GEOLOGIJA. 2007. No. 60. P. 76–82

Engineering geology • Inžinerinė geologija • Инженерная геология

Deformations of till soils while installing vibro-piles

Tomas Kairys,

Gintaras Žaržojus,

Kastytis Dundulis

Kairys T., Žaržojus G., Dundulis K. Deformations of till soils while installing vibro-piles. *Geologija*. Vilnius. 2007. No. 60. P. 76–82. ISSN 1392-110X

In the world there are various physical models of soil deformation while installing vibro-piles, which have become standard; however, this aggravates the substantiation of the pile load-bearing capacity calculation methodology under engineering–geological conditions of Lithuania. In 2005, direct tests of the change in moraine soil deformations and its physical properties while driving in vibro-piles in Klaipėda and Vilnius were performed in Lithuania. The tests showed that neither of the standard soil deformation models was fully suitable for the conditions of Lithuania, and these researches helped to establish it and to improve the calculation methodology of the pile load-bearing capacity.

Key words: vibro-pile, till, deformation zone, model

Received 03 July 2007, accepted 15 September 2007

Tomas Kairys, Gintaras Žaržojus, Kastytis Dundulis. Department of Hydrogeology and Engineering Geology, Vilnius University M. K. Čiurlionio 21/27, LT-2009 Vilnius, Lithuania. E-mail: Kastytis.dundulis@gf.vu.lt

INTRODUCTION

The first vibro-pile studies were performed in 1930 in Germany and in 1931 in Russia (Viking, 2002). The research works of the Russian soil dynamics researcher Pavyluk has shown that with the help of vibration, soil resistance can be reduced during pile driving; vibro-hammers were used even more widely.

The effect of vibration is different for various soils. Scientists from various countries have made various physical models of soil deformation distribution around a vibro-pile, which presently are standard and the calculation methodology of the pile load-bearing capacity is based on them. A. Caquot (1934), A. S. K. Buisman (1935), K. Terzaghi (1943), G. G. Meyerhof (1951, 1953), V. G. Berezantzev (1961), A. W. Skempton, A. Yassin and R. E. Gibson (1953), A. Vesic (1975, 1977) and others participated in the drafting of these models and also in their mathematical reasoning.

The scientists have performed the researches in soils of various geneses – even in the technogenical one, but the moraine soil, which is the base of the foundations in the greater territory of Lithuania, has not been analyzed. It is necessary, using direct methods, to explore the deformations of the till soil and the impact areas that form within it while driving in a vibro-pile and are conditioned by a frequent usage of this type of piles in making the foundations. It is also important to ascertain the validity of the pile load-bearing capacity calculation schemes.

A REVIEW OF PILE AND SOIL INTERACTION MODELS

Traditionally, marginal load-bearing capacity of a pile in sandy soils is the multiplication of the initial strains (σ'_{vo}), which appear due to the load in the tip of the pile, and the pile load-bearing capacity coefficient (N_q). The values of the pile load-bearing capacity

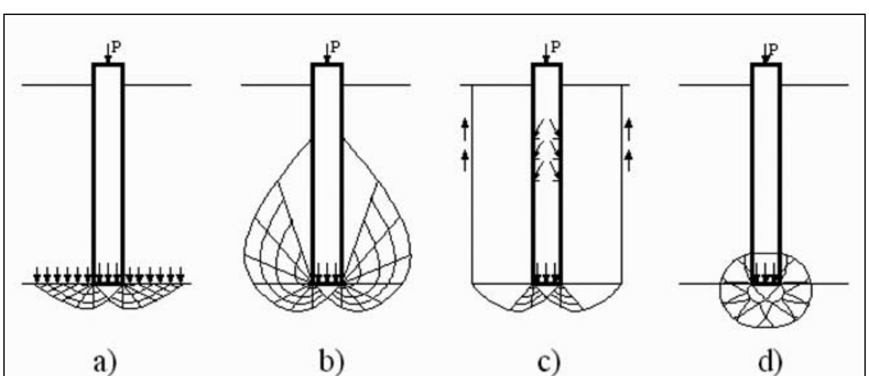

Fig. 1. Physical models of soil and pile interaction in sandy soil
1 pav. Smėlinio grunto ir įgilinamo polio tarpusavio sąveikos fiziniai modeliai

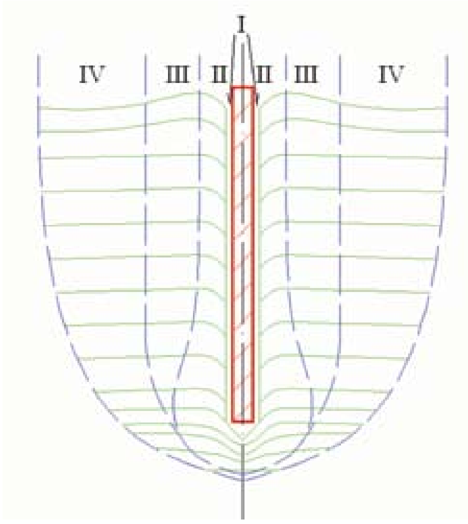

Fig. 2. Areas of soil deformation around the pile
2 pav. Grunto deformacijų zonos aplink įgilintą polį

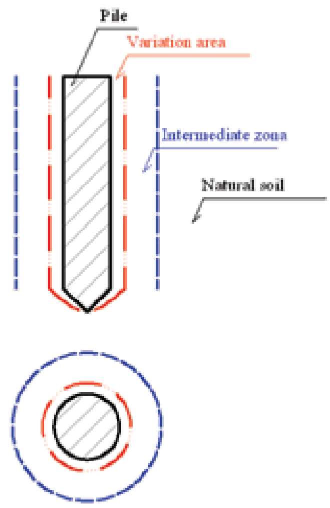

Fig. 3. Zones around the pile
3 pav. Zonos aplink polį

coefficient (N_q) depend on the ratio of pile length (l, m) and diameter (*D, m*). In literature, different values of the N_q coefficient are presented. They depend on the physical model of the chosen soil solid disturbance during the driving in of a pile.

R. Lancellotta in his book "Geotechnical engineering" (1995) presents four physical models of sand behaviour when a pile is driven in (Fig. 1).

Type (a) – suggested by Caquot (1934), Buisman (1935), Terzaghi (1943). In this model, a widely used solution for shallow foundations when the soil extrusion from under the tip of the pile is proportional to the transmissible vertical pressure irrespective of the soil and pile interaction.

Type (b) – Meyerhof (1951, 1953) suggests another soil extrusion model which covers a much larger base deformation area, but the use of this solution is not consistent with the assumption of the non-compressible medium.

Type (c) – Berezancev (1961) suggested a model in which the vertical effective pressure at the end of the pile is calculated considering the base arching phenomenon which is perpendicular.

Type (d) – suggested by Skempton, Yassin and Gibson (1953) and Vesičius (1975, 1977). Disintegrations under the tip of the pile must be evaluated similarly as during the penetration into elastic plastic medium.

The presented hypotheses of the soil behavior under the tip of the pile highly effect the values of the pile load-bearing capacity coefficient (N_q), which vary from 55 up to 500, when $\varphi' = 35°$.

In Russian literature, there is a model presented, in which four different areas are distinguished (Fig. 2) (Далматов, 2002).

Area I. A very stiff shell of 2.0–10.0 mm thick forms around the pile.

Area II. An area of compacted soil. The thickness of this zone varies from 0.7 up to the diameters of three piles (D).

Area III. This is a ring of soil which makes up to 5–6 pile diameters. The composition of the soil remains similar to the natural one. In this area the layers of the soil have an upward inflection.

Area IV. The diameter of this area is from 5 up to 12 m (16–21 D). In IV area, a moderate (partial) change in the initial attributes is noticeable. The radius of this area increases together with the driving depth of the pile. Under the tip, the medium of the thick soil is formed and the thickness in its center is close to 2 D.

In clayey soils, the effect of the driven pile upon the environment is different. According to C. S. Chen, S. S. Liew and Y. C. Tan (2005), in clayey soils, around the driven pile there are two zones (Fig. 3.) During the pile driving, a soil shifting appears in that direction, where the soil resistance is the least.

A variation area is a ring around the side of the pile, the thickness of which is 10.0–15.0 cm (Flaate, 1972), and which has formed during the pile driving. In an intermediate area the characteristics of the soil change less than in the variation area. The width of this area / zone depends on the characteristics of the soil, pile driving method and pile parameters. Outside the intermediate area the soil preserves its initial characteristics.

In Russian literature, there is an opposite conception about the behavior of clayey soils near the shaft and the tip of the pile.

In clayey soils, the vibro-driving forms a soft base, the thickness of which is about 5.0–10.0 cm. This base sharply (in a number of cases up to 40%) decreases the load-bearing capacity of the pile base, which during the construction works never re-establishes. For its increase, the pile, after the vibration, is hammered additionally. Such thickening of the base can increase the load-bearing capacity of the pile even up to 60–70%. The side friction of the pile is not high.

In Egypt, a test using the pile model was performed (Gamal El-Din) (Цытович, 1966). The aim of the test was to establish the constituent parts of the marginal load-bearing capacity of the pile base and to evaluate the influence of the clay consistence upon pile driving.

During the test, the pile was fixated stationary alongside the devices, and a sample of the soil was put on the moving platform, which was rising upwards at a steady pace. The devices registered the soil resistance against pile sticking. The tests were performed in soft and firm clays.

While performing the tests, it was noticed that when the pile was driven into the soil, the soil was rising around its shaft (Fig. 4). The highest points of the rise (a) were located as a ring around the pile shaft, the centre of which matched the centre of the pile. It was established that the volume of the clay getting into the area of this ring (between the maximum rising points) is approximately equal to the volume of the driven pile.

It was also noticed that due to the fairly high pile driving rate (6.35 mm/min), no consolidation of the clay could occur. The clay was pushed and extruded to the sideways from underneath the tip of the pile, and after that, it made a shell of a particular thickness and was raised upwards. Therefore, in the upper part, the rise of the soil occurred.

METHODOLOGY OF TESTING

During 2005, five tests were performed in Lithuania. The aim of these tests was the following: using direct methods, to establish the shift in the spaces of the soil which were under the tip and around it, to determine the changes in its physical properties and the regularities of the varied areas in the space. Another aim of the tests was, on the grounds of the received data, to make a physical model of the soil behavior when the pile is driven in. The tests were performed in two objects: in Klaipėda (the construction site of a factory "Orion global pet") and in Vilnius (the territory of ISC "Geostatyba" manufacturing base) (Fig. 5).

The engineering–geological structure of the factory "Orion global pet". This territory comprises the Nemunas formation of the Upper Pleistocene, Baltija subformation marginal till formations (gtIIIbl) – sandy clay, sandy–silty clay till. In the test points up to the reached depth (~2.0 m) there is stiff clay till with lenses of gravel and sand.

The engineering–geological structure of the territory of ISC "Geostatyba" manufacturing base. This base is in the district of Vilnius, Riešė subdistrict. In the test site, down to the reached depth, the Nemunas formation of the Upper Pleistocene, Grūdos subformation marginal till formations (*gtIIIgr*) are prevailing. These formations are composed of very stiff clay till with gravel and cobbles.

During the experiment, a pile of 1.5 m in length and 0.45 m in diameter was settled. Alongside the pile there was a system of markers intended for monitoring the deformations and the shift in the space of the soil, which is in the vicinity of the pile. When the pile is excavated, the change in the markers position is measured, the measurements are taken from the axes of the pile and the soil samples are taken for the laboratory analysis, the geological cross-section is also described (Fig. 6). The moisture content of the soil (W, %), natural density (ρ, Mg/m^3), density of particles (ρ_s, Mg/m^3), limits of plasticity (W_P) and liquidity (W_L) dampness were established in the laboratory (Table).

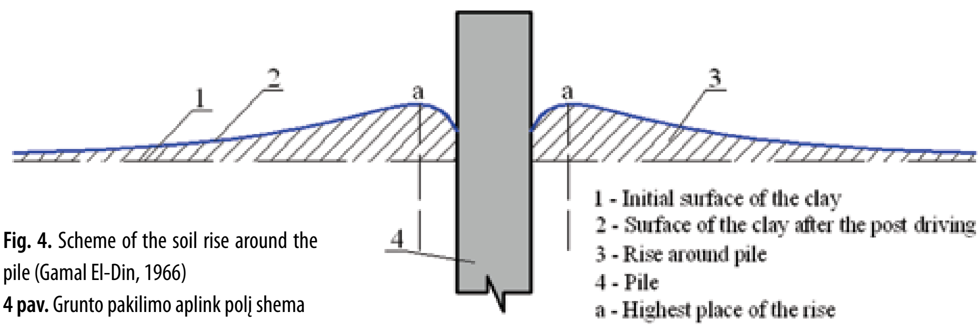

Fig. 4. Scheme of the soil rise around the pile (Gamal El-Din, 1966)
4 pav. Grunto pakilimo aplink polį shema

Fig. 5. The tests were performed in Klaipėda and in Vilnius
5 pav. Bandymai atlikti Klaipėdoje ir Vilniuje

Table. **Geotechnical parameters of undisturbed soils properties**
Lentelė. **Nesuardyto grunto fizinės mechaninės savybės**

Location	W, %	ρ, Mg/m^3	ρ_s, Mg/m^3	W_p,%	W_L, %
KLAIPĖDA	14.0	2.10	2.65	16.91	22.87
VILNIUS	9.34	2.10	2.67	13.67	18.05

Fig. 6. The excavation intended for performing the measurement and soil sampling
6 pav. Kasinys matavimams ir bandiniams

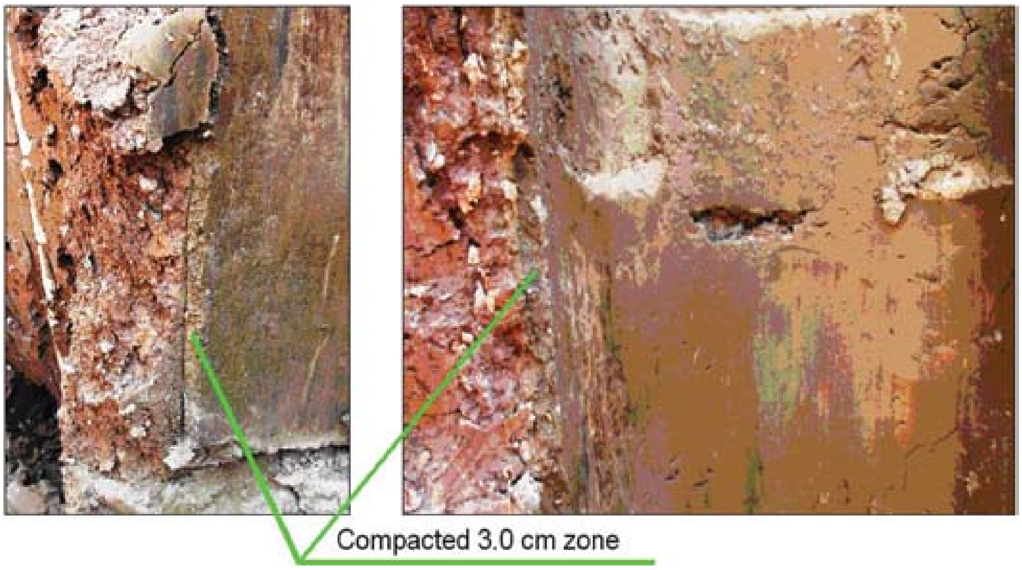

Fig. 7. Zones of compacting around the pile
7 pav. Sutankėjimo zona aplink polį

RESULTS OF THE TESTS AND SELECTION OF THE PHYSICAL MODEL

The models that have been made before are various and they describe the behavior of various soil groups when the piles are driven in using various methods. The tests performed in Lithuania by driving the pile into the till soil using a vibro-hammer showed only partial suitability of these models under our conditions. According to the test results, for the creation of the model we should use three already created models used worldwide, also we should compare them with the results obtained in Lithuania:

1. Chen C. S., Liew S. S. and Tan Y. C. (2005) model – two zones / areas form around the driven in pile: I – variation, II – intermediate zone (Fig. 3). In zone I, the soil is overly thickened, the shell of the thick soil forms around the pile;
2. "The Russian model" – a zone of weak soil forms around the pile, this zone decreases the load-bearing capacity of the pile base;
3. Gamal El-Din (1966) model – the shell of soil forms around the pile – its volume is equal to the volume of the pile. This shell, during the pile driving in moves upwards and causes the soil swelling in the upper part. The highest points of swelling are the external margins of this shell.

The first model (Chen et al., 2005) is not completely applicable, because the laboratory analyses haven not shown the thickening zone of the soil. During the testing it was noticed, that a very thick shell (around 3.0 cm) was formed around the pile (Fig. 7), also the soil, which was forming a cone (up to 5.0–7.0 cm) under the tip of the pile was very thick. This shell can be zone I of the variations, which is mentioned in the model, although the authors say that this zone is up to 10.0–15.0 cm. According to the data of the laboratory analysis, the soil density in the zone of 10.0–15.0 cm thick is up to 10% higher than the background. The difference between the intermediate zone and the natural soil could not be established by the laboratory analysis.

The Russian authors, having used the presented model, state that the weak layer (up to 5.0–10.0 cm thick) forms around the pile and under its tip after its driving into the clayey soils. The performed tests showed the opposite result: under the tip of the pile, in the moraine, a layer of the thickened soil (5.0–7.0 cm thick) formed. In sandy soil, according to the authors, the area of the thickened soil having disturbed the structure forms, its thickness is 3.0 D around the pile. The formation of the similar zone is observed in the Lithuanian till also, but its thickness is only 0.2–0.3 D (Fig. 8). In this zone, according to the data of the laboratory analysis, the density of the soil is 10% higher. Therefore, the model of the sandy soil is closer. And even more, the upright shifting of the soil is noticed there, what is observed in the till also.

Fig. 8. The disturbed zone of the soil
8 pav. Suardyto grunto zona

When analyzing the results of the test performed by Gamal El-Din and by using the pile model, we can find a lot of similarities with the tests performed in Lithuania. It must be noted, that during the test which was performed by Gamal El-Din, the pile

Fig. 9. Shift of the markers after pile driving
9 pav. Žymeklių polinkis po polio įgilinimo

Fig. 10. The cone under the tip of the pile when the pile was driven in
10 pav. Po įgilinimo susiformavęs kūgis po polio padu

Fig. 11. Uneven soil extrusion from under the tip of the pile in respect of the depth
11 pav. Nevienodas grunto išstūmimas iš po polio pado gilėjant

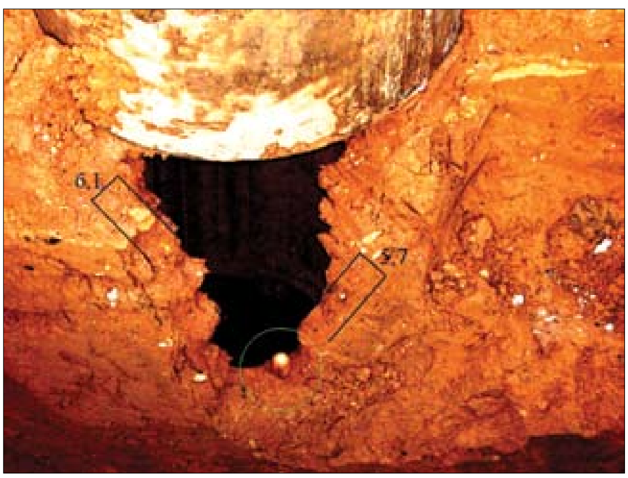

Fig. 12. Markers extruded from under the center of the pile when it was driven in
12 pav. Žymeklių išstūmimas iš polio centro įgilinimo metu

was driven into the soil using the static load. In Lithuania the piles are driven using a vibro-hammer – the dynamic load.

The Egyptian states that around the driven pile the media of the disturbed soil form. The volume of this medium is equal to the volume of the soil which is extruded from under the tip of the pile. The highest point of the extruded soil matches the exterior limit of the clayey soil medium which has a disturbed structure (Fig. 4).

When the five tests were performed, it could be noticed, that the behavior of the soil is close to these conditions. The medium having a disturbed structure makes a ring of 7.0–15.0 cm thick. The surface shape of the upper part when the pile is driven in is similar to that presented in the Gamal El-Din model (Fig. 4). A visual inspection of the cross-section, the position of markers after the pile driving, an increase in the soil density, indicates the disturbance of the medium.

The markers show the shifting of the soil by moving horizontally from their axis into one side or another. There is also the vertical shift of the markers (Fig. 9).

When the pile is driven in, the zones of the disturbed soil forms under its tip, and the cone of the dense soil having the angle of 90 degrees forms when these zones interconnect (Fig. 10).

When the pile is driven in, the soil under the tip of the pile is disturbed and extruded aside in uneven parts where the strength and resistance of the soil is less. Using this, the uneven push of the markers can be explained in respect of the depth (Fig. 11). In the picture we can see the soil extrusion from under the tip of the pile in the direction of the markers.

The extrusion of the soil is indicated by the markers which were installed under the centre of the pile when test № 5 was performed, the markers were found to have been pushed aside up to 10 cm after the driving in of the pile (Fig. 12).

The soil which was extruded aside and which was between the vibrating pile and the strong soil having undisturbed structure was thickened. The laboratory analysis showed the density of the soil which has increased up to 10% in this zone in comparison with the density of the soil which has a natural structure.

When all the noticed peculiarities of the till soil behavior when the pile was driven in using a vibro-hammer have been summarized, we can describe a hypothetical model of its behavior.

The impact zone of the pile driven into the soil is the formation of the mechanical strain variation and separation zone, which is complicated, conditioned by the mechanical effect upon the soil, and which also determines the physical soil alterations.

When the pile is driven in, the soil under the tip of the pile is affected by the external mechanical force which causes the strain in the soil and soil deformations as well as internal mechanical shifts. At the tip of the pile, in the process of the compressibility deformation, a layer of the thickened soil forms and on the edges – there form zones where the shear deformations are transformed into haul deformations. When these zones are interconnected, a cone-shaped pile "tip" from the thickened soil is formed. The laboratory analyses show the soil density in the cone by 10% higher than that having the natural structure, and the angle of the cone point is close to 90 degrees (Fig. 10).

Deformations, which are next to the formed cone and under the tip of the pile, become plastic. The strains in this zone are caused by the mechanical effect of the pile tip and are higher that the strains in the soil which is in the pile environment. Therefore, the deformation is concentric in respect of the pile and stops only in the natural soil when the inner strains caused by the deformation matches the active strains. In the process of this deformation, the zone of the soil having the shape of a hollow cylinder forms around the pile, its thickness is 0.3 D (pile diameter). The inner surface of this zone is restricted by the pile. The layer of the soil, which is 20–30 mm thick and near the wall of the pile, is very stiff (Fig. 7). The external surface of the zone is irregular due to the strength features of the soil in the impact zone (Fig. 11). In the weaker soil, the deformation of compressibility can be considerably higher till the induced strains match externally. In this zone, the density of the soil samples is by up to 10% higher than the natural, but no clear line between the redeployed and the natural soil could be established neither by visual observation nor by laboratory analyses.

The elevation of the soil observed on the ground surface is conditioned by the soil strains which appear at the beginning of the pile driving. The soil elevation is active until the upward internal soil stresses become equal to the overburden pressure by their value.

CONCLUSIONS

The obtained results show that neither of the before-mentioned soil behavior models completely matches the model which is set by testing. The effect zone (soil deformations) of the pile which is driven in by a vibro-hammer in the tested till is up to 1 D. The model of the soil effect when the pile is driven in will be different when the soil strength features are different. This model is hypothetical and will be tested and revised in further tests.

References

1. Lancellota R. 1995. Geotechnical Engineering. Department of structural engineering. Technical University of Turin. Rotterdam, Brookfield: A. A. Balkema. P. 414.
2. Далматов Б. И., Бронин В. Н., Карлов В. Д. и др. 2002. Основания и фундаменты. Часть 2. Основы геотехники. Учебник. М.: Изд-во АСВ; СПб.: СПбГАСУ, 392 с.
3. Chen C. S., Liew S. S. and Tan Y. C. 2005. Time effects on the bearing Capacity of driven piles. SSP Geotchnics Sdn Bhd. Malaysia.
4. Цытович Н. А. 1966. Механика грунтов и фундаментостроение (труды V международного конгресса). Москва: Издательство литературы по строительству. 260–269.
5. Žaržojus G., Sausaitis J. 2004. Plant "Orion Global Pet" Metalo str., Klaipėda. Geotechnical investigation. ISC "Geotestus". (In Lithuanian)
6. Buisman A. S. K. 1935. De Werstand van Paalpunten in Zand. *De Ingeniue*. ***50***. 31–35.
7. Berezancev V. S. 1961. Load-bearing capacity of foundations under eccentric and inclined load. *Proc. of III ICSMFE conf*. ***1***. 440–445.
8. Holeyman A., Vanden Berghe J-F., Chazue N. 2002. Vibratory pile driving and deep soil compaction. Tokyo: A. A. Balkema. P. 233
9. Kulhawy F. H. 1984. Limiting tip and side resistance. Factand fallacy. Analysis and design of pile foundations. ASCE. 337–340.
10. Meyerhof G. G. 1951. The ultimate bearing capacity of foundations. *Geotechnique*. ***2***. 301–332.
11. Meyerhof G. G. 1961. Bearing capacity of foundations under eccentric and inclined load. *Proc. of III ICSMFE conf*. ***1***. 440–445.
12. Skempton A. W. 1951. The bearing capacity of clays. *Bulding Research Congress*. ***1***. 180–189.

Tomas Kairys, Gintaras Žaržojus, Kastytis Dundulis

MORENINIO GRUNTO DEFORMACIJOS ĮSPRAUDŽIANT VIBROPOLIUS

Santrauka

Lietuvoje pastaraisiais metais plačiai naudojami vibropoliai, kurie įspraudžiami į šalies teritorijoje paplitusius moreninius gruntus, tačiau polių ir šių gruntų sąveikos pasekmės nėra nagrinėtos. Įvairių mokslininkų sudaryti mechaniniai proceso modeliai yra labai skirtingi ir nusako nemoreninių gruntų bei kitais būdais įgilintų polių sąveiką.

Polių įspraudimo moreniniuose gruntuose ir gruntų deformacijos stebėjimai Klaipėdoje ir Vilniuje sudarė galimybę ištirti moreninių

gruntų, esančių po polio padu ir aplink jį, deformacijų pobūdį, pakitusių zonų išsidėstymą bei grunto fizikinių savybių pokyčius. Bandymų metu buvo įspraudžiamas 1,5 m ilgio ir 0,45 m skersmens polis, laukiamų deformacijų zonoje įrengta deformacijoms stebėti žymeklių sistema. Atkasus polį buvo matuojamas žymeklių padėties pokytis, imami bandiniai laboratoriniams tyrimams, aprašomas geologinis pjūvis.

Polio pade gilinant polį formuojasi sutankinto grunto sluoksnis, kurio kraštuose intensyvėja šlyties deformacijos. Susijungus šlyties deformacijoms susidaro kūgio formos sutankinto grunto antgalis, kurio smaigalio kampas artimas 90°. Kūgyje grunto tankis padidėja 10%. Už kūgio ir polio šoninėje aplinkoje yra 0,3 polio skersmens deformacijų zona. Prie šoninės polio sienelės stebimas 20–30 mm labai sutankintas moreninio grunto sluoksnis. Išorinis deformacijos zonos paviršius yra netaisyklingos formos, šioje zonoje paimtas grunto bandinio tankis taip pat yra padidėjęs. Polio įgilinimo metu užfiksuotas žemės paviršiaus pakilimas. Gruntas kyla tol, kol grunte sukeltų įtempimų, nukreiptų į viršų, dydžiai susilygina su geostatiniais įtempiais.

Apibendrinant gautus penkių bandymų rezultatus sudarytas hipotetinis polio ir moreninio grunto sąveikos pasekmių modelis.

Томас Кайрис, Гинтарас Жаржоюс, Каститис Дундулис

ДЕФОРМАЦИИ МОРЕННОГО ГРУНТА ПРИ ПОГРУЖЕНИИ ВИБРОСВАЙ

Р е з ю м е

В Литве последнее время широко применяют вибросваи, которые погружают в широко распространенные на территории страны моренные грунты. Однако последствия взаимодействия вибросвай и моренных грунтов практически не исследованы.

Теоретические модели, созданные разными учеными, предполагают другие разности грунтов и иные способы погружения свай.

Осуществленные исследования погружения вибросвай и наблюдения за деформациями моренных грунтов в Клайпеде и Вильнюсе позволили выявить характер деформации как под подошвой сваи, так и в зоне боковой поверхности, распределение зон изменения, а также изменение физических свойств грунтов.

В ходе испытаний погружалась вибросвая длиной 1,5 м и диаметром 0,45 м. В зоне ожидаемых деформаций была установлена система маркеров. После погружения сваи измерялось изменение положения маркеров, отбирались образцы для лабораторных исследований, описывался геологический разрез.

На основе результатов пяти испытаний составлена гипотетическая модель последствий взаимодействия вибросваи и моренного грунта.

При погружении сваи под подошвой формируется слой уплотненного грунта, по краям которого проявляются сдвиговые деформации. После соединения зон пластических деформаций под подошвой сваи образуется конус переуплотненного грунта, угол которого составляет около 90°. Плотность грунта в конусе повысилась на 10%.

За пределами конуса и околобоковом пространстве сваи наблюдалась зона деформации, составляющая около 0,3 диаметра сваи. При боковой поверхности сваи отмечена зона переуплотненного грунта толщиной 20–30 мм. Поверхность зоны деформации криволинейная. Плотность грунта в данной зоне повышена. В процессе погружения сваи, пока дополнительные напряжения не достигают величины природного давления, наблюдалось вспучивание земной поверхности.