WARSAW UNIVERSITY OF TECHNOLOGY Index 351733
FACULTY OF CIVIL ENGINEERING
COMMITTEE FOR CIVIL AND WATER ENGINEERING
POLISH ACADEMY OF SCIENCES ISSN 1230-2945

DOI: 10.24425/ace.2025.154134
ARCHIVES OF CIVIL ENGINEERING
Vol. LXXI ISSUE 2 2025

 pp. 503 –519

Research paper

Model studies and quantitative evaluation of the reduction of gyttja consolidation settlement by different types of piled raft foundations

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Abstract: Combined Pile-Raft Foundations (CPRF) are widely employed to mitigate vertical settlement in building and engineering structures. For structures resting on cohesive and organic soils, understanding the time required for complete stabilization of settlement is crucial. The total settlement involves instantaneous settlement (elastic ground deformation), consolidation settlement (water squeezing from pore space), and secondary settlement (structural changes in the ground skeleton, known as secondary consolidation or soil creep). Urban expansion, notably in cities like Warsaw, compels developers to construct in previously overlooked areas, potentially containing organic carbonate sediments like gyttja and chalk. Buildings on such organic soils often settle due to gyttja consolidation during construction and operation. Analyzing long-term settlement, especially of CPRF on reconsolidated organic soils, becomes paramount. Model tests on a laboratory scale offer a cost-effective alternative to large-scale tests, providing quantitative insights into CPRF settlement reduction through piles. This study presents results from model tests conducted on natural organic soils, enabling the prediction of CPRF settlement solely based on gyttja's geotechnical parameters.

Keywords: CPRF, combined pile-raft foundation, settlement, consolidation of gyttja, soil compressibility, coefficient of consolidation, settlement reduction

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1. Model studies

In the early 1970s, monitoring of buildings on slab-on-grade foundations began in England [1]. In 1990 and the following years, during the erection of high-rise buildings in Frankfurt, Katzenbach et al. [2], conducted automated measurements of the piles and subsoil under the slab foundations. The information gained from these observations allowed a better understanding of the mechanisms accompanying the operation of Combined Pile-Raft Foundations (CPRF). Mandolini and Viggiani [3] and Mandolini et al. [4] collected 42 well-documented case studies of pile-raft foundation settlement. Some of them showed a significant increase in settlement in the final stage of the monitored structures, resulting from consolidation settlement of fine-grained soils [2, 3] and/or creep of cohesive soils [3]. Hooper and Wood [5] compared the settlement of a slab and a slab-pile system in London silt, ultimately concluding that the monitored final long-term settlement of the direct-foundation slab was 1,5 times greater than the settlement observed at the end of the execution phase, while the final settlement of the pile-raft foundation was close to the value of the settlement observed at the end of the building erection phase. The information collected by Morton and Au [6] from the monitoring of 7 buildings founded in London silt made it possible to conclude that the ratio of the settlement from the end of the execution phase to the total settlement was in the range between 0,4 and 0,7 (final long-term settlement of the direct foundation slab was 1.43–2.5 times greater than the settlement observed at the end of the building erection phase), regardless of whether the building was founded directly or on a slab-and-pile foundation. Summarizing the observations cited above, it can be concluded that the time-varying settlement of pile-raft foundations plays an important role in their design. All the abovementioned cases involved foundations founded on "strong" cohesive mineral soils (reconsolidated London and Frankfurt clays).

The idea of in-house model testing of foundations on gyttja was initiated immediately after large-scale load tests were carried out in the Rynna Żoliborska on a foundation founded directly, as well as on a foundation reinforced with displacement concrete columns [7–9]. Due to the cost and technical difficulties associated with inflicting large loads, field tests were limited to examining only the slab and the slab founded on a foundation reinforced with 9 columns. The authors prepared laboratory-scale test stands while maintaining the interrelationships of the elements building the pile-raft foundation, which was tested *in-situ* and in soil conditions corresponding to the natural ones from the study area. The difference in the laboratory tests compared to the field tests was the assumption that both the slab and piles work in gyttjas, without resting the pile bases on the bearing layer. Thanks to this procedure, results showed that the process of foundation settlement depended only on the geotechnical parameters of the gyttja. Own laboratory-scale model tests of pile-raft foundations were prepared and carried out similarly to the tests performed by Bajad and Sahu [10]. These studies included, among other things, observation of settlement over time of pile-raft foundations modeled as a system of a 100 mm wide square steel slab cooperating with 1, 4, 9 and 16 piles made of steel rods of 10 mm diameter and 100 mm length. The tests were carried out in a tank with a diameter of 600 mm and a height of 500 mm and were performed at the Geotechnics Laboratory in the Department of Geotechnics and Underground Structures, Faculty of Civil Engineering, Warsaw University of Technology. The tests included test loading of a direct-founded slab model, as well as 3 pile-raft foundation models (slab $+2 \times 2$, 3×3 and 4×4 piles). Independently, 4 containers filled with reworked soil of Eemian gyttja lithology were prepared. In the course of preparing the soil, its original structure was partially altered, depriving it of its heredity [11], understood as structural reinforcement [12] related to its geological history. Subsequently, the soil substrate was reconstituted by compaction to obtain a homogeneous model with parameters similar to those *in-situ*.

Complementary to 1:1 scale monitoring of civil structures, model studies can be carried out. The studies known from the literature have been carried out primarily in sandy soil [7] to [8] and [9–12]. So far, the subject of the interaction of pile-raft foundations with organic or poorly consolidated cohesive mineral subsoil has been addressed by a narrow range of researchers [10–15]. The model studies presented in this chapter were performed to analyse the consolidation settlement of pile-raft foundations over time. The authors focused on changes of the coefficient of consolidation in the function of the number of piles in the CPRF. An additional application goal was to indicate the quantitative effect of piles on the reduction in settlement of pile-raft foundations.

Organic soils are almost always (e.g., according to Wiłun [16]) assigned very high moisture content (100–2200%), low shear strength ($\varphi = 0 - 10°$; $c = 2 - 20$ kPa) and high compressibility ($M_0 = 0.2 - 2$ MPa). Given the characteristics of the organic soils he studied, Wolski [17] assigned organic and mineral gyttjas shear strength $c_u = 8 - 26$ kPa. Such low values of geotechnical parameters caused gyttjas to be formerly classified as weak-bearing substrates [18] and only in certain cases were they considered together with lake chalk as soil for the foundation of buildings [19]. Extrapolation of the results collected within the framework of the present study makes it possible to extend the conclusions from an eminently local character to older organic soils, primarily gyttjas in other non-Warsaw areas.

Field tests (min. CPT/CPTU static soundings (Fig. 1), carried out at test load sites and from other test plots, provided results that were superimposed on selected SBT (soil behaviour type) standard classification nomograms. Interpretation of about 6500 individual measurements made on 55 static soundings at 2 locations from Warsaw (the vicinity of the Siberian Deportees

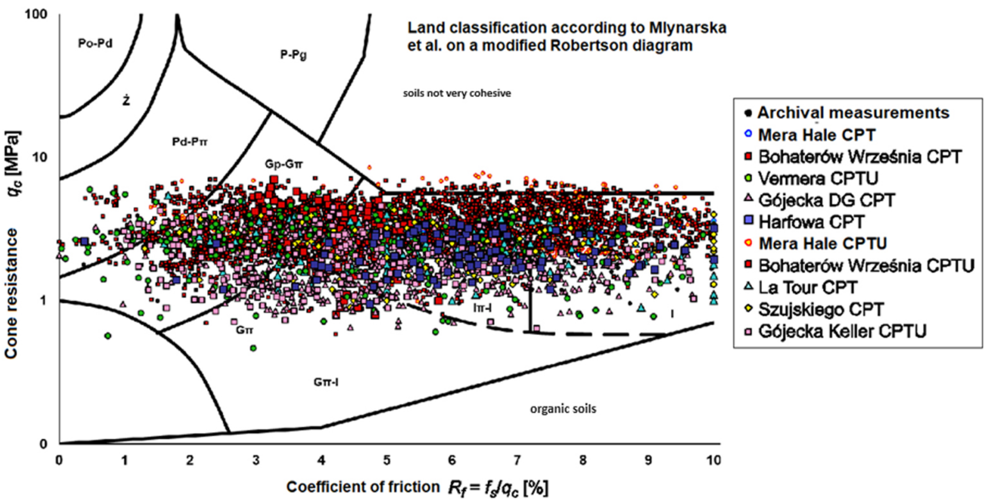

Fig. 1. Results of CPT/CPTU studies conducted in the vicinity of the Siberian Deportees Roundabout and Grójecka Street in Warsaw [20], [21]

Roundabout and the vicinity of Grójecka Street), indicated that gyttja, which are organic soils with variable contents of organic parts, carbonate parts and other mineral parts, behave like mineral soils with dominant clay and silt fractions.

Based on the above observations, the analysed soils were considered mineral soils and assumed the possibility of interpretation according to the rules developed for mineral soils.

2. Reconstitutedsoil and the methodology of model studies

Four containers with a diameter of 310.50 mm and a height of 40 m mm were filled with soil, which was compacted under laboratory conditions using a lightweight dynamic plate (LPD). Compaction of the crumbled soil in the form of a homogeneous paste was carried out by placing 8–9 cm thick layers in samplers, up to a height of 35 cm. Each layer was compacted with LPD by compacting 3 times (a 10 kg weight falling from a constant height). A dynamic modulus reading of E_{vd} [MPa] was taken as a control each time. In addition, selected mechanical parameters were measured in the soil each time (e.g., shear strength under no-drain conditions s_u [kPa]).

Taking the simplification that the obtained average value of the dynamic modulus E_{vd} = 5.7 MPa, determined by the dynamic deflection meter method, is close to the value of the secondary static modulus E_{v2} = 4.2 MPa. This value should be considered similar to the values of secondary moduli obtained by Pietrzykowski [21] in laboratory tests of E = 6.7 MPa (a value interpreted from M = 11.8 MPa at ν = 0.37, as an average value from 13 measurements).

Additional comparison of the estimated value of the secondary static modulus E_{v2} with the values of the strain moduli $E = 4 - 11$ MPa from the triaxial compression apparatus test under no-drain TXCIU conditions (Table 1) from the large-dimensional test site, for axial strain ε_1 = 1% indicates the similarity of E_{v2} to the lower limit of the interval of E values from the test site. The coincidence of the documented measurements from the monograph studies and the large-scale tests, as well as the control measurements at the reconstitution of the specimens, confirms the correct preparation of the specimens for the model tests in terms of the tested deformations. The collated values confirm that the prepared soil was characterized by homogeneous properties that do not deviate from the *in-situ* parameters attributed to the gyttjas in question and described in industry publications and/or studies documenting soil tests for the foundation of buildings.

Table 1. Parameters of gyttja from tests in triaxial compression apparatus under conditions without drainage TXCIU from the area of large-scale load tests

10	**Depth Retrieval [m]**	**Type of test**	**Final stresses consolidation**		**Inner angle of attack ϕ' [°]**	**Cohesion c' [kPa]**	**$E_{1\%}$ at min. σ'_h [MPa]**	s_u
			σ'_v [MPa]	**σ'_h [MPa]**				
P1	5.6 ÷ 6.2	TXCIU	50 150 300	50 150 300	30	4	4	48.5

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Table 1 – *Continued from previous page*

10	**Depth Retrieval [m]**	**Type of test**	**Final stresses consolidation**		**Inner angle of attack ϕ' [°]**	**Cohesion c' [kPa]**	**$E_{1\%}$ at min. σ'_h [MPa]**	s_u
			σ'_v **[MPa]**	σ'_h **[MPa]**				
P1	7.6 ÷ 8.2	TXCIU	100 200 300	100 200 300	31	4	8	62.5
P1	9.6 ÷ 10.2	TXCIU	150 250 400	150 250 400	31	9	8	112.5
P10	7.2 ÷ 7.5	TXCIU	200 275 350	200 275 350	33	8	11	105
P10	8.7 ÷ 9.2	TXCIU	100 200 300	100 200 300	33	11	9	70
P10	10.0 ÷ 10.5	TXCIU	100 200 300	100 200 300	28	2	6	47.5

In the containers, foundation models were prepared in four variants (see Figure 2), respectively:

1. a square slab with side $B_r = 100$ mm;
2. a slab based on a group of 4 piles (in a 2×2 arrangement);
3. a slab based on a group of 9 piles (in a 3×3 arrangement);
4. a slab based on a group of 16 piles (in a 4×4 arrangement).

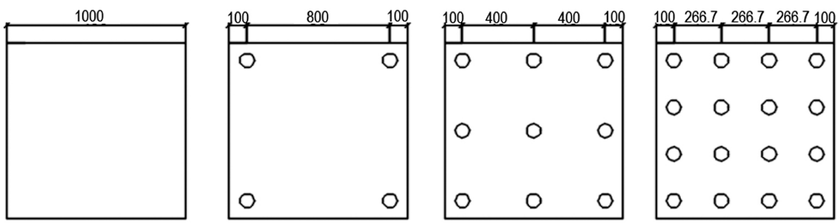

Fig. 2. Diagram of pile placement under the foundation slab, model tests

The geometry of the piled raft systems for the model tests was selected proportionally to the geometry of the system tested in natural conditions described in [7–9]. The ratio of pile length/ slab width $L/B_r = 2.0$ was maintained. Pile axial spacing was set of $0.8B_r$ for 4 piles, $0.4B_r$ for 9 piles, $0.27B_r$ for 16 piles. The alignment of the edge/corner pile $0.1B_r$ from the edge of the slab was also maintained. The length of the pile models equal to 200 mm was selected for the height of the container so that the zone of influence of loads did not reach the bottom of the container and conventionally extinguished in the ground without hitting the "resistance" of the bottom of the container. The zone of influence was estimated at about

150 mm as 1.5 times of slab width measured from pile toe. Cement "piles" were prepared in the form of a paste, which was injected into previously drilled and "cleaned" holes in the ground, and then cemented together with a foundation "plate" set on the ground surface. Schematically, the following steps were carried out for each site:

1. Drilling a pile hole with a drill bit
2. Inserting the casing pipe to the depth of the drilled hole (plastic pipe with an outer diameter of D = 12 mm and a wall thickness of 0.5 mm).
3. Cleaning the hole inside the casing pipe with a drill.
4. Concreting the piles; cement mortar was used for concreting, consisting of cement, sand and water in a ratio of about 0.3:0.3:0.4.
5. Installing reinforcement in the form of a rod with a diameter of 5 mm.
6. Placing the formwork and making the foundation slab.
7. Making a 100 mm × 100 mm and 10 mm thick unreinforced slab. Cement mortar was used – cement, sand and water in proportions of about 0.3:0.3:0.4.

The time to lay the soil in the samplers was about 1 day. Piled raft foundations were also made in 1 day.

The structures modelling the direct and intermediate foundations were subjected to loading, unloading and reloading in a total of 24 stages (for configuration as plate + piles: 0.66–377.37 kPa). The load for each type of foundation consisted of weights suspended from a steel arm resting on the slab at one point and attached to a steel beam perpendicular to the arm at another point.

On the top surface of each slab were 2 dial gauges measuring slab settlements with a reading accuracy of 0.001 mm. During the loading, the soil in the testers was protected from drying out with foil and water-soaked blotting paper. After the test was completed, the test stand was dismantled, and soil samples were taken from the samplers for testing its physical-mechanical properties (Table 2).

The non-drain shear strength values from the rotary shear and piston penetrometer soil tests after the model tests (Table 2) correspond to the s_u values from the TXTCU test in the triaxial compression apparatus.

The obtained average value of dynamic modulus E_{vd} = 9.5 MPa determined by the dynamic deflection gauge method, corresponding to the value of secondary static modulus E_{v2} = 12.0 MPa (based on Road and Bridge Research Institute correlation [22]) is higher than the initial values (E_{vd} = 5.7 MPa and E_{v2} = 4.2 MPa. This higher value of modulus may be due to a change in the consistency of the model soils during the test, i.e. a transition from a hard-plastic state to a semi-hard-plastic state due to drying, which could not technically be avoided with such a long test time. An additional (parallel) cause may have been consolidation changes in the subsoil in the stress-affected region during the testing.

In general, it can be summarized that the results of laboratory tests after the completion of model loading tests of pile-raft foundations at all test sites, confirm that the subsoil is characterized by similar physical-mechanical parameters (granulometry, strength and deformation characteristics). Hence, it can be considered reasonable that all the tested foundations in the model tests operate under similar soil conditions are similar to the real ones in which the high-dimensional measurements were conducted.

Table 2. Values of physical and mechanical parameters of soil from model sampler.

Type foundation	**Location download samples**	**Rotary shear** s_u	**Piston Penetrometer** s_u	**Av.** w_n	I_L	**Plate dynamic Module**	**Cylinders for TXT**	φ_{uu}	c_{uu}	f_p	$\frac{f}{\pi}$	f_I
		kPa	**kPa**	%		**MPa**		**[°]**	**[kPa]**	%	%	%
1–2 slab	at the tile level	71		33.58	–	–				27	54	19
	Below the plate (between the cylinders)			34.2			$\Delta = 1 - 5$					
	20 cm below tile)	65	–	35.39	< 0	9,72	$\Delta = 6 - 10$					
	Below cylinders			36.31				$\sigma_3 = 75 - 150 - 300$ 8 $\sigma_3 = 100 - 200 - 400$ 10	95 85	25	56	19
3–4 9 piles	at the tile level	60	$\boldsymbol{I_L = 0}$ $s_u = 71.93$ $q_f = 410$	39.36	$I_L = 0.01$					25	55	19
	Top of the pile			36.57	$I_L = 0.01$		$\Delta = 1 - 4$					
	In the middle of the pile		$\boldsymbol{I_L = 0.05}$ $s_u = 52.63$ $q_f = 300.0$	37.7								
	Bottom of the pile (20 cm below the tile)	50	$\boldsymbol{I_L = 0}$ $s_u = 70.70$ $q_f = 403.0$	38,18	$I_L < 0$	8.28	$\Delta = 5 - 9$					
	The bottom of the bubble			37.32				$\sigma_3 = 75 - 100 - 200$ 6 $\sigma_3 = 100 - 200 - 300$ 5	80 90	22	59	19

Table 2 continued on the next page

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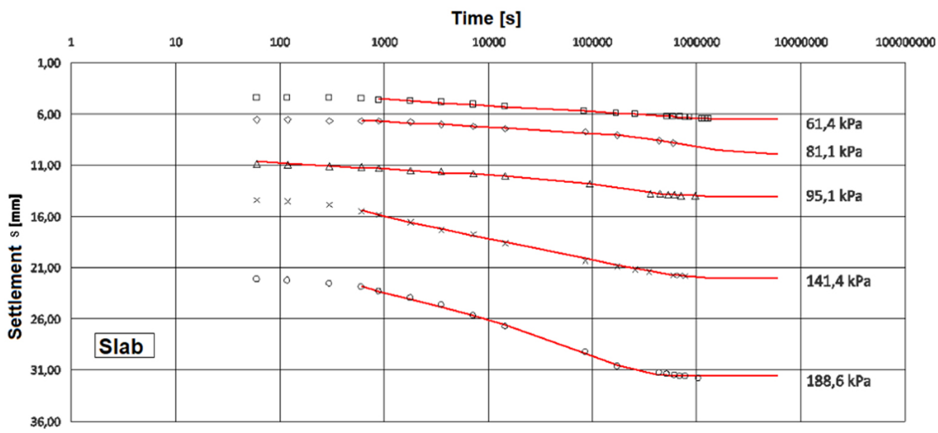
Type foundation	**Location download samples**	**Rotary shear** s_u	**Piston Penetrometer** s_u	**Av.** w_n	I_L	**Plate dynamic Module**	**Cylinders for TXT**	φ_{uu}	c_{uu}	f_p	f_π	f_I
		kPa	**kPa**	%		**MPa**		**[º]**	**[kPa]**	%	%	%
5–6 4 piles	at the tile level	53	$\boldsymbol{I_L}$ **= 0.04** $s_u = 59.65$ $q_f = 340.0$	39.46	$I_L = 0.01$		$\Delta = 1-4$ $\Delta = 0$			22	55	23
	The center of the pile under the tile		$\boldsymbol{I_L}$ **= 0.05** $s_u = 54.39$ $q_f = 310.0$	36.99								
	Bottom of the pile (20 cm below tile)	52	$\boldsymbol{I_L}$ **= 0.02** $s_u = 70.04$ $q_f = 360.0$	37.32	$I_L = 0.01$	10.42	$\Delta = 5-9$	$\sigma_3 = 75-150-250$ 3 $\sigma_3 = 100-200-300$ 2	90 100	24	53	23
7–8 16 piles	Under tile	59	$\boldsymbol{I_L}$ **= 0** $s_u = 75.44$ $q_f = 430.0$	37.32	$I_L = 0.03$		$\Delta = 1-4$			23	53	24
	Top of the pile		$\boldsymbol{I_L}$ **= 0.2** $s_u = 38.60$ $q_f = 220.0$	38.85								
	Bottom of the pile		$\boldsymbol{I_L}$ **= 0.04** $s_u = 60.00$ $q_f = 342.0$	38.01								
	Bottom of the pile (20 cm below tile)	55	$\boldsymbol{I_L}$ **= 0** $s_u = 67.89$ $q_f = 387.0$	38.01	$I_L = 0.04$		$\Delta = 5-9$	$\sigma_3 = 100-200-300$ 7	85	22	54	24

3. Survey results and their analysis including calculation of consolidation settlement

The results of the tests were compiled into tabular lists. Based on them, graphs analogous to those of typical oedometer tests were prepared, i.e. consolidation curves showing foundation settlement over time at each degree of applied load on a time-element scale.

The results from the model tests for load stage of 62 kPa, 82 kPa, 95 kPa, 141 kPa and 188 kPa were used to analyse the process of consolidation settlement, as well as to quantify the reduction of this type of settlement through different types of pile-raft foundations (taking into account the varying number of piles). In order to estimate the settlement stabilization time and determine the final settlement, the consolidation curves from own model tests were analysed by fitting a Meyer curve to them (Fig. 3– 6). The consolidation curves of the test bed without and with reinforcement by making piles under the slab take the shape of an "inverted S", typical of oedometer tests.

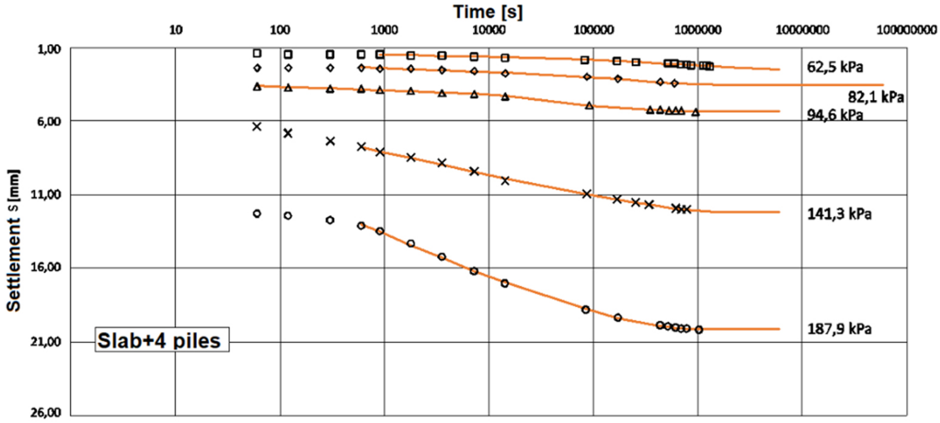
According to the current PN-EN ISO 17892-5 [22] standard originating in *British Standards* and regulating the methodology and interpretation of oedometer testing, testing in the oedometer is carried out within 24 hours (t1day = 86400 s). Settlements mobilized later, i.e., after 24 hours (t1day = 86400 s) until they reached a value of s∞, were considered to be settlements mobilized during the building operation stage. The final settlement, approximated by the Meyer curve, was therefore related to the settlement after the conventional time of the end of the



σ [kPa]	188.6	141.4	95.1	81.1	61.4	
S∞	31.60	22.05	14.07	9.93	6.60	
D	0.68	0.52	1.15	0.76	0.58	
p	0.10	0.13	0.05	0.06	0.11	
alpha	6.62E-06	1.94E-06	3.66E-06	9.25E-07	1.15E-06	
t1day [S]	86400	86400	86400	86400	86400	
S∞/St1day	1.08	1.09	1.10	1.27	1.15	1.10

σ [kPa]	188.6	141.4	95.1	81.1	61.4	
cv [m2/s] h=0.10m	1.10E-07	1.91E-07	4.50E-08	1.16E-08	5.71E-08	5.71E-08
cv [m2/s] h=0.15m	2.47E-07	4.29E-07	1.01E-07	2.62E-08	1.28E-07	1.28E-07

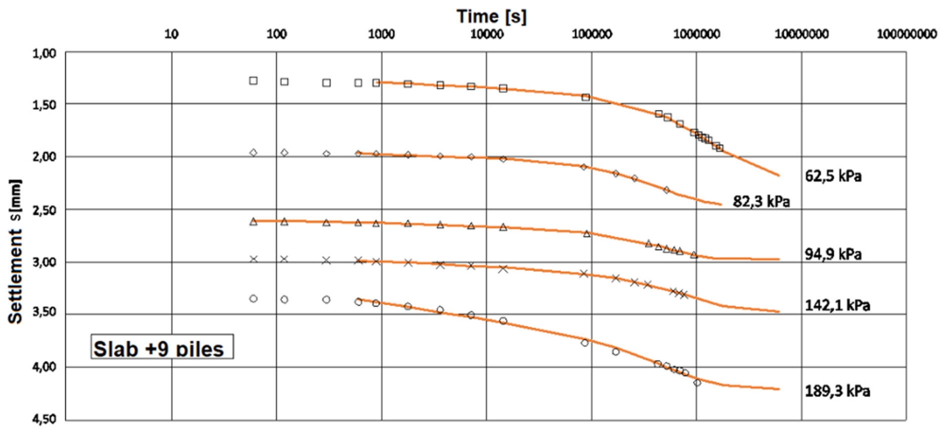
Fig. 3. The slab without piles; approximation of selected compressibility curves with Meyer curve to determine total consolidation settlement; contribution of total settlement to settlement after 24 h; values of the c_v coefficient for H_{dr} = 10 and 15 cm



σ [kPa]	187.9	141.3	94.6	82.1	62.5	
S∞	20.17	12.19	5.31	3.51	2.51	
D	0.34	0.37	0.94	0.69	0.54	
p	0.17	0.15	0.05	0.08	0.07	
alpha	2.66E-06	1.41E-06	1.07E-05	3.44E-06	6.74E-07	
t1day [S]	86400	86400	86400	86400	86400	
S∞/St1day	1.07	1.11	1.08	1.17	1.37	1.11

σ [kPa]	187.9	141.3	94.6	82.1	62.5	
cv [m2/s] h=0.10m	2.53E-07	3.63E-07	7.04E-08	3.37E-08	7.35E-09	7.04E-08
cv [m2/s] h=0.15m	5.69E-07	8.17E-07	1.58E-07	7.59E-08	1.65E-08	1.58E-07

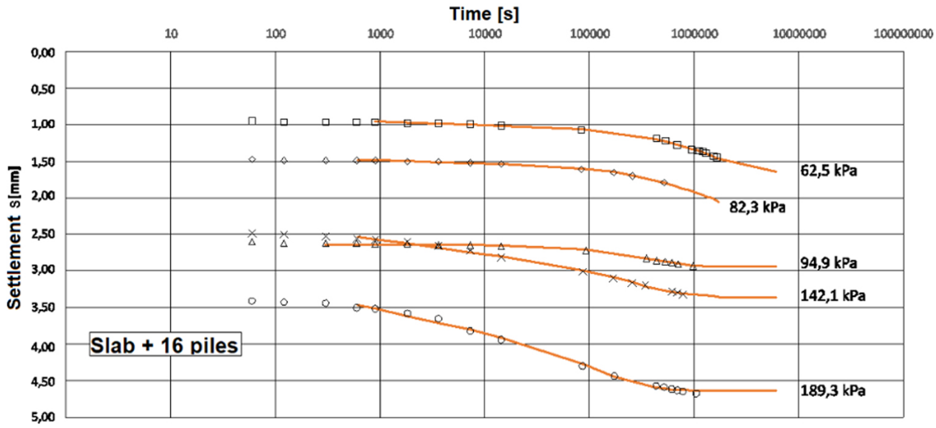
Fig. 4. The slab with 4 piles; approximation of selected compressibility curves with Meyer curve to determine total consolidation settlement; contribution of total settlement to settlement after 24 h; values of the c_v coefficient for H_{dr} = 10 and 15 cm



σ [kPa]	189.3	141.4	94.9	82.3	62.5	
S∞	4.20	3.47	2.98	2.46	2.20	
D	1.14	1.71	1.98	1.46	0.75	
p	0.05	0.02	0.01	0.01	0.02	
alpha	1.47E-06	1.02E-06	1.88E-06	2.03E-06	6.08E-07	
t1day [S]	86400	86400	86400	86400	86400	
S∞/St1day	1.13	1.11	1.10	1.17	1.55	1.13

σ [kPa]	189.3	141.4	94.9	82.3	62.5	
cv [m2/s] h=0.10m	1.77E-08	5.55E-09	7.82E-09	7.73E-09	2.33E-09	7.73E-09
cv [m2/s] h=0.15m	3.99E-08	1.25E-08	1.76E-08	1.74E-08	5.25E-09	1.74E-08

Fig. 5. The slab with 9 piles; approximation of selected compressibility curves with Meyer curve to determine total consolidation settlement; contribution of total settlement to settlement after 24 h; values of the c_v coefficient for H_{dr} = 10 and 15 cm



σ [kPa]	189.2	141.9	94.9	82.3	62.5	
S∞	4.63	3.36	2.94	2.23	1.66	
D	0.82	0.90	2.26	0.95	0.73	
p	0.08	0.07	0.00	0.02	0.03	
alpha	6.14E-06	2.61E-06	2.80E-06	7.15E-07	6.03E-07	
t1day [S]	86400	86400	86400	86400	86400	
S∞/St1day	1.08	1.12	1.09	1.39	1.55	1.12

σ [kPa]	189.2	141.9	94.9	82.3	62.5	
cv [m2/s] h=0.10m	6.40E-08	3.20E-08	8.02E-09	4.21E-09	2.37E-09	8.02E-09
cv [m2/s] h=0.15m	1.44E-07	7.21E-08	1.81E-08	9.48E-09	5.34E-09	1.81E-08

Fig. 6. The slab with 16 piles; approximation of selected compressibility curves with Meyer curve to determine total consolidation settlement; contribution of total settlement to settlement after 24 h; values of the c_v coefficient for H_{dr} = 10 and 15 cm

building execution phase, i.e. after 24 hours, thus defining the share of operational settlement (presented in Figures 3, 4, 5, 6). The averaged results from 5 loading steps (from 62 kPa to 189 kPa as the most common loading range for foundations of typical buildings) for each model (slab, slab with 4, 9 and 16 piles) indicate that the final settlement s∞ based on intepretation is 1,10–1,13 times greater than the settlement mobilized at the contractual completion phase of the structure (after t1day = 24 h).

The basic differential equation of Terzaghi's consolidation theory shown below Eq. (3.1), where c_v denotes the consolidation coefficient $c_v = \frac{k \cdot M}{\gamma_w}$, was derived based on the assumption that an equilibrium condition is maintained between the difference in the amount of water flowing in and out of the elementary volume of the soil and the change in this volume. On the other hand, the change in the elementary volume of the soil is due to the change in the porosity coefficient of the soil, linearly dependent on the increase in stress, related to a constant value of the oedometric compressibility modulus E_{oedo}.

$$\frac{\partial u}{\partial t} = \frac{kE_{\text{oedo}}}{\gamma_w}\frac{\partial^2 u}{\partial z^2} = c_v\frac{\partial^2 u}{\partial z^2} \tag{3.1}$$

Most often, the solution of Terzaghi's equation is presented in the form of the quotient of consolidation settlements s_t mobilized after time t, related to total settlements s_c after the consolidation process, defined as the averaged degree of consolidation $U_{av} = s_t/s_c$. Sivaram and Swamee [23] proposed an empirical formula for U_{av} [%]:

$$\frac{U_{av}}{100} = \frac{(4T_v/\pi)^{0.5}}{\left[1+(4T_v/\pi)^{2.8}\right]^{0,179}} \tag{3.2}$$

where

$$T_v = \frac{c_v t}{H_{dr}^2} \tag{3.3}$$

t – time, H_{dr} – filtration path.

In order to estimate the course of the consolidation settlement process in reality, the Taylor method (square root method) is used:

$$c_v = \frac{T_{90} H_{dr90}^2}{t_{90}} \tag{3.4}$$

where: $T_{90} = 0.848$, t_{90} – period after which $U_{av} = 90\%$ (90% of total consolidation settlement), H_{dr90} – filtration path w t_{90} and Casagrand (logarithmic method):

$$c_v = \frac{T_{50} H_{dr50}^2}{t_{50}} \tag{3.5}$$

where: $T_{50} = 0.197$, t_{50} – period after which $U_{av} = 50\%$ (50% of total consolidation settlement), H_{dr50} – filtration path w t_{50}

Originally, in the one-dimensional consolidation equation, c_v is calculated as the product of the filtration coefficient and oedometric modulus, determined for a given load range, related to the volume weight of water $\frac{kE_{\text{oedo}}}{\gamma_w}$.

The values of c_v determined by one of the methods given above Eq. (3.4) and (3.5) form the basis for calculating the dimensionless time index T_v, needed to estimate the consolidation settlement s_t after time t, with the known (assumed) value of the total settlement mobilized in the consolidation process and assuming that the actual filtration path H_{dr} is known. The curves in Fig. 3–6 show the variation of settlement as a function of the logarithm of time, allowing easy determination of t_{50}, and, ultimately, the determination of the consolidation coefficient c_v.

Figure 7 shows the variation of the consolidation coefficient c_v as a function of load determined assuming $H_{dr} = 0.10$ m.

The dependencies of c_v on load for slab and pile-raft foundations (Fig. 7), supported by the averaged c_v results of Figs. 3–6, indicate two different types of behaviour: stand-alone slab and slab with 4 piles, and slab supported by 9 and 16 piles. The first group is characterized by c_v consolidation coefficients with average values of 571E-8 (slab) and 7.04E-8 m^2/s (slab + 4 piles). The average c_v values for the second group are an order of magnitude lower: 7.73E-9 (slab + 9 piles) and 8.02E-9 m^2/s (slab + 16 piles). Such observations lead to the conclusion that the consolidation of the subsoil of pile-raft foundations, in which the piles are spaced widely (about $r/D \geq 6$), proceeds similarly to the consolidation of the subsoil of direct foundations. The CPRF foundation, with piles in close proximity, behaves differently,

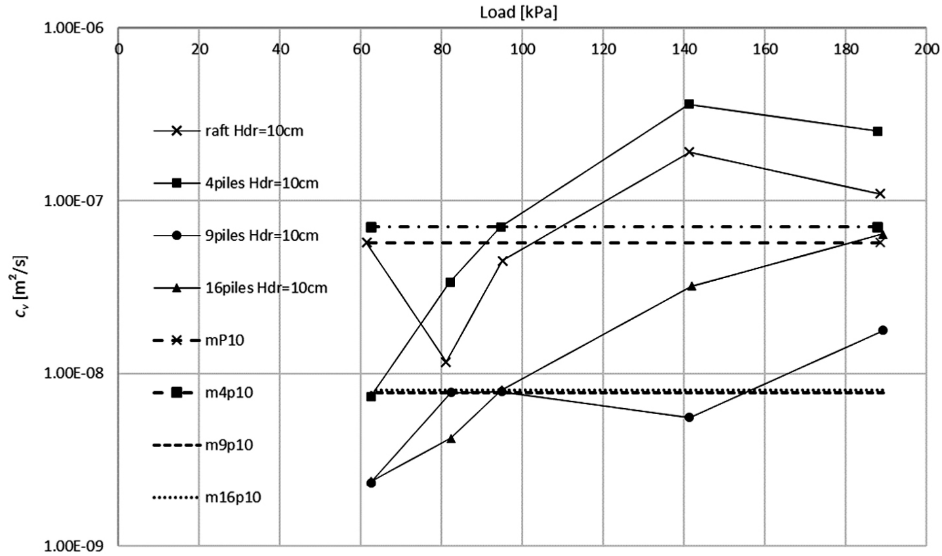


Fig. 7. Values of the c_v coefficient for H_{dr} = 10 cm as a load function; designations: mP10 – the median for the slab, m4p10 – the median for 4 piles, m9p10 – the median for 9 piles and m16p10 – the median for 16 piles

where consolidation is slower (median of c_v < 1E-8 m^2/s) then for subsoil of direct foundation (median of c_v close to 1E-7 m^2/s.

Analysing the last two loading steps (142 and 189 kPa) for the slab and the slab with 4 and 16 piles, the c_v takes similar values, close to 1×10^{-7} m^2/s. Such a conclusion based on the observations of laboratory tests may mean that after exceeding a certain conventional load value, greater than, for example, the value of reconsolidation stress, characteristic of the soil, i.e. when the soil is considered normally consolidated, direct and pile-raft foundations, regardless of the number of piles, settle in time in a similar manner. The value of $c_v = 1 \times 10^{-7}$ m^2/s from own research is close to that obtained by Skarzyńska [24] for gyttja from the vicinity of Janki near Warsaw using the logarithmic method.

As Lambe and Whitman [25] pointed out, determining and selecting the c_v for a specific engineering task is difficult. It should be remembered that the actual settling velocity of a structure's foundation is often two to four times higher than the velocities predicted from the c_v measured using intact samples [26].

The graphs of the compressibility curves from Figures 3–6 are redrawn in Figure 8, where the course of settlement over time for the foundation without piles and the application of piles is shown for 5 selected loading steps. Figure 8 allows us to conclude that the absolute consolidation settlement of pile-raft foundations decreases with an increase in the number of piles (an increase in piles causes the foundation to settle less). Piles reduce consolidation settlement for all (of the selected) load steps. At loads of 94, 142 and 189 kPa, the reduction in consolidation settlement using 16 instead of 9 piles is identical or very similar, which means that using more than 9 piles for the studied foundations in the analysed load range is ineffective.

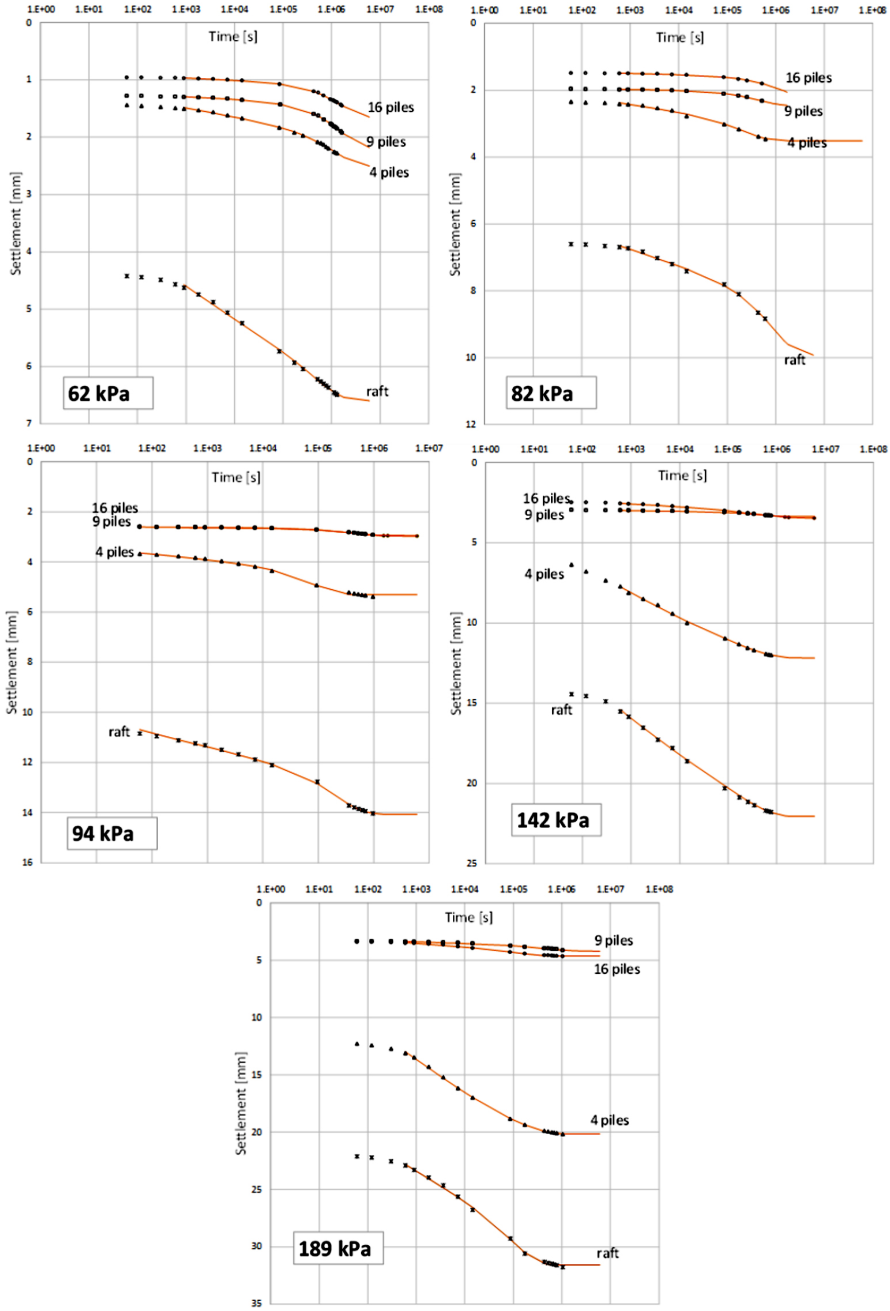


Fig. 8. Compressibility curves for selected load steps, approximated by a Meyer curve

4. Conclusions

The presented methodology for preparing test sites indicated that it is possible to reproduce the actual soil conditions in the laboratory. The model tests and their results show that it is possible to assess the interaction of slab and pile foundations with the soil medium on a laboratory scale.

The dependencies of c_v on load for slab and pile-raft foundations supported by the averaged c_v results indicate two different types of behaviour: stand-alone slab and slab with 4 piles, and slab supported by 9 and 16 piles. Such observations lead to the conclusion that the consolidation of the subsoil of pile-raft foundations, in which the piles are spaced widely (about $r/D \geq 6$), proceeds similarly to the consolidation of the subsoil of direct foundations. The CPRF foundation, with piles in close proximity, behaves differently, where consolidation is slower (median of $c_v <$ 1E-8 m^2/s) then for subsoil of direct foundation (median of c_v close to 1E-7 m^2/s.

The c_v takes similar values analysing the last two loading steps (142 and 189 kPa) for the slab and the slab with 4 and 16 piles. This may mean that after exceeding a certain conventional load value, greater than, for example, the value of reconsolidation stress, characteristic of the soil, i.e. when the soil is considered normally consolidated, direct and pile-raft foundations, regardless of the number of piles, settle in time in a similar manner.

The graphs of the compressibility curves (Fig. 8), where the course of settlement over time for the foundation without consolidation and the application of piles is shown for 5 selected loading steps allow us to conclude that the absolute consolidation settlement of pile-raft foundations decreases with an increase in the number of piles (an increase in piles causes the foundation to settle less). The reduction in consolidation settlement using 16 instead of 9 piles is identical or very similar, which means that using more than 9 piles for the studied foundations in the analyzed load range is ineffective.

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Badania modelowe i ocena ilościowa redukcji osiadań konsolidacyjnych gytii przez różne typy fundamentów palowych

Słowa kluczowe: fundament płytowo-palowy, osiadanie, konsolidacja gytii, konsolidacja podłoża wzmocnionego, krzywa ściśliwości, współczynnik konsolidacji, stopień konsolidacji, redukcja osiadań

Streszczenie:

Fundamenty płytowo-palowe (FPP) są powszechnie stosowane w celu redukcji przemieszczeń pionowych, przy posadowieniu budynków, jak również konstrukcji inżynierskich (obiektów mostowych), w sytuacji kiedy warunek stanu granicznego nośności jest spełniony, ale możliwe jest przekroczenie dopuszczalnych osiadań.

Przy posadowieniu obiektów na gruntach spoistych i organicznych ważnym aspektem staje się czas, po jakim następuje pełna stabilizacja osiadań. Należy pamiętać, że osiadania cał kowite podłoża gruntowego są wynikiem sumy osiadania natychmiastowego (związanego z odkształceniami sprężystymi gruntu), osiadania konsolidacyjnego (powstałego z wyciskania wody z przestrzeni porowej) oraz osiadania wtórnego (rezultat zmian strukturalnych szkieletu gruntowego, nazywanego konsolidacją wtórną lub pełzaniem gruntu). Szybka urbanizacja dużych miast, w tym Warszawy, zmusza inwestorów do budowania na obszarach do tej pory nie branych pod uwagę, do których można zaliczyć tereny paleodoliny rynnowej przecinającej południkowo Warszawę, od Żoliborza do Okęcia, wypełnionych organicznymi osadami węglanowymi, w tym gytią i kredą. Osiadania obiektów budowlanych posadowionych na tego typu gruntach organicznych wynikają przede wszystkim z osiadania gytii, w tym jej osiadania konsolidacyjnego, przypadającego na fazę ich realizacji i eksploatacji. Stąd za bardzo istotną należy uznać analizę długoterminowego osiadania fundamentów, w tym fundamentów płytowo-palowych posadowionych na prekonsolidowanych gruntach organicznych. Jednym z możliwych narzędzi umożliwiających taką analizę i wskazanie ilościowego wpływu pali na redukcję osiadania fundamentów płytowo-palowych są badania modelowe wykonane w skali laboratoryjnej. Stanowią one logiczną alternatywę dla badań wielkoskalowych charakteryzujących się dużymi kosztami oraz trudnościami technicznymi związanymi z zadawaniem dużych obciążeń. Zaproponowany artykułprzedstawia wyniki przeprowadzonych badań modelowych rzeczywistych gruntów organicznych, dzięki którym możliwa jest prognoza osiadań fundamentu płytowo palowego, bazując tylko na parametrach geotechnicznych gytii. Wykonane badania modelowe zakładają usytuowanie FPP – również jego podstawy – w gruntach organicznych, tym samym zarówno płyta jak i pale pracują w ośrodku gruntowym o dużej odkształcalności. Przeprowadzone badania umożliwiły przeprowadzenie analizy parametrów istotnych z punktu widzenia procesu osiadań konsolidacyjnych i wskazanie wytycznych do optymalnego projektowania posadowień przedsięwzięć inwestycyjnych.

Received: 2024-02-28, Revised: 2024-04-09