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STATIC AND DYNAMIC TESTS OF METAL PILE-SHELLS OF A SEA PIER MARINE TERMINAL

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Abstract. The behavior of the piles in the engineering and geological conditions of the construction water area of a new two-way pier of the berth on the territory of the sea trade port of Chornomorsk as a part of the expansion of production capacities for transshipment of grain, food products, processing of oil and grain crops was studied. During the construction, it is envisaged to use shell piles made of metal pipes 35.0...42.7 m long with the outer diameters 1020, 1220 and 1440 mm.

The layers include Neogene sediments of the upper Miocene subdivision of the Sarmatian and Meotic horizons, which are represented by clayey soils from plastic to hard consistency with the lenses of silty sands saturated with the water and the interlayers of limestone. The piles cut through the sporadic layering of loams and clays, and their bottom ends are stopped in clay and dusty sands.

A comprehensive approach to conducting the control tests of the piles was developed, which included conducting the tests in two stages. At the first stage, a group of four piles with the diameters 1020 and 1220 mm were tested under both static and dynamic loads at the construction site on the first supporting structures. The values of η and M coefficients, which are used to determine the bearing capacity of metal pile-shells during the dynamic tests under special geological conditions, were adjusted. At the second stage, only dynamic loads were tested on the rest of supporting structures.

The inventory metal beam was as a jack pad when testing the piles with static loads, which was fixed to the anchoring piles with the welded terminations, and the movement of the head was recorded by deflection gauges. Pile driving during the dynamic load test was performed with a hydraulic hammer, which was used for their driving.

According to the received data analysis of the pile test results with static and dynamic loads, it was determined that in order to obtain the values of allowable pile design loads, close to those ones determined by the static test results, it is necessary to adjust the values of η and M coefficients. It was established that for metal pile shells, which driven in the marine environment, when their bearing capacity is determined based on the results of test comparisons with static and dynamic loads, the coefficient η should be taken as $2500 kN/m^2$, and $M=1.0$ – when the bottom end of the pile stopped in clay soils and M=1.6 – when in the sand.

Key words: port, pier, hydraulic structure, basin deposits, silt, pile shells, static tests, dynamic tests, bearing capacity.

Introduction. The research object is the engineering and geological conditions of the site of the two-way pier of berth № 10a/11a on the sea trade port territory of Chornomorsk. According to the project, during the construction, the use of shell piles made of metal pipes with the length 35.0...42.7 m with the external diameters 1020, 1220 and 1440 mm is provided. The construction was carried out from the sea area using the barges as the technological platforms, which were maneuvered with the help of tugboats.

Driving of each pile was carried out in two stages. At the first stage it was with a "Dieseko PVE 40VM" vibrating pile hammer until the moment when the speed began to decrease significantly, further driving was performed with a "PE 14/16 NL" hydraulic hammer with the weight of the striking part 16 tons.

The engineering and geological surveys have established significant variability of the sediment bedding, both in depth and in plan, and the site in general can be classified as "unsuitable" for the construction. In such conditions, the project provides for testing the piles with static loads on each supporting structure, there are more than 20 piles. The adopted current construction method, which was supposed to provide the facility commissioning on time, did not allow, taking into account the time for pile resting, the availability of the equipment and its sufficient quantity for simultaneous using (strength beams, jacks, measuring devices), to carry out the tests of all the planned piles with static loads. Therefore, it was urgent and generally important for this type of construction to develop the method of control tests of piles that would provide reliable quality control of their installation and would not negatively affect the critical technological processes of the pier construction.

Analysis of the last research. The shell piles made of metal pipes with an open end have found their application in hydraulic construction [1-3]. Both static and dynamic tests can be performed to confirm their bearing capacity. The priority is given to static tests, as the most reliable, but dynamic tests can also be performed, which are easier to carry out. The optimal variant is when both static and dynamic tests are performed on the same object [4-7, 8], which allows to reduce the number of time-consuming static tests, to cover a larger number of tested piles due to less expensive dynamic tests, to reduce the time for conducting the tests, which allows to apply the construction current method on linear objects.

The bearing capacity of the piles during dynamic tests is determined by the formula, according to the recommendations of State Standard [9], based on the work balance spent on pile driving and soil resistance. In particular, the formula includes the coefficients η and M, which consider the elastic properties of the pile material and the soil conditions of its bottom end location, respectively. These coefficients are established for prismatic reinforced concrete piles based on the experience of joint tests by dynamic and static methods. The peculiarity of the tested piles is that they are partially filled with the soil – from the bottom level of the water area, and the formation of the soil plug in the internal cavity of the piles and its adding in the work depends both on their diameter and the type and characteristics of the soil they cut through [10, 11].

The goal of the research is providing the quality control of pile installation based on the results of their tests with static and, subsequently, dynamic loads, taking into account the current method of pier construction; the correction of the coefficients η and M, which characterize the operation of the piles when their bearing capacity is determined by dynamic loads according to the recommendations of State Standard. The basis for determining the coefficients was a comparison of pile test results under the action of vertical compressive static and dynamic loads.

Research methodology. A comprehensive approach to conducting the control pile tests was developed, which included conducting the tests in two stages. At the first stage, at the construction site, on the supporting structures located at the beginning of the pier, a group of four piles with the diameters 1020 and 1220 mm from the pile field were tested under both static and dynamic loads to compare the obtained results. The correct comparison of pile bearing capacity was provided by the same test conditions for the piles of the same type. In particular, the loading of each pile was completed when close settlement values were obtained, which approached the value $0,2S_u$, at which the bearing capacity of the pile is determined according to the standards.

On the first supporting structures of the pier, the pile tests with static loads were carried out. Later, after resting, they were re-tested with dynamic loads. The obtained results and their analysis made it possible to justify the possibility of replacing the test piles with static loads on dynamic loads on the rest of supporting structures and at the same time not impairing the reliability of quality control of their installation.

At the second stage, the piles with the diameter 1220mm were tested under dynamic loads on the rest of the supporting structures, and three piles with the diameter 1420mm were tested under static loads on the most responsible supporting structures.

The loading method of the piles during testing with static and dynamic loads is adopted in accordance with the requirements of the State Standard of Ukraine [12].

The axial compressive static load on each of the experimental piles was transmitted by the hydraulic jack named DV-400-200, it is capable to transmit the forces up to 4000kN (400ts). The demountable metal beam, which was attached to the anchor piles with the help of welded terminations from the rolled channels, was the jack pad when testing the piles with static loads (Fig. 1). The crown movements of the tested piles were recorded by measuring the movements fixed on the piles terminations relative to the fixed reference system, with PAO-6 protractors.

The piles were driven after resting during the dynamic load test with the hydraulic hammer "PVE 14/16 NL", which was used for their sinking. Pile driving was done sequentially, first with 3, then with 5 strokes. The drop height of the hammer striking part was the same for all the strokes and was 1.1 m.

Research results. According to the geomorphology the site is located within the western (right) underwater coastal slope of Suhiy Liman, the bottom surface of the water area is characterized by the absolute marks -11.8...-13.8m, the adjacent area has the absolute marks +1.7...+1.9m.

Neogene deposits of the Upper Cretaceous subtype of the Sarmatian and Meotic horizons are involved in the geological structure of the planned construction area, which are represented by loam soils from mildly plastic to hard consistency with the silty sand lenses saturated with the water, which lay sporadically, and the layers of limestone. Glacial alluvial-estuarine deposits are located on the eroded surface of the native soils, which are represented by large-clastic and shelly soils, sands, loam soils from solid to very soft consistency, and silt.

The experimental piles from the bottom surface of the water area (-11.8...-13.8m) cut through the following engineering and geological elements (EGE), the thickness passed by the piles is:

EGE2_4^2 – Heavy loam, very soft – 1.65...3.25 m;
EGE2_5^2 – Heavy loam, plastic – 1.0...4.8m;
EGE2_5^3 – Light plastic clay – 1.1...4.2m;
EGE2_6^3 – Heavy, stiff clay – 0.2...0.5 m;
EGE2_7^3 – Light semi-hard clay – 0.8...2.83m;
EGE2_6^2 – Light stiff loam – 2.5...4.5m;
EGE3c^4 – Medium coarse sand – 0.1...0.4m;
EGE3^6 – Wood gravel ground, heterogeneous – 0.3...1.2m;
EGE5_7^2 – Sandy clay, semi-hard – 0.2...0.3m;
EGE5_6^2 – Sandy clay, hard – 0.5...1.4m;
EGE5_7^3 – Light semi-hard clay – 1.0m.

The bottom ends of the tested piles TP-1, TP-2, TP-4 are driven in EGE 5_7^3 – light, hard clay to the depth: TP-1 – 3.8m, TP-2 – 8.6m, TP-4 – 2.5m. The tested pile TP-3 is driven in EGE-5n^1 – silty sand, tight at 2.8 m. The vertical positioning of the tested piles, combined with the engineering and geological cross-section, is shown in Fig. 2.

Fig. 1. Test bench for testing the piles with static loads

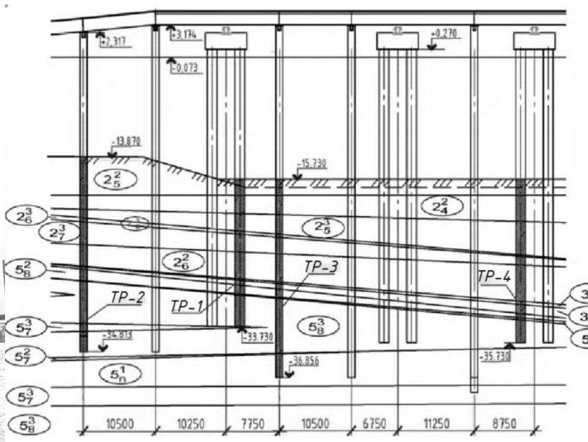

Fig. 2. Position of the piles on the engineering and geological cross-section

The main results of soil tests with the piles under static loads and the tests data with dynamic loads are given in Table. 1. According to the dependence graphs of pile settlement under the load *S=f(P)*, Fig. 3, the bearing capacity was determined for each of the tested piles at the settlement equal to 15 mm, which provided a correct comparison of the obtained results.

Based on the obtained test results, the permissible vertical load on the pile, *N*, was determined according to State Standard [13] by formula (1):

$$N=F_d/\gamma_k\,, \tag{1}$$

where: F_d is the bearing capacity of the pile;

γ_k is the reliability coefficient 1.2 was taken when the bearing capacity of the pile was determined based on the results of the field tests with static loads and 1.4 when tested with dynamic loads.

An individual value of the ultimate resistance F_u, based on the data of pile driving with actual residual failures $S_a>0.002m$ (0.2 cm), obtained when they were tested with dynamic loads, was determined by formula (2):

$$F_u=\frac{\eta\cdot A\cdot M}{2}\left[\sqrt{1+\frac{4\cdot E_d}{\eta\cdot A\cdot Sa}\cdot\frac{m_1+\varepsilon^2\cdot(m_2+m_3)}{m_1+m_2+m_3}}-1\right], \tag{2}$$

where: η is the coefficient, the value of which is recommended to be taken according to Table 1 [5] depending on the pile material, kN/m^2;

A – the cross-sectional area of the pile shaft, m^2;

M – the coefficient, the value depends on the soil type in which the pile end is stopped;

E_d – the calculated hammer stroke energy, $E_d=G(H-h)$;

G – the weight of the hammer striking part, $G=160kN$;

H – the hammer drop, $H=1.1\ m$;

h – for a tubular hammer, $h=0.3\ m$;

$E_d = 160\cdot(1.1-0.3) = 128\ kJ;$

ε^2 – the coefficient, $\varepsilon^2 =0.2$;

s_a – the actual final set from one stroke, m;

m_1 – the hammer mass, $m_1=24.0m$;

m_2 – the mass of pile and crown, t;

$m_3 = 0$ – the mass of the tailstock;

m_4 – the mass of the hammer striking part, $m_4=16$ tons.

Table 1 – Characteristics of the piles and the results of their tests at the first stage

Mark and outer diameter of the pile, mm	Duration of pile rest stat./day test, days	Pile length, m	Total number of hammer strikings with pile driving	Static load tests		Dynamic load tests	
				Bearing capacity of the pile, kN	Stabilized pile settlement, mm	Blow count (number of strokes)	Actual final set from one stroke, S_a, mm
TP-1 1220	18/36	35.0	540	3452	15.00	3	6.0
						5	6.6
TP-2 1020	27/32	37.1	740	3150	15.00	3	3.0
						5	3.2
TP-3 1020	25/30	42.7	808	3321	15.00	3	6.7
						5	12.0
TP-4 1220	28/33	37.0	385	3550	15.00	3	5.5
						5	6.6

According to the received data results of the pile test results with static and dynamic loads, it was determined that in order to obtain the values of allowable calculated pile loads, close to those determined according to the static test results, it is necessary to adjust the coefficient values of the formula (2). In particular, the use of the values of coefficients η and M given in State Standard [9] did not allow to obtain the allowable calculated pile load close to the values, obtained when the pile was tested with static loads. Thus, in the State Standard recommendations, the value of the coefficient η for metal pile shells is completely absent, for metal piles this value is recommended to be 5000 kN/m^2.

On the basis of the conducted tests, the dependence graphs $Fu_{dyn} = f(\eta)$, Fig. 4, which made it possible to determine the value of the coefficient η for each pile, under the condition that $Fu_{dyn} \approx 1{,}4N_{stat}$, when the accepted pile load was the same when determined both by the results of static and dynamic pile tests. To construct the graphs, the value of the vertical ultimate resistance when testing the piles with dynamic loads, $Fu_{dyn.}$, was determined with a step change of the coefficient η from 1500 to 4000. At the same time, the value of the coefficient M was taken to be 1.0 – for clay soils and 1.6 – for sandy soils. The results of Fu_{dyn} are shown in Table 2.

Table 2 – Calculated results of the pile ultimate resistance at the first test stage

Conventional symbol of the tested pile	Diameter, mm	Allowable calculated pile load, when tested with static loads, kN		Limit values of the vertical resistance when testing piles with dynamic loads, Fu_{dyn} with η, kN					The value η, kN at the ratio $1.4N_{stat}/Fu_{dyn} \approx 1$
		N_{stat}	$1.4N_{stat}$	1500	2000	2500	3000	4000	
TP-1	1220	2877	4028	3193	3559	4116	4136	4570	2420
TP-2	1020	2625	3675	4210	4929	5418	5849	6569	1240
TP-3	1020	2767	3874	3111	3534	3825	4121	4470	2580
TP-4	1220	2858	4001	3150	3515	4054	4080	4481	2450

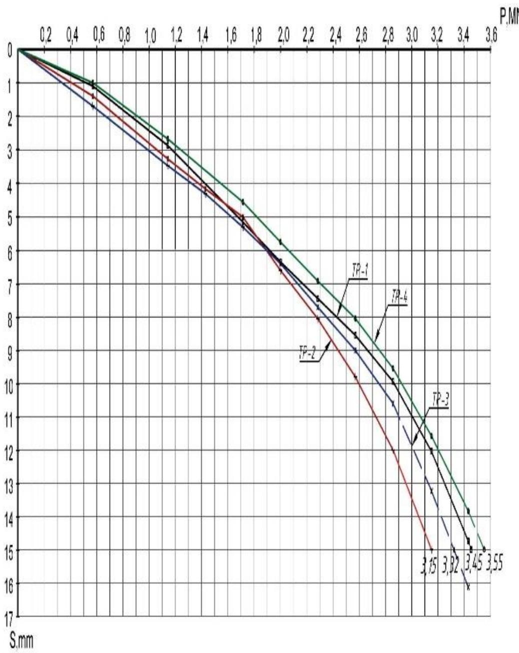

Fig. 3. Dependence graphs of pile settlement under the load

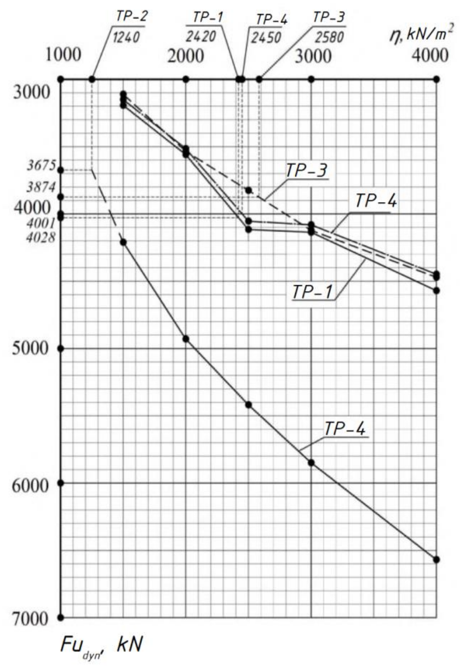

Fig. 4. Dependence graphs $Fu_{dyn} = f(\eta)$

The necessity for adjusting the coefficients η and M is due to the peculiarities of both the engineering and geological conditions of the bottom soils and the construction of the piles. The bottom soils are characterized by significant stratum irregularity, each of the piles cuts through selected EGE, which have different thickness and density. In the process of pile driving, their inner cavity is filled with the soil, which is compacted, in particular, under the action of friction against the inner wall. In addition, for shell piles, the behavior of the bottom end largely depends on the degree of soil compaction, which fills the cavity in the final part of the pile, where the formation of a soil plug with a self-closing effect is possible. And this process is difficult in heterogeneous soils, and the density is significantly affected by the size of the pile absolute diameter.

The results of pile dynamic load tests with the diameter 1220 mm at the second stage are shown in Table 3. The bearing capacity of the piles was determined by the formula (2), using the coefficients determined at the first stage of testing, which were taken as $\eta=2500\ kN/m^2$, $M=1.0$ – when the bottom end of the pile stopped in clay soils and $M=1.6$ – when in sand. The obtained data for all the test piles confirmed the calculated pile load accepted in the project.

Table 3 – Results of pile tests at the second stage

Conventional symbol of tested pile (axis of the clump location)	Duration of rest, days	Pile length, *m*	The total weight of the pile with the soil and the head, m_2, *t*	Operating time of the vibro-driver+the number of hammer blows	Soil conditions under the pile toe	Actual failure of the pile S_a, *mm*	F_u, *kN*	Permissible pile load, *N*, *kN*	Design pile calculated load, *kN*
1	2	3	4	5	6	7	8	9	10
TP-6 (13)		41.0	74.3	537bl	sand	10	4240	3029	2640
TP-7 (19)		41.0	74.3	68min	sand	7	5372	3837	2640
TP-8 (21)		41.0	74.3	66min	clay	6	3710	2650	2640
TP-11 (25)	22	39.6	71.0	20.8min	clay	6	3752	2680	2640
TP-13 (28)	8	41.0	74.3	15min+390bl	sand	9.5	3970	2835	2640
TP-14 (30)	9	41.5	75.5	12min+740bl	sand	10	4224	3017	2640
TP-15 (33)	9	41.5	75.5	45min+199bl	sand	9	4536	3240	2640
TP-16 (33)	9	41.5	75.5	51min+162bl	sand	9.5	4445	3175	2640
TP-17 (36)	5	39.6	71.0	45min+350 bl	sand	8.5	4784	3417	2376
TP-18 (36)	5	39.6	71.0	48min+340bl	sand	8.0	4981	3559	2376
TP-19 (39)	12	37.0	65.2	39min+196bl	clay	6.5	3637	2598	2376
TP-20 (39)	12	37.0	65.2	36min+227bl	clay	5	4297	3069	2376
TP-21 (42)	5	35.0	60.5	30min+296bl	clay	8.5	3113	2223	2376

Conclusions:

1. The results of pile testing with static loads, which are determined by the regulatory requirements, are a convincing and reliable experimental evidence that confirms the validity of the adopted design considerations when driving pile foundations of the buildings.

2. It was established that the conditions under which the results of pile tests with dynamic loads are correlated with the test results with static loads, which made it possible to conduct the tests with dynamic loads for piles with the diameter 1220 mm in the next sections of the pier construction.

3. It has been established that for metal pile shells, which are driven in the sea water area, when their bearing capacity is determined based on the comparison of test results with static and

dynamic loads, the coefficient η should be taken as $2500kN/m^2$.

4. The database seeding with the pile test results with static and dynamic loads in similar complex engineering and geological conditions can be the basis for adjusting the adopted normative calculation coefficients, as well as for effective optimization of foundation design decisions.

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СТАТИЧНІ І ДИНАМІЧНІ ВИПРОБУВАННЯ МЕТАЛЕВИХ ПАЛЬ-ОБОЛОНОК МОРСЬКОГО ПРИЧАЛУ

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Анотація. Досліджувалась робота паль в інженерно-геологічних умовах акваторії будівництва нового двостороннього пірсу причалу на території морського торгового порту м. Чорноморська у складі розширення виробничих потужностей з перевалки зерна, харчових продуктів, переробки олійних та зернових культур. При будівництві передбачено застосування паль оболонок із металевих труб довжиною 35,0…42,7м з зовнішніми діаметрами 1020, 1220 і 1440 мм.

Нашарування включають неогенові відкладення верхнеміоценового підвідділу сарматського та меотичного горизонтів, які представлені глинистими ґрунтами від м'яко пластичної до твердої консистенції з лінзами пилуватих пісків насичених водою та прошарками вапняків. Палі перерізають спорадичне нашарування суглинків і глин, а їх нижні кінці зупинені в глині і в пилуватих пісках.

Розроблено комплексний підхід до проведення контрольних випробувань паль, який включав проведення випробувань в два етапи. На першому етапі на майданчику будівництва на перших опорах проводились випробування групи із чотирьох паль діаметрами 1020 і 1220 мм як статичними, так і динамічними навантаженнями. Було проведення корегування значень коефіцієнтів η і M, які використовуються при визначені несучої здатності металевих паль-оболонок при динамічних випробуваннях в особливих геологічних умовах. На другому етапі на подальших опорах виконувались випробування тільки динамічними навантаженнями.

Упором для домкрата при випробуванні паль статичними навантаженнями слугувала інвентарна металева балка, яка закріплялась до анкерних паль привареними випусками, а переміщення голови фіксувалися прогиномірами. Добивання паль при випробуванні динамічними навантаженнями виконували гідромолотом, якій використовувався для їх занурення.

По результатам аналізу отриманих даних результатів випробувань паль статичними і динамічними навантаженнями визначено, що для отримання величин допустимих розрахункових навантажень на палю, близьких до визначених за результатами статичних випробувань, потрібне корегування значень коефіцієнтів η і M. Встановлено, що для металевих паль-оболонок, які занурені в акваторії моря, при визначені їх несучої здатності за результатами порівнянь випробувань статичними і динамічними навантаженнями коефіцієнт η слід приймати $2500 кН/м^2$, а $M=1,0$ – коли нижній кінець палі зупинявся в глинистих ґрунтах і $M=1,6$ – коли в пісках.

Ключові слова: порт, пірс, гідротехнічна споруда, лиманні відкладення, мул, палі оболонки, статичні випробування, динамічні випробування, несуча здатність.

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