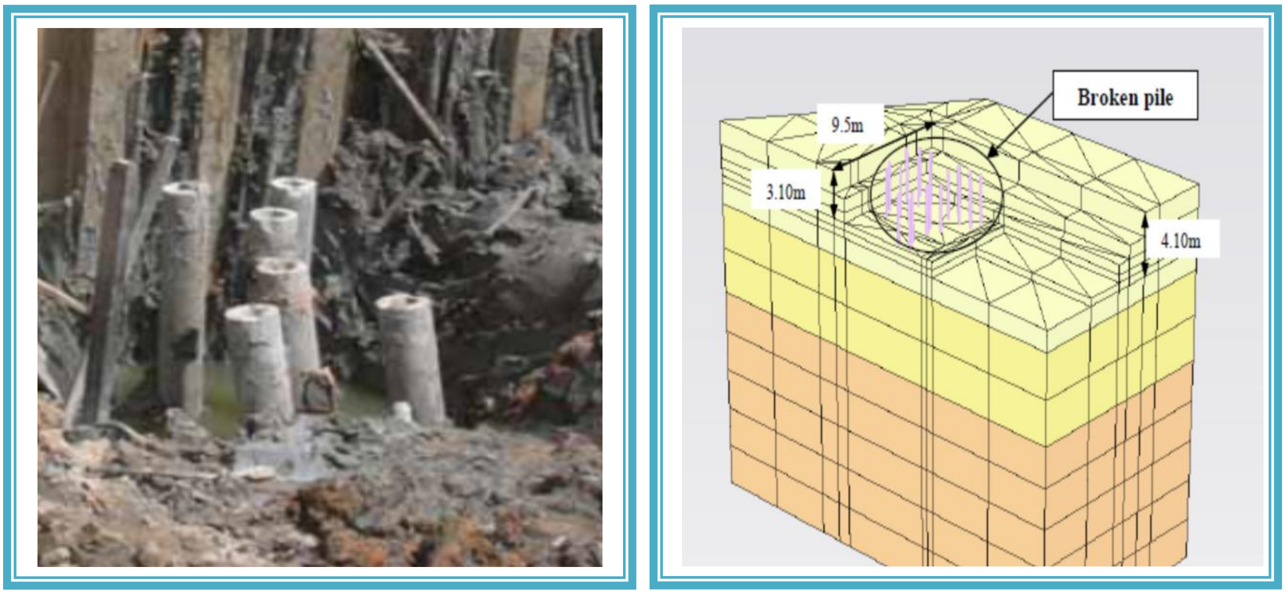

Figure 4. (a) Picture showing a 3-pile group of broken piles: (b) Excavation profile for final phase of staged construction ,**Kok et al., 2009**.

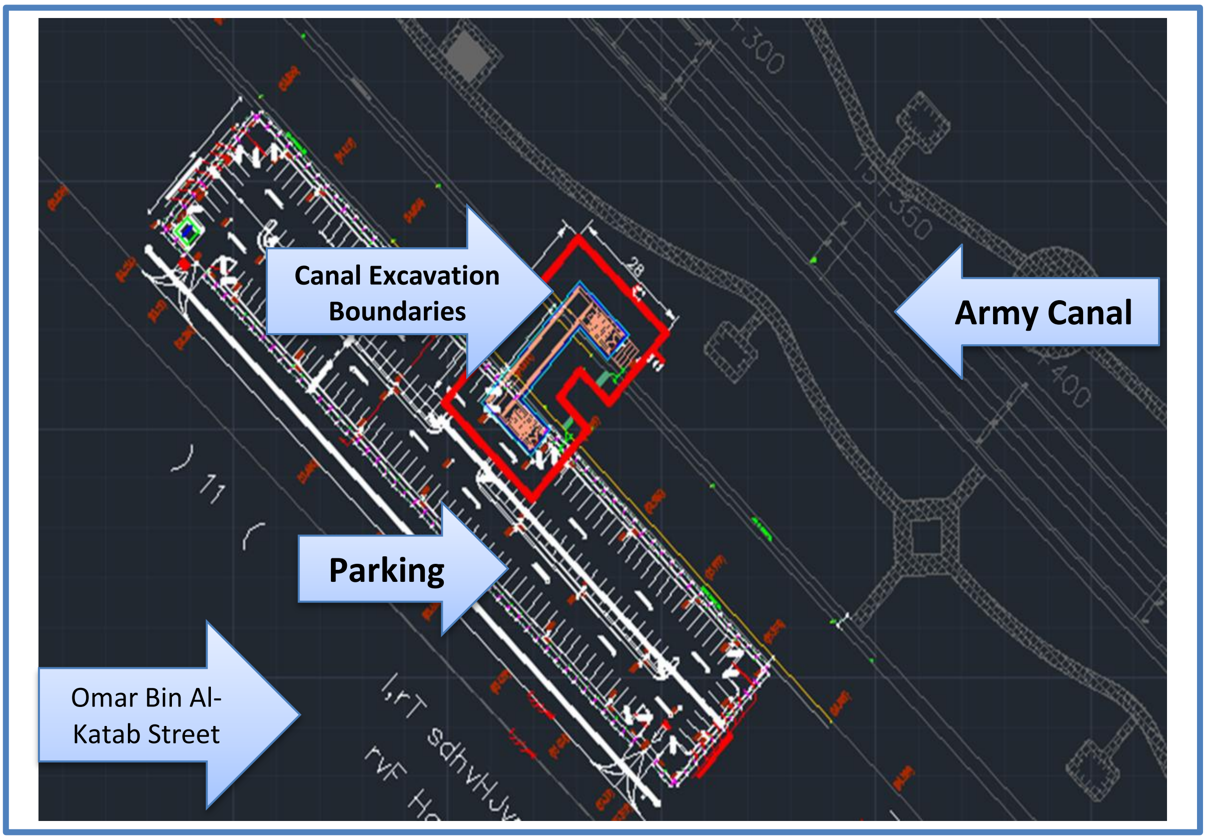

Figure 5. The project of Zayona tunnel at army canal and surrounding activities .

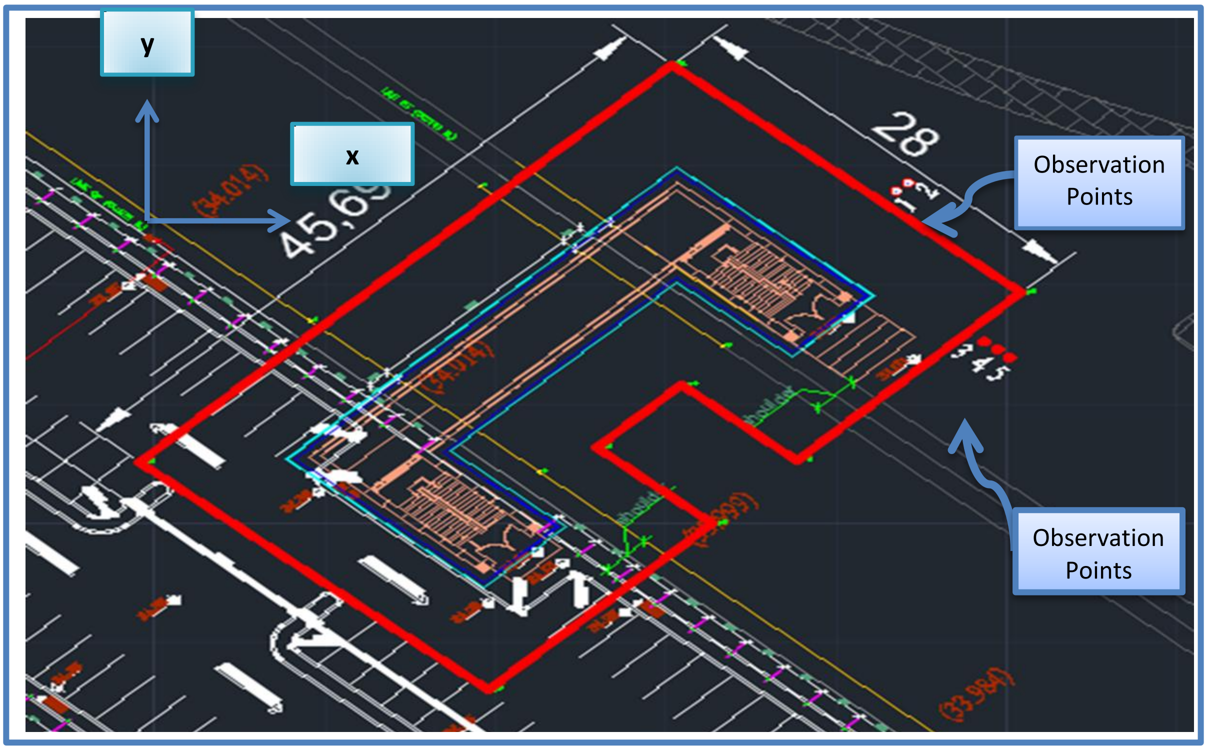

Figure 6. The project of Zayona tunnel at army canal and observation points.

Figure 7. Photos of points installation before excavation of Zayona tunnel at army canal.

Figure 8. Photos of excavation work of Zayona tunnel at army canal project.

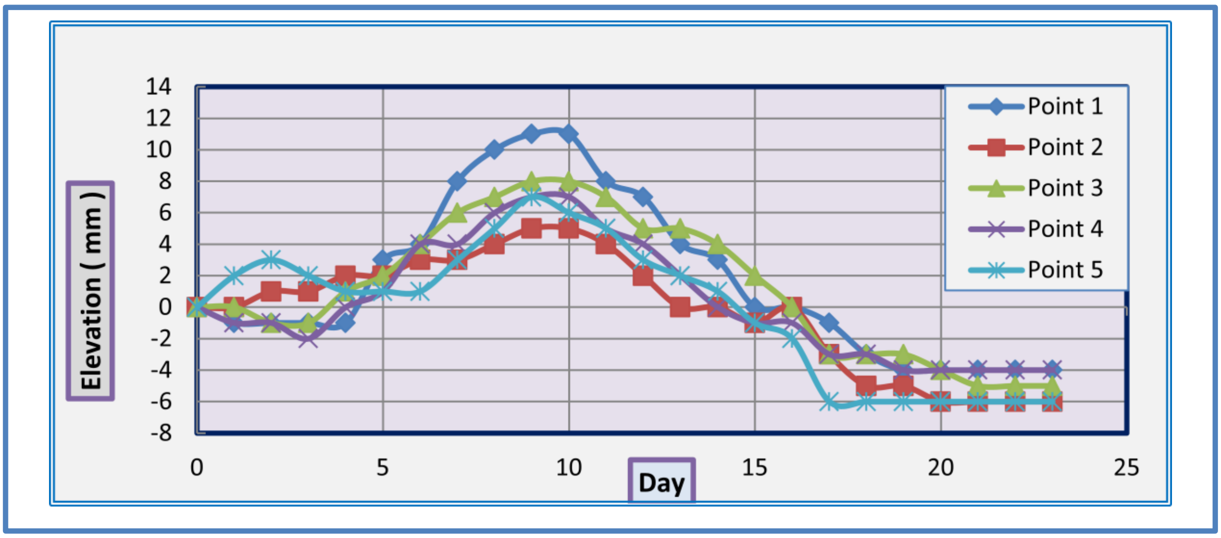

Figure 9. Vertical displacement of observation points.

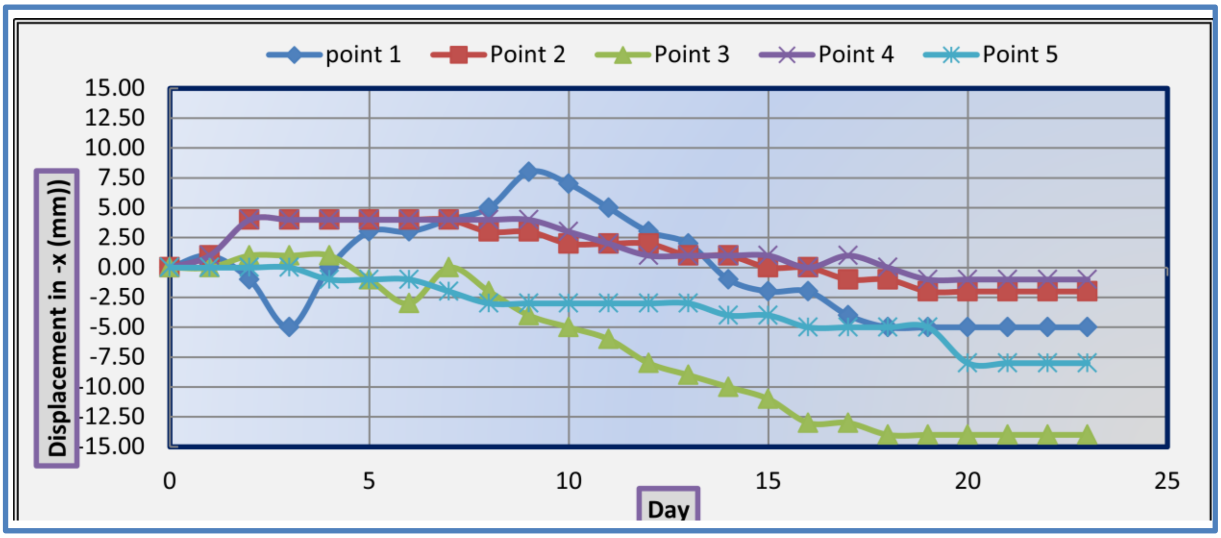

Figure 10. X- coordinates variation of observation points.

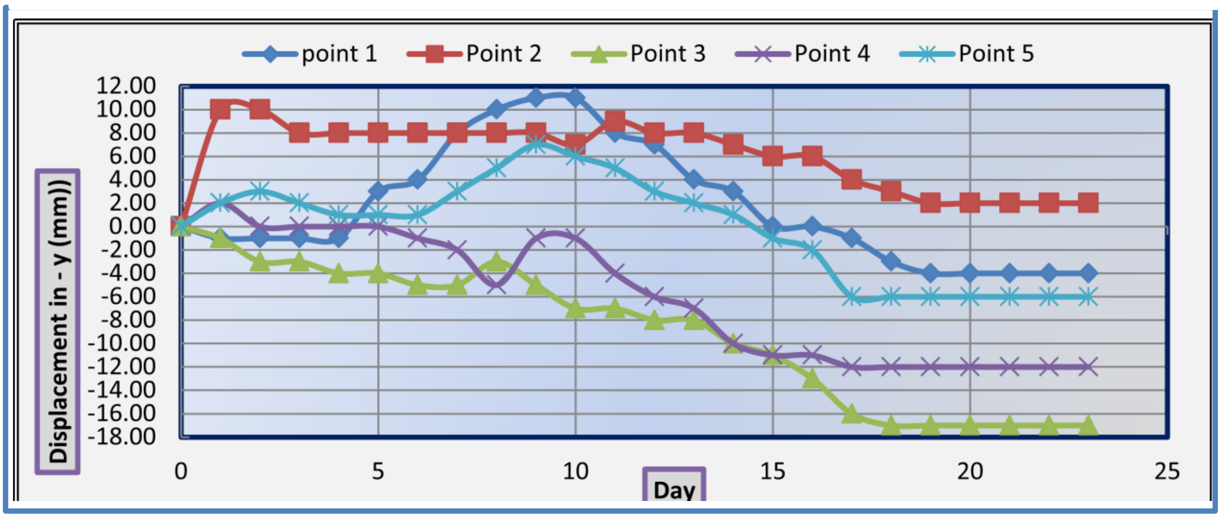

Figure 11. Y- coordinates variation of observation points.

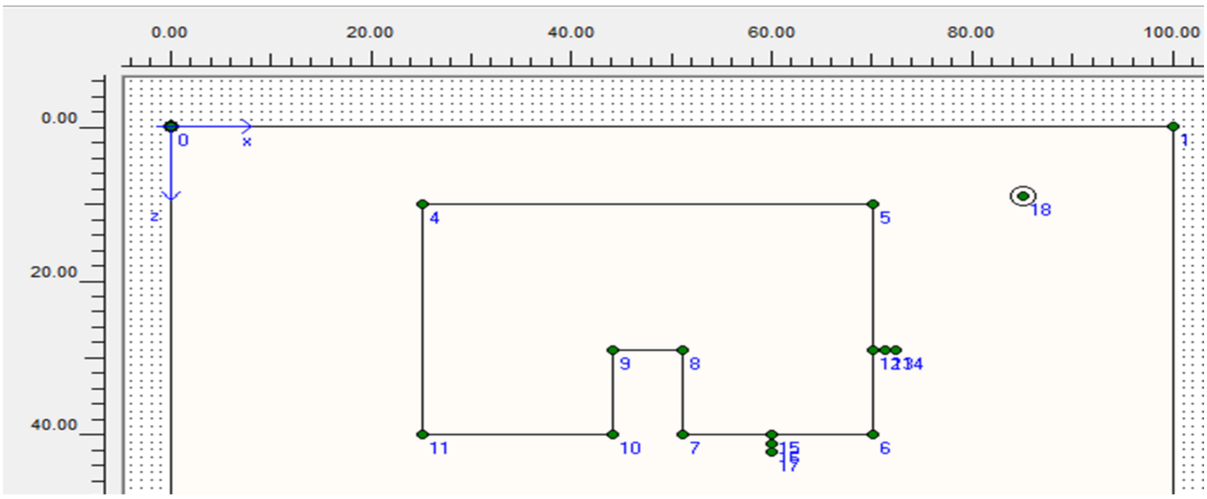

Figure 12. Top view of tunnel boundaries and on of observation points.

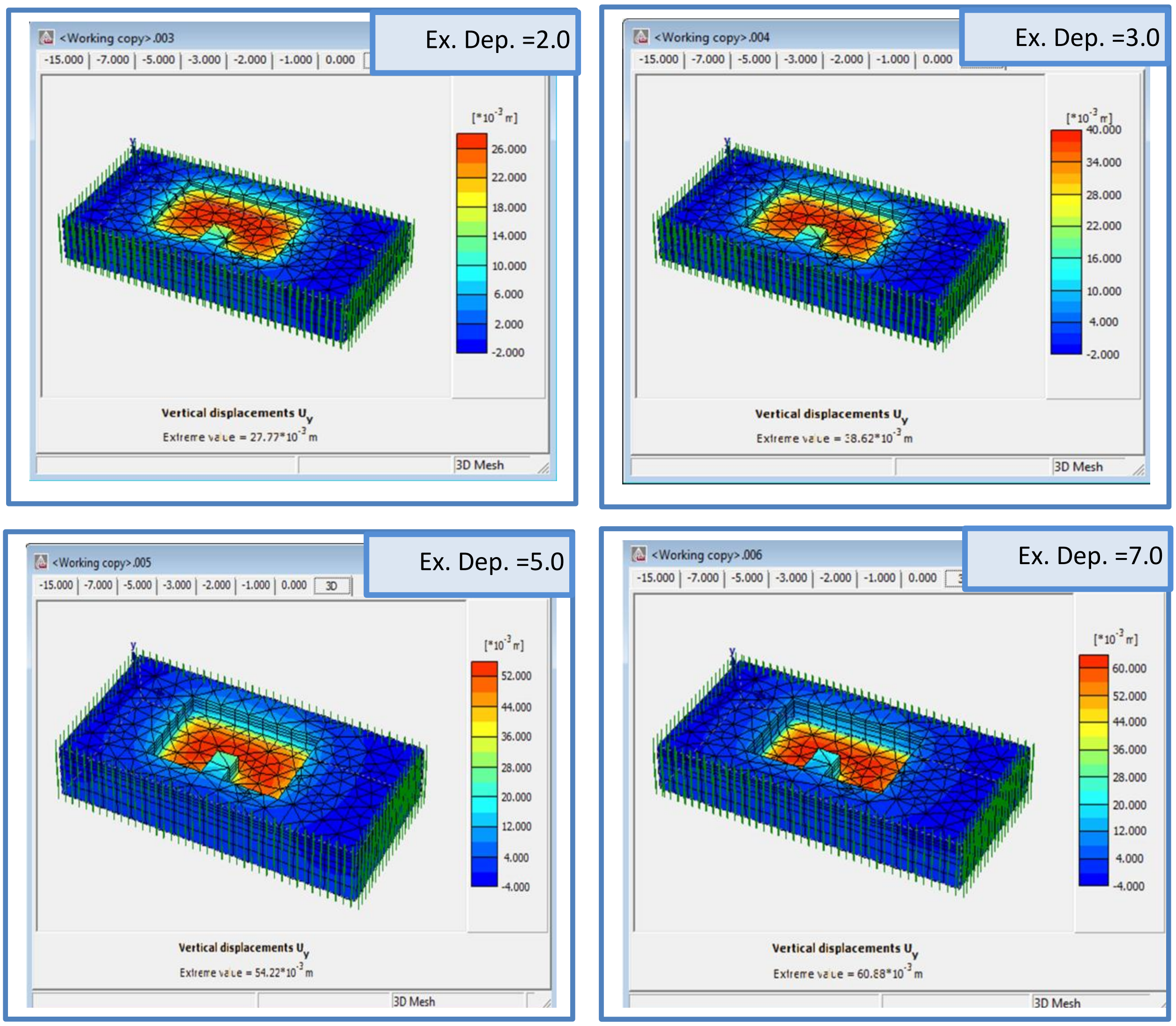

Figure 13. Vertical ground movement distribution with excavation depth.

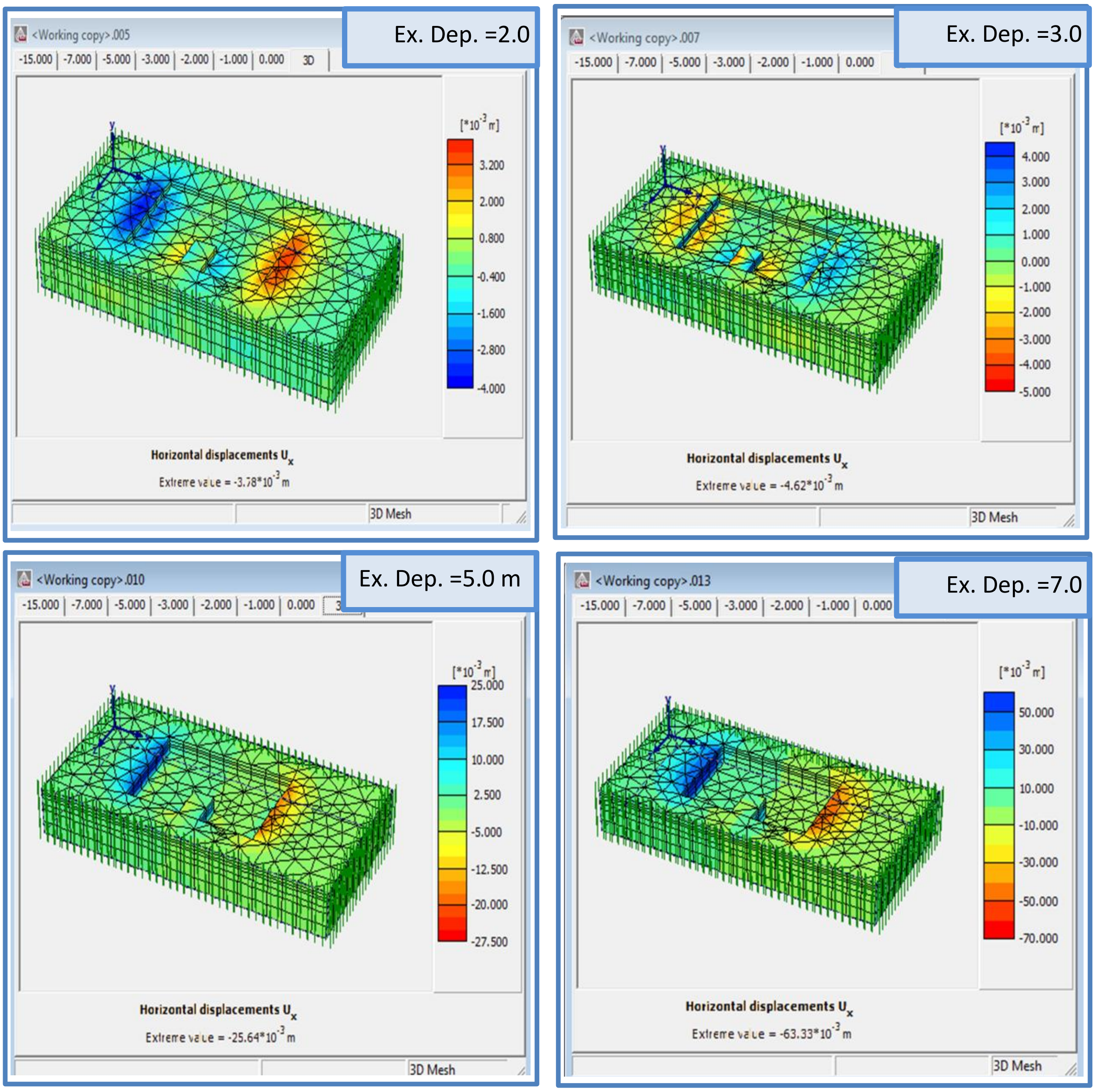

Figure 14. Lateral ground movement distribution with excavation depth

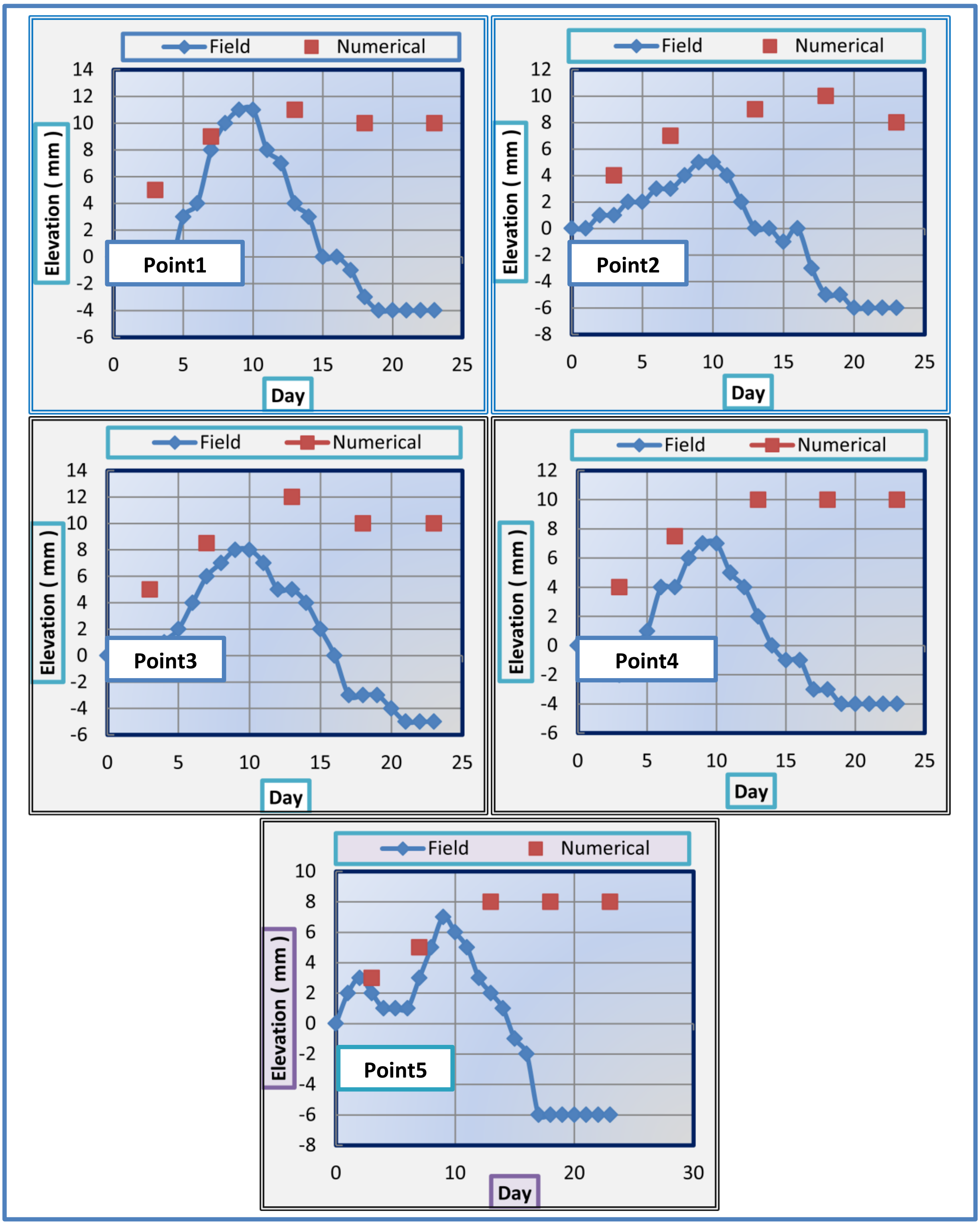

Figure 15. The comparisons of the field and numerical vertical ground movement of observation points.

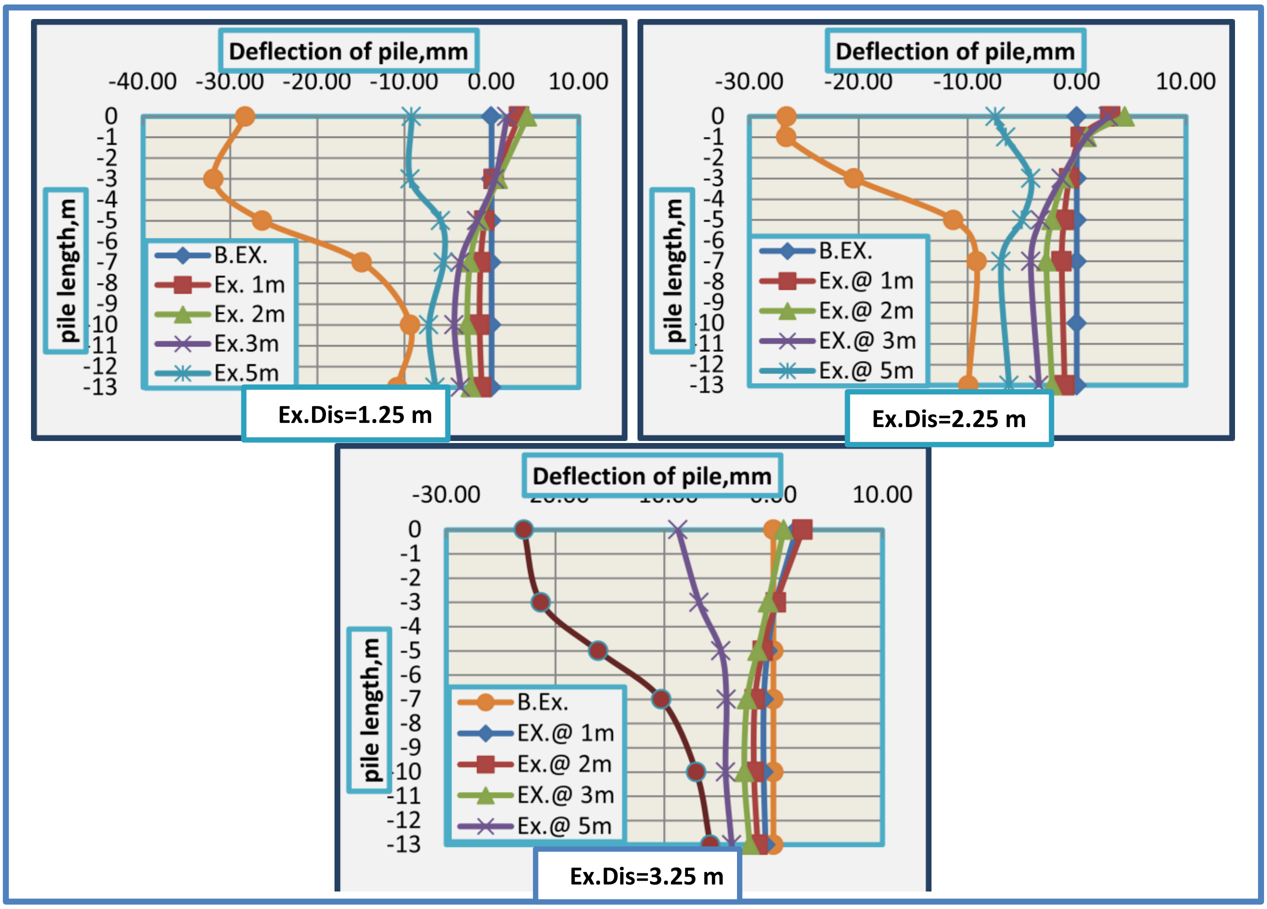

Figure 16. The variation of pile deflection with excavation depth and for each horizontal distance of excavation.

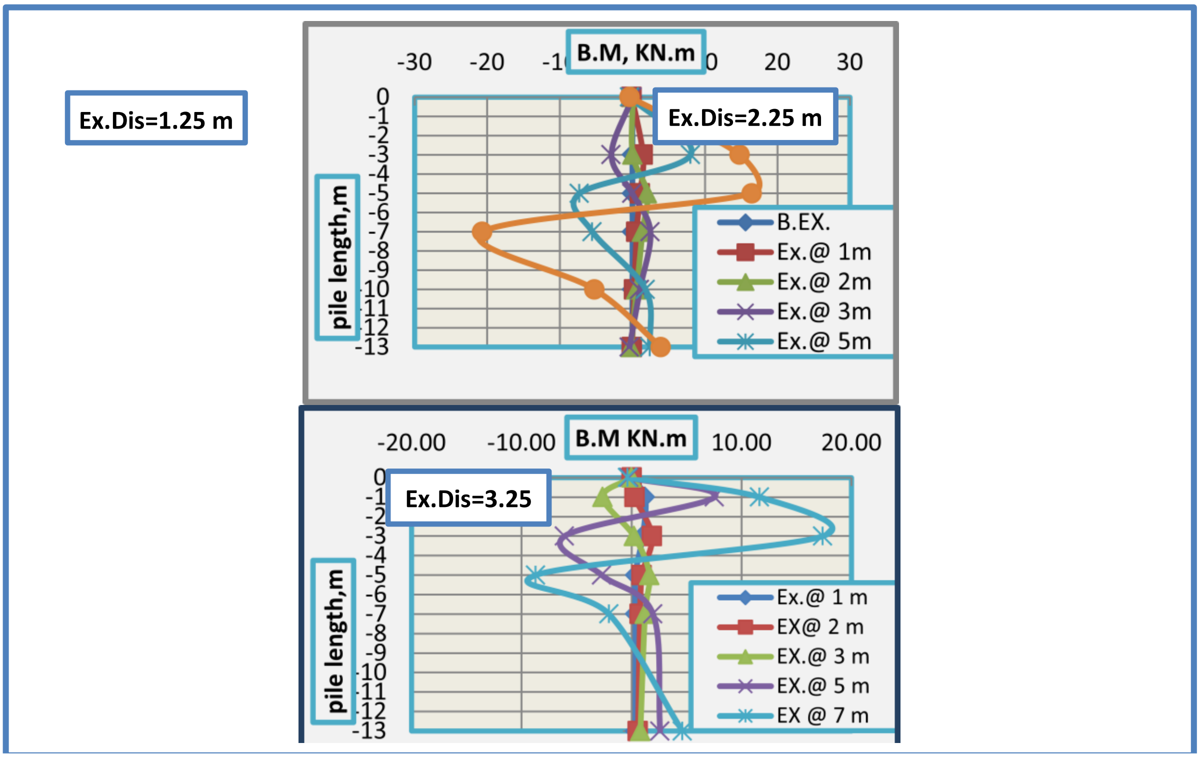

Figure17. The variation of pile bending moment variation with depth and for each horizontal distance of excavation.

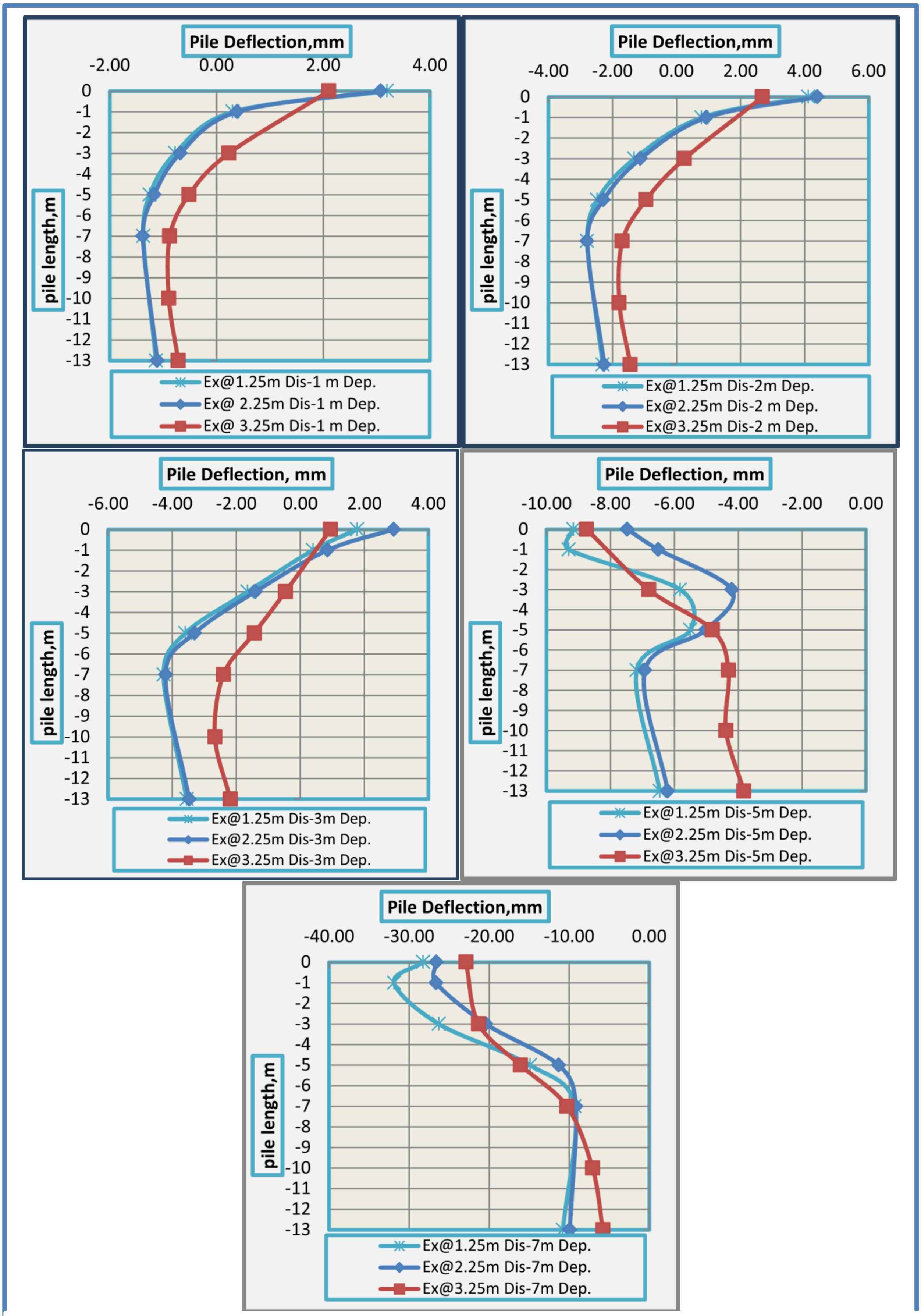

Figure 18. Comparisons of piles deflection with excavation depth and horizontal distance of excavation.

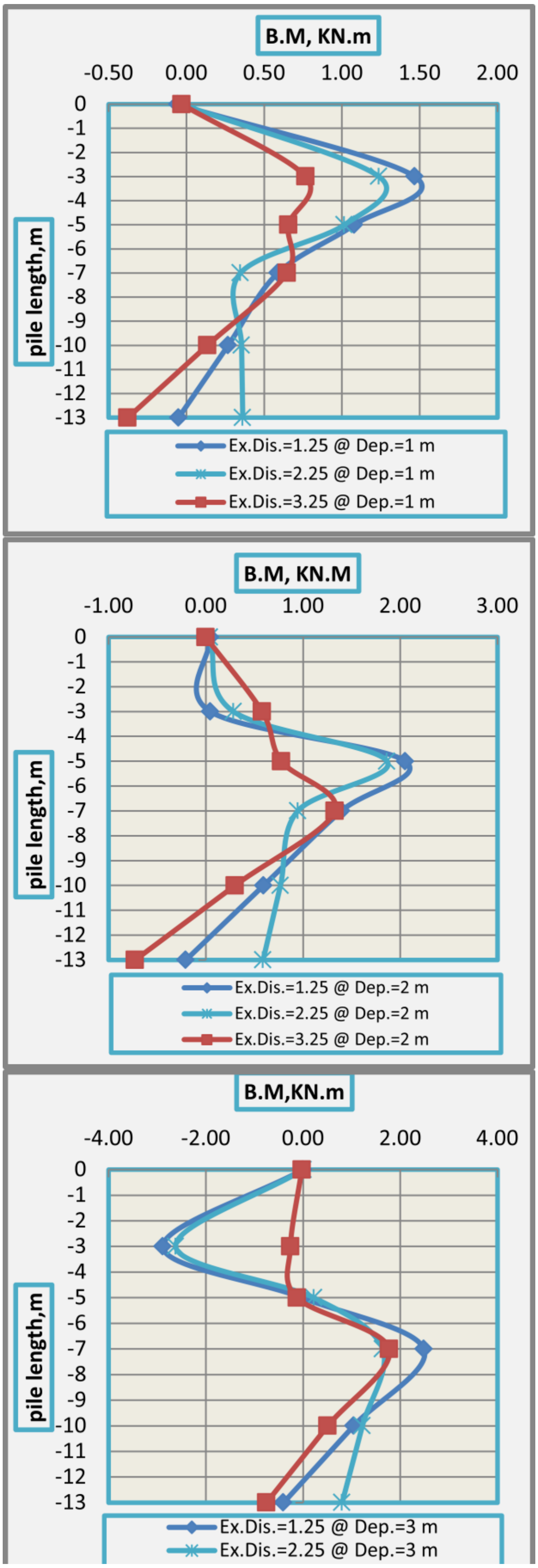

Figure 19. Comparisons of piles bending moment with excavation depth and horizontal distance of excavation.

Table 1. Location and global coordinates before excavation of observation points.

Point	Perpendicular distance from excavation face, m	x	y	Z
1	1.25	450493.517	3688077.891	33.363
2	2.25	450494.225	3688078.589	33.339
3	1.25	450498.727	3688065.220	33.500
4	2.25	450499.461	3688064.558	33.504
5	3.25	450500.164	3688063.851	33.508

Table 2. Material properties of in situ soil.

Parameter	Name	Value	Unit
Material model	Model	Hardening soil model	-
Type of material behavior	Type	Drained	-
Unit weight of soil	γ_{unsat}	18.0	KN/m^3
Young's Modules	E^{ref}_{50}	6500	KN/m^2
	E^{ref}_{oed}	6500	KN/m^2
	E^{ref}_{ur}	15000	KN/m^2
Poisson's ratio	ν	0.30	-
Cohesion	C_{ref}	75.0	KN/m^2
Friction angle	ϕ	15.0	°