Centrum Rzeczoznawstwa Budowlanego

Inżynieria Bezpieczeństwa
Obiektów Antropogenicznych 1 (2023) 33-45
https://inzynieriabezpieczenstwa.com.pl/

Inżynieria Bezpieczeństwa Obiektów Antropogenicznych

Anthropogenic factors as determinants of deformation and damage of a bridge structure founded on clayed ground – a case study

Leopold KRUSZKA[*1], Maciej KLOSAK[2], Aleksander KRAVCOV[3], Jiri STOLLER[4], Izabela SKRZYPCZAK[5]

[1] Military University of Technology, Faculty of Civil Engineering and Geodesy, ul. gen. Sylwestra Kaliskiego 2, 00-908 Warsaw, Poland
[2] Universiapolis, Technical University of Agadir, Technopole d'Agadir, Qr Tilila, 80000 Agadir, Morocco
[3 4] University of Defence, Kounicova 65, 66210 Brno, Czech Republic
[5] Rzeszow University of Technology, Faculty of Civil and Environmental Engineering and Architecture, Powstańców Warszawy 12 Av., 35-959 Rzeszow, Poland

Abstract

Bridges are often built in complex geological-engineering conditions, on difficult soils, landslide areas or within the range of negative mining influences. These objects are classified as geotechnical category II or III and a ground investigation documentation or geotechnical design should be carried out for them. The correct investigation of the soil, design, and execution in accordance with technical knowledge, the art of construction and the Building Code are a guarantee of the safety of the facility. However, very often it is the complexity of the geological structure, concentrated vertical and horizontal loads transferred by the supports to the soil that make it necessary to use deep foundations, special methods of soil reinforcement and imposing a special technological regime during the execution of construction works. The paper pays attention to anthropogenic and natural factors, as well as those that are the consequence of inappropriate human actions at the design or execution stage. Particular attention is paid to errors in the soil analysis and execution of construction works. An example of such errors analysis is presented in relation to the implementation of a bridge structure.

Keywords: bridge structure, safety, geotechnical investigation, execution, errors

1 Introduction

In the case of linear investments, it is possible during the stage of basic geotechnical investigations to select the course of the road and location of bridges and accompanying facilities (flyovers, bridges, culverts, noise barriers), select technical solutions of the structure and assess the cost of the investment or determine geotechnical parameters of the soil (Rybak and Stilger-Szydło, 2009, Obolewicz and Baryłka, 2021, Gosk, 2022, Sobczyk, K. et al., 2022). The legal basis for soil investigations in Poland derives from the following regulations:

1) Act of 9 June 2011 - Geological and Mining Law (Journal of Laws 2016, item 1131),

* **Corresponding author:** E-mail address: (leopold.kruszka@wat.edu.pl) Leopold Kruszka

https://dx.doi.org/10.37105/iboa.166
Received 15 March 2023
Available online 27 March 2023
ISSN 2450-1859, eISSN 2450-8721
Published by Centrum Rzeczoznawstwa Budowlanego

2) Regulation of the Minister of Environment of 8 May 2014 on hydrogeological and geological-engineering documentation (Journal of Laws of 2014, item 596, as amended),
3) Act of 7 July 1994 - Construction Law (Journal of Laws of 2016, item 290),
4) Regulation of the Minister of Transport, Construction and Maritime Economy of 25 April 2012 on establishing geotechnical conditions for the foundation of buildings (Journal of Laws of 2016, item 672),
5) Regulation of the Minister of Transport, Construction and Maritime Economy of 25 April 2012 on the detailed scope and form of the construction design (Journal of Laws of 2012, item 462).

In the case of Poland, geological and engineering surveys are performed in accordance with the Geological and Mining Law (Journal of Laws 2016, item 1131), while geotechnical surveys are performed on the basis of the provisions of the Construction Law (Journal of Laws 2016, item 290). The Construction Law (Journal of Laws 2016, item, 290) indicates in Article 34(3)(4) that the construction project should include, as required, the results of geological-engineering studies and a list of geotechnical foundation conditions of construction objects. These requirements are defined in the Regulation of the Minister of Transport, Construction and Maritime Economy of 25 April 2012 (Journal of Laws 2016, item 672). On the other hand, the general guidelines for soil investigations are formulated in Eurocode 7: PN-EN 1997-1 (PN-EN 1997-1:2008) and PN-EN 1997-2 (PN-EN 1997-2:2009) and regulate the whole issue of geotechnical design. It distinguishes between two scopes of activities:

1) geotechnical studies including planning of the analysis, definition of the geological model, field and laboratory studies and documentation of soil studies,
2) design including interpretation of test results, determination of geotechnical parameters and coefficients (geotechnical model), geotechnical and structural design and specifications of works, control and supervision program.

According to the provisions of the Ordinance (Journal of Laws 2016, item 672) the scope of the study on soil performances necessary for the assessment of geotechnical foundation conditions results directly from the geotechnical category of the construction object.

The geotechnical category is determined by two factors: the complexity of the soil conditions and the type of building. The geotechnical category of an object is determined by the designer and may be changed when geotechnical conditions differ from those assumed in the study. The geotechnical category should be verified at each stage of the project, from the concept stage to the design and construction stage of the facility.

For bridges, the basic surveys are exploratory borings, the number and location of which in simple and complex geological conditions depend on the width of the bridge and the span (Stilger-Szydło, 2005; Rybak and Stilger-Szydło, 2009). In the case of a complex geological structure, the grid of primary drillings is thickened, or auxiliary drillings are envisaged. The required depth of exploratory borings depends on the type of structure and the value of loads transferred to the soil, as well as on the soil conditions (Rybak and Stilger-Szydło, 2009):

- direct foundations of bridges - the depth of the base holes should not be less than 5.0 m below the intended bottom of the foundation (indicatively 6.0÷8.0 m below ground level); it is possible to make the holes shallower, but at least 2.0 m below the floor of the bearing layer; the auxiliary boreholes are brought to a depth of 1.0÷2.0 m below the bottom of the soil with low bearing capacity,

- deep foundations for bridges - the required depth of boreholes may be assumed equal to the depth of piles, increased by at least 3.0 m (indicatively 10.0÷25.0 m below ground level), and wells or caissons - by 5.0 m (15.0÷30.0 m from ground surface). However, the boreholes should be sunk at least 6.0 m into the bearing soil layer. These depths may be reduced if boreholes are carried out in homogeneous layers of high thickness (e.g., Pliocene clays, Cracovian clays, etc.),

- retaining structures - the depth of the boreholes should exceed the possible slip surface and reach a depth below the bottom of the foundation at least equal to the height of the wall or ground fault.

The results of the tests are used when calculating the ultimate bearing capacity and settlement of direct foundations as well as the bearing capacity and lateral displacements of pile foundations (Instruction ITB 231/1980; Tejchman and Krasiński, 1992; Rybak and Stilger-Szydło, 2009).

The scope of the analysis is regulated by the Regulation of the Ministry of the Interior and Administration (Journal of Laws 2016, item 672), introducing the concept of geotechnical category. For bridges, this is usually category II or III. On the other hand, the scope of soil study in relation to pile foundations is determined by the standard PN-83/B-02482, which indicates the requirements concerning the penetration of the pile base into load-bearing soils, the penetration of the pile base into the layer in which the load-bearing capacity of the base has been determined, and the minimum distances from the roof and bottom of the layer in which the pile is terminated. Determining the full range of analysis is not possible without selecting the piling technology and calculating the pile length.

Main errors in geotechnical documentation result from improperly programmed and executed studies, as well as careless execution of foundation works and a lack of forecasting of changes in these parameters over time. Improperly programmed surveys may lead to (Rybak and Stilger-Szydło, 2009):

- reducing the scope of field work to a minimum, which results in overinterpretation of the information obtained and geotechnical errors/overlooks,

- drilling an increased number of shallow boreholes, e.g., for pile foundations,

- poor planning of boreholes, e.g., omission of the area outside the contour of the foundation,

- omission in the surveys of non-bearing soils without specifying their geotechnical parameters.

Errors in the execution of field analyses include (Rybak and Stilger-Szydło, 2009):

- inappropriate manner of making test holes, drilling without casing, which gives a falsified picture of water relations and condition of cohesive soils,

- sticking to the contractually agreed scope of work, which limits the possibility to determine the extent of weak soils,

- completing the boreholes in non-bearing soils, which makes the boreholes unsuitable for designing, or leads to significant oversizing of foundation elements,

- completing boreholes at depths which allow for calculating the load capacity of a single pile, but do not allow for calculating the settlements of a group of piles.

Errors occurring at the stage of laboratory tests include:

- performing laboratory tests that do not comply with the recommendations of the standards,

- omission of shrinkage limit tests in semi-hardened soils, which makes it impossible to properly design according to the recommendations of the standard PN-83/B-02482,

- omitting the determination of the characteristics of non-bearing soils (embankments, silts, peats), which makes it impossible, for example, to properly design their reinforcement,

- failure to use advanced testing methods,

- inadequate sampling and limiting the number of samples for testing to a minimum.

It should be emphasized that the implementation of a construction project that interferes with the soil in a temporary or permanent manner usually results in the disturbance of local water relations, e.g., the disturbance of the moisture balance state in expansive soils. During construction excavations the state of stress in the soil is altered, and the local directions of rainwater runoff are also altered. Sealing the ground surface on the one hand reduces the inflow of water into the ground, but on the other hand contributes to reducing transpiration through the previously spread vegetation (Gorączko, 2017).

The following is an example of a situation in which improperly implemented geotechnical, and foundation works led to significant complications in the execution of a bridge structure.

2 Description of the object of research and analysis

The object of analysis is a monolithic, six-span bridge structure with a beam-and-slab cross-section (Fig. 1). The cross-section for the left-hand carriageway (direction of traffic X-Y) is designed as a three-beam one, while the right-hand carriageway (direction of traffic Y-X) is designed as a two-beam cross-section, all made of prestressed concrete with a reinforced concrete deck slab. Above the outermost supports, monolithic crossbeams are extended under the deck slab supports. Above the intermediate supports, monolithic crossbeams only occur between the main beams. The height of beams is 1.30 m. The deck slab between the beams is 0.30 m thick, increased locally to 0.45 m when fixed to the girders. On both sides of the structure, the podium cantilevers of the deck slab are designed with an overhang of approximately 2.60 m. The deck slab along the length of the cantilevers has a variable thickness of 0.20 m to 0.45 m. The superstructure is supported by hinged and pinned bearings.

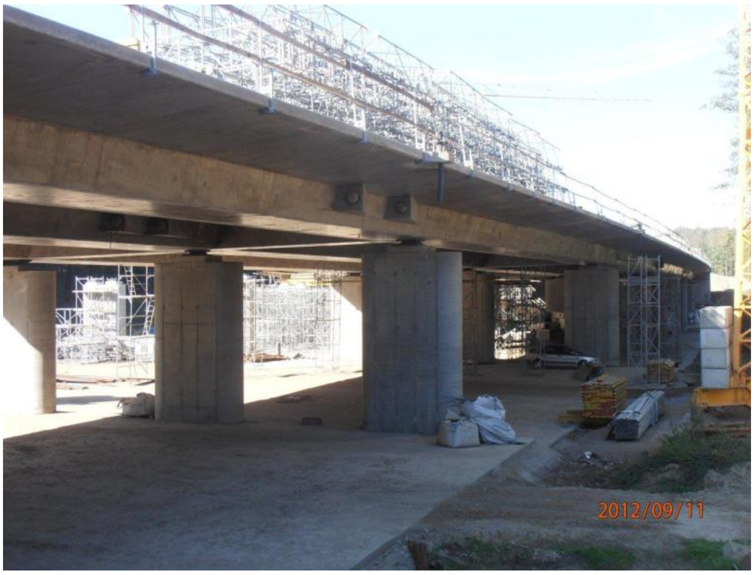

Figure 1. *General view from below of the superstructure of the left-hand carriageway and of the pillars*

The abutments are designed and realized as solid walls with wings in the form of standing sidewalls on the embankment side, founded directly on footings. The sidewalls on one side are dilatated from the body of the abutment, while on the other side they are designed as side walls monolithically connected to the abutment. The intermediate supports are designed as pillars with 3 (left-hand carriageway) and 2 columns (right-hand carriageway) respectively, each of the oval cross-section, founded shallow in a layer of silty clay (Fig. 2).

Figure 2. *General view of the oval support columns of the left-hand carriageway of the bridge structure*

The technical parameters of the structure in a horizontal curve are the following:

- Angle of haunch: variable, an arc with radius R = 2000 m,
- Spans: L_t = 25.0 + 4 x 33.0 + 25.0 m,
- Length of superstructure: L_U = 183.0 m,
- Overall width: B_C = 42.22 m,
- Height of superstructure: h = 1.30 m,
- Thickness of deck slab: t = 0.20 to 0.45 m,
- Variable load: A according to PN-85/S-10030, special vehicle STANAG 2021 class 150 and class 100.

The cross-section of the structure (left carriageway) is adapted to the normal cross-section of the road and consists of the following:

- Reinforced concrete cap widening under the lamp post 0.36 m,
- Elevated roadside shoulder (parapet, band, curb, with light screen and barrier) 2.985 m,
- Roadway 3.50 + 3 x 3.75 = 14.75 m,
- Elevated shoulder (band, curb, parapet) 5.04 m,
- Total of 23.135 m (the width of the deck slab (without parapet planks is 23.055 m.

During the execution of the structure (after dismantling of the deck formwork and scaffolding), the settlement of intermediate supports occurred, significantly exceeding the settlements determined at the design stage by calculations, with a sudden increase in settlement occurring approximately 2.5 months after the supports were "freed". Hence, the primary objective of this paper is to determine the probable causes of this sudden increase in settlement of the supports caused by anthropogenic factors.

1.1 Soil and water conditions in the area of the foundation of the supports

The description of the subsoil in the area of the object's foundation: 3 series of soil are developed. First, a series of sandy formations lie directly beneath the near-surface soil layer with a thickness of approx. 0.1-0.5 m, to a maximum depth of 3.9 m. Lithologically, these are fine sands and silty sands. The sands are in a loose to medium dense state. In places, cohesive fluvial deposits 0.5-1.6 m thick occur under the sandy layers and locally directly under the ground level. Lithologically, they are formed as silty clay, compact silty clay, and sandy silt. Under the Quaternary formations lie layers of older origin, these are Miocene clays. The top of this layer was found at the depth of 0.2-3.9 m below ground level, where the top of the Cracovian clays (Miocene-Neogene) was reached during drilling.

The layer is formed as silty clay with frequent interbedding of sandy dust and locally silty sand. This layer was found to a drilling depth of 30.0 m below ground level. The clays are in a hard-plastic and semi-hard-plastic state, while locally in the ceiling they are in a plastic state. The top of the semi-hardened clays was found at a depth of 13.5-17.9 m. The studied zone is the top part of a several-hundred-meter-thick complex of Miocene deep-sea formations, developed generally in the form of clays, siltstones, mudstones and claystones, described in the literature as "Cracovian silts". In the identified ground conditions, it is recommended that the structure be founded indirectly on piles, the footings of which can be founded in the layer of semi-consolidated Cracovian clays (IIIa). The Miocene clays are swelling soils with swelling pressures of 150-300 kPa. The clays increase in volume on contact with water.

The primary water-bearing level was found in the sandy layers. This level is characterized by a free water table. The water table stabilized at different depths depending on the drilling date. The aquifer is recharged by rainwater and snowmelt, which causes periodic fluctuations in the groundwater table.

2 Adopted solutions of founding of the object supports

2.1 Foundation solutions in the Architectural and Construction Project

The design of the foundations of the structure at the stage of the Architectural and Building Project considered the recommendations given in the Construction Law regarding intermediate foundations. The outermost supports and pillars were founded on suspended, large diameter reinforced concrete piles, 1500 mm in diameter, with the footing in a silty clay layer (layer IIIb). To increase the load capacity of the piles, as the soil surrounding the piles is hard-plastic cohesive soil, the pile bases were designed to be widened to diameters depending on the external load per pile. The number of piles and their basic geometrical parameters are summarized in Table 1.

Position of the support	**Number of piles [piece]**	**Diameter [mm]**	**Widening of the pile base [mm]**	**Length of the pile [m]**
Abutment body and sidewalls of left-hand carriageway	7	1 500	2 200	10,0
Supports - pillars of the left-hand carriageway	6	1 500	2 700	10,0
Abutment body of left-hand carriageway	9	1 500	2 200	10,0
Abutment sidewalls of left-hand carriageway abutment	4	1 500	2 200	10,0
Abutment body and sidewalls of the right carriageway	6	1 500	2 200	10,0
Supports - pillars of the right-hand carriageway	5	1 500	2 700	10,0
Abutment body of right-hand carriageway	7	1 500	2 200	10,0
Abutment sidewalls of the right-hand carriageway	4	1 500	2 200	10,0

Table 1. *Specification of designed piles*

The quantitative and qualitative selection of piles was the result of static-strength calculations of the structure supports and determining their ultimate and serviceability limit states, as well as the degree of utilization of their load-bearing capacity. Selected results for the most stressed piles for the outermost and middle supports are shown in Table 2.

Position of the pile	**Characteristic loads [kN]**	**Design loads [kN]**	**Bearing capacity of the pile in a group [kN]**	**Bearing capacity utilization rate [-]**
Pile under the abutment body of the left-hand carriageway	3 108,6	3 944,5	5 523,0	0,80
Pile under support - pillar of the left-hand carriageway	5 460,3	7 021,8	8 315,4	0,88
Pile under support - pillar of the left-hand carriageway	5 109,3	6 577,3	8 315,4	0,94

Table 2. *Bearing capacity of selected piles*

The serviceability limit state, i.e. pile settlement, was determined for single piles, and the settlement values for the piles listed in Table 2 ranged from: s = 29 mm to s = 46 mm and were lower than the allowable settlement value for a single pile. The acceptable settlement value for a single pile s_{dop} = 86 mm was determined according to the Technical Guidelines for the Design of Large Diameter Piles in Bridge Structures (IBDiM Guidelines 1993).

2.2 Foundation solutions in the Architectural and Construction Project

As a result of the optimization process of the design solutions, it was decided, in consultation with the construction supervision, to change the foundation of the building from an indirect foundation to a direct foundation on strip (slabs) footings. The bottom foundation level (of the pillars) of which the width is B = 4.20 m, and the length is approx. 21.20 m for the left-hand carriageway was established in the topsoil layer of silty clays with admixtures of sandy dust and silty sands. These clays are marked with the number III and the letters d and c, in a hard-plastic state with a general degree of plasticity I_L = 0.22, volumetric density = 18.8 kN/m^3, internal friction angle = 100, cohesion

c_U = 48 kPa and edometric primary compressibility modulus M_0 = 23 MPa for layer IIId (supports numbers 2, 3, 4 and 7) and with degree I_L = 0.10, volumetric density = 19.0 kN/m^3, internal friction angle = 110, cohesion c_U = 54 kPa and edometric modulus of primary compressibility M_0 = 30 MPa for the layer IIIc (supports numbers 5, 6), except for the abutment no. 1, which was founded in geotechnical layer I, i.e. in fine sand with doming. of silty sand and sandy dust in the state loose/medium-compacted, with the degree of compaction I_D = 0.33, volumetric density = 19.5 kN/m3, angle of internal friction = 290, and edometric primary compressibility modulus M_0 = 44 MPa.

The geometric dimensions of the footings were established on the basis of static-strength calculations for all the supports, for which the ultimate and serviceability limit states were checked, assuming the soil parameters given above, while the loads on the individual supports were the result of their respective combinations in each phase of their operation and taking into account the standard load combinations resulting from the design codes PN-85/S-10030, PN-81/B-03020 and PN-83/B-03010. Table 3 gives the basic results of the calculations for the ultimate limit state for the individual supports of the left-hand carriageway for the basic loading system.

Number of the support	**Vertical characteristic loads [kN]**	**Vertical design Loads [kN]**	**Design ultimate resistance of the subsoil [kN]**	**Bearing capacity utilization rate [-]**
Abutment No. 1 of the left carriageway (from X side)	17 058,0	21 495,0	28 798,4	0,75
Abutment No. 2 - pillar of the left-hand carriageway	25 317,7	31 674,2	41 975,8	0,75
Support No. 3 - pillar of the left-hand carriageway	27 136,9	35 247,6	42 386,0	0,83
Support No. 4 - pillar of the left-hand carriageway	27 299,5	35 370,3	38 648,6	0,92
Support No. 5 - pillar of the left-hand carriageway	28 377,6	36 531,3	40 097,7	0,89
Support No. 6 - pillar of the left-hand carriageway	27 716,5	35 414,4	41 200,1	0,84
Abutment No. 7 of the left carriageway (from Y side)	24 410,4	30 317,8	40 900,7	0,74

Table 3. *Load-bearing capacity of the footings (slabs) at the supports*

Due to the specific nature of the soil at the level of the strip footing foundation and below this level, the serviceability limit state was determined and strip footing settlements were calculated with the use of the edometric analogue method, adopting the calculation methodology in accordance with PN-81/B-03020; at the same time, an indirect coefficient was adopted (in relation to the standard PN-81/B-03020) which took into account the degree of subsoil deformation equal to 0.25 due to the specific properties of silty clay (possibility of swelling) and possible periodic watering during the execution of foundation works. The settlements (with the same soil parameters) from the self-weight of the bridge (without equipment and payload) and from the supports were also calculated, as well as the settlements from the self-weight of the supports only (without backfill soils on the side of the road embankment). The results of the settlement calculations for the individual footings (slabs) of the supports are summarised in Table 4.

Number of the support	**Total settlement of the footing [mm]**	**Settlement of the footing due to permanent loads [mm]**	**Settlement of the footing due to self-weigh of supports [mm]**
Abutment No. 1 of the left carriageway (from X side)	13,9	11,1	2,0
Abutment No. 2 - pillar of the left-hand carriageway	28,0	18,6	2,5
Support No. 3 - pillar of the left-hand carriageway	27,8	18,7	1,7
Support No. 4 - pillar of the left-hand carriageway	30,6	17,5	1,9
Support No. 5 - pillar of the left-hand carriageway	25,7	17,3	1,6
Support No. 6 - pillar of the left-hand carriageway	25,9	17,2	2,3
Abutment No. 7 of the left carriageway (from Y side)	21,2	17,1	2,1

Table 4. *Settlement of footings of supports at different stages of building erection*

3 Comments on the adopted design solutions

The results of calculations of limit states of footings for the supports of the left-hand carriageway of the facility confirm the validity of the optimized foundation changes. The adopted levels of direct foundations, the dimensions of footings are calculatively correct in terms of satisfying the ultimate limit state $N_r \leq m\cdot Q_{fNB}$ and the degree of utilization of the subsoil bearing capacity (design ultimate resistance) $n = N_r/m\cdot Q_{fNB}$ was not exceeded (reserves can be seen in the majority of supports, which are the result of standardization of support dimensions for the flyover pillars). This provided a guarantee of safe erection and subsequent use of the structure, provided the design soil parameters were maintained. In the case of serviceability limit state, standards and other regulations for the design of bridges and engineering structures do not specify the amount of allowable settlement of directly founded supports, nor do they describe the relative settlement conditions of such supports.

However, using the Russian regulations (analogy to the settlements of indirectly founded supports) (SNiP, 1994), the allowable settlement of supports of continuous multi-span structures is expressed by the formula $S_{dop} = 15\cdot(L_T)^{\wedge 1/2}$, which gives a settlement magnitude for the support of the structure with the shorter span L_T = 25,0 m equal to 75 mm, and 43.1 mm for the longer span of L_T = 33,0 m. The allowable difference in settlement of intermediate supports $\Delta S_{dop} = 7{,}5\cdot(L_T)^{\wedge 1/2}$ is 37.5mm for the shorter span and 43.1 mm for the longer span. On the other hand, US regulations for such structures propose the relative difference in settlement to be calculated as $\Delta S/L_T = 0{,}004$, resulting in ΔS_{dop} = 100 mm and ΔS_{dop} = 132 mm, respectively in the case studied.

As far as the technological provisions concerning the backfilling of the object excavations are concerned, it was proposed that the excavations in the area of the abutments, as indicated in the design drawings, should be made of natural or artificial material of varying grain size and with the following parameters: volume density □□19,0 kN/m^3, angle of internal friction φ □ 34º, degree of compaction $I_S \geq 1{,}0$. This provision did not directly refer to the backfilling of the pillar excavations. Such parameters of the backfill soil were also included in the technological project, the medium sand, compacted in layers to the appropriate I_S, was used as the backfill material filling the space between the designed sheet piling (for the implementation of supports 3 and 4) up to the height of the ground level. Also, the Technical Specifications with the symbols M.11.01.00 „Excavation works” and M.11.01.04 „Backfilling of excavations with compaction” contains statements regarding the properties of the backfill soil: "for backfilling of excavations, unless otherwise specified in the design documentation, previously excavated soil, unfrozen and without impurities such as parts of plants, humus, peat, waste construction materials, etc., corresponding to the requirements of the PN-B-02205 standard (PN-BS-02205:1998) may be used.

It should be concluded that the design documentation (including technological project) did not properly refer to the risks in the form of a high groundwater level in the layers of sandy overburden over clays, posing a threat to their irrigation, all the more so as the sand-filled near-field excavations of the supports (in accordance with the design requirements) are a kind of artificial depressions, filling with groundwater when its level is stabilized after the foundation works of the supports have been performed. Therefore, the lack of provisions for making foundation backfill from impermeable soil or the necessary suggestion of making horizontal cofferdams (membranes) from such soil (e.g. clay) to prevent the soil from becoming wet (soaked) at the foundation level should be considered an inappropriate approach.

On the basis of the inventory carried out, it was found that there was a displacement at the junction of the wall of the left and right carriageway support bodies as shown in Figure 3.

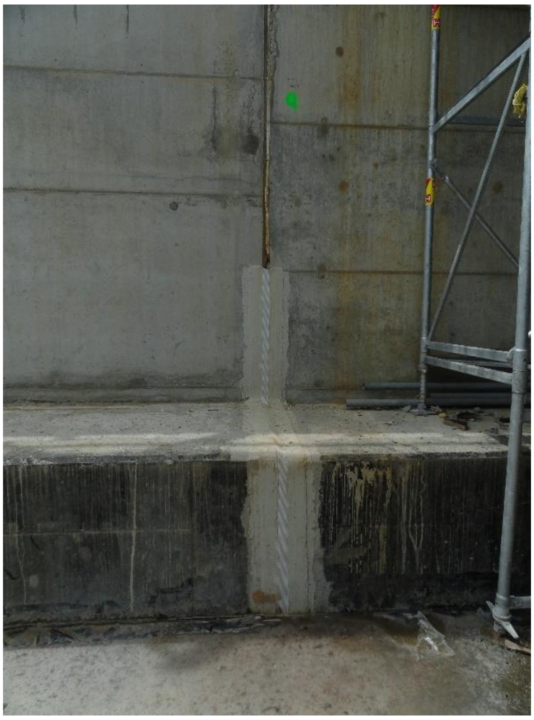

Figure 3. *Oblique deformation of the tape crossing the expansion joint (dilatation) of abutment P1*

Moreover, a displacement (outwards - approx. 40 to 50 mm at the top of the wing wall) of the wall of the standing wing, founded on a separate foundation, from the side wall of the body of the P7 abutment was observed, which proves that the settlement is uneven with respect to the part of the outermost supports for both roadways and wings. The concrete working platform on both sides of the P4 abutment was also observed to crack, at a distance of approx. 4.0 m each, and there was also visible collapse of the platform in the line of the abutment columns, which resulted in the formation of a small local sinkhole, filled with rainwater coming, among other things, from the platform's drainage facilities not connected properly to the collector (Fig. 4). On the side of the platform, significant cavities (caverns) in the ground underneath were also detected, which may be indicative of soil washing out during heavy rainfall and may also be indicative of high groundwater run-off underneath the platform (Fig. 5).

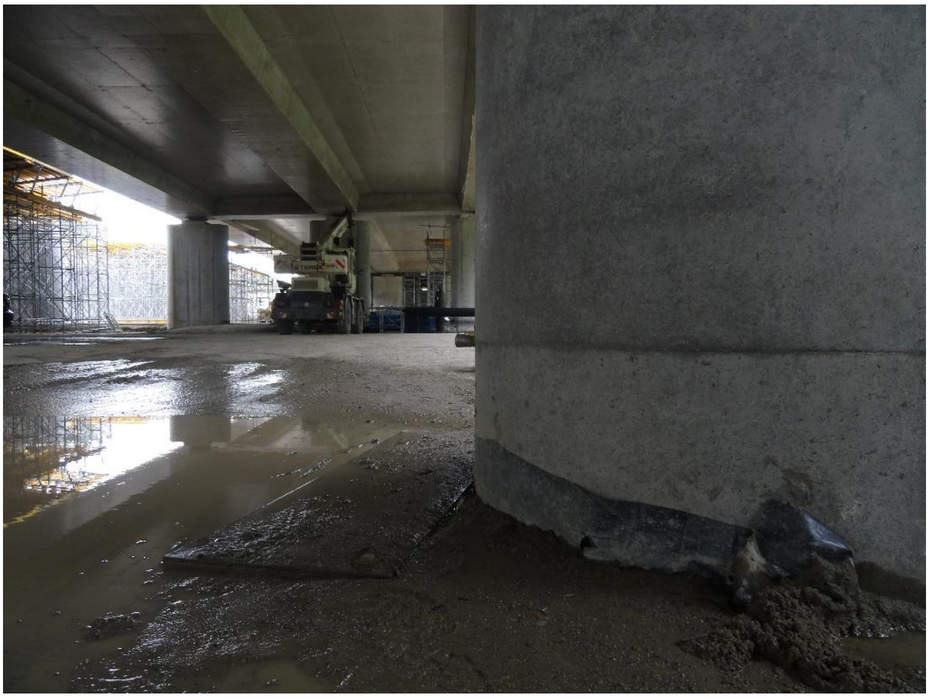

Figure 4. *Deformation and local depression of the working platform at support P4*

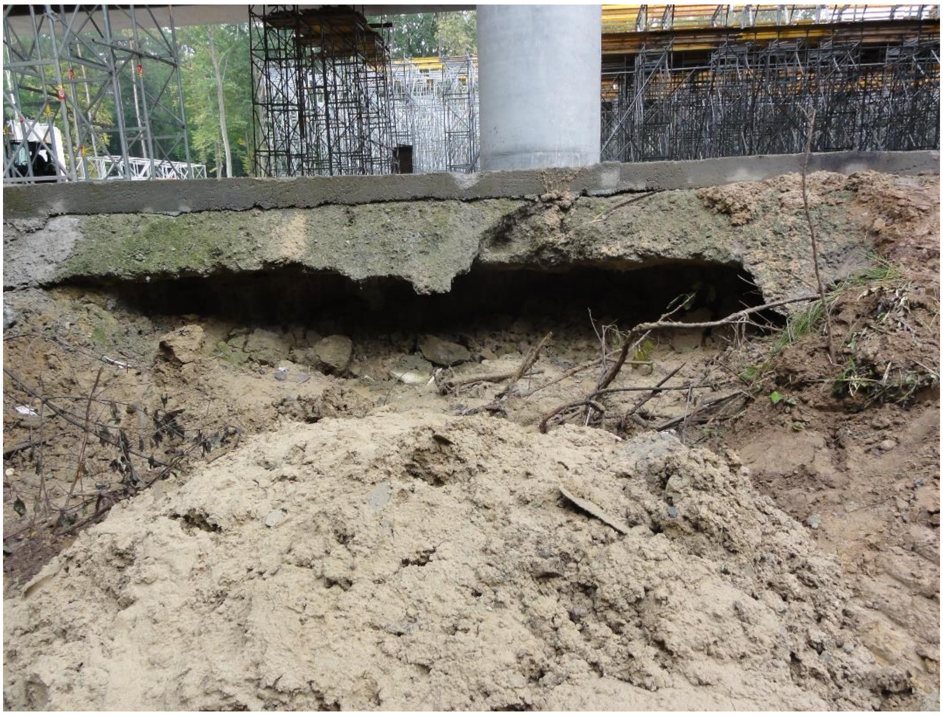

Figure 5. *Caverns under the working platform at support P4*

Concerning the deformation of the left-hand carriageway deck, which could be caused by increased settlement of the supports, they are minimal. The line of the parapet of the concrete slab of the bridge deck is straight, with no visible deformation, and the line of the bottoms of the main beams also appears straight. Possible deformations of the left-hand carriageway deck required accurate geodetic measurements including the bottom level of the structure. A careful observation of the bridge deck (both the surface and the underside of the slab and the main beams) did not reveal any scratches (apart from superficial shrinkage cracks on the surface of the slab from above) resulting from significantly increased settlement of the supports.

4 Analysis of the causes of increased settlement

There may be various reasons for higher than calculated settlements of supports. They may be the result of an inaccurately determined ground conditions (silty clay) indicated in the documentation. This could signify that the subsoil beneath the footings was already much weaker at the time of foundation works (insufficient geological reconnaissance), or that there was a change in the condition of the soil during the works carried out during the erection of the building, or that the two could coincide. Documentary information on the construction site (Construction Logbook) shows that the object excavations were subject to geological acceptance and the type and condition of the soil were confirmed in accordance with the design codes.

Thus, there must have been a deterioration of the geotechnical parameters of the subsoil at a later time, i.e. during the execution of the works. Another reason for the construction of pillars P3 and P4 was the installation of a steel sheet piling, which was necessary due to the high groundwater table and the depth of the foundation excavations for these pillars. During the pulling in or pushing out of the sheet piling ground destruction may have occurred, not only around the foundations, but also underneath them.

As these soils are very sensitive to water (due to the overlapping of silty sands and dust), even slight wetting (soaking) could have caused significant changes in the condition of the soil and thus, a drastic deterioration in its geotechnical parameters (including compressibility and deformation moduli). In addition, after the foundation trenches were backfilled with sandy soil (to a level allowing to build the platforms), there was a permanent state of watering of these soils with a groundwater column of D_{MIN} height which could also have caused a state of soaking of the near-surface clay layer. There could have been other reasons for the deterioration of the soil at the foundation level - and the consequent increased settlement of the supports, but the abovementioned ones should be considered highly probable and the most important.

5 Conclusions

An inadequately design program, under-designed geotechnical documentation, under-designed structural, architectural, construction and execution projects, or execution errors during the implementation of bridge foundations can lead to the following:

- hydration of the layers or loosening of the soil and changes in the physical and mechanical parameters of the soil,

- failure to achieve the load-bearing capacity or serviceability of the designed foundations.

In the presented case study, as a result of wide-ranging anthropogenic errors both at the stage of design and execution of the bridge structure, the following was concluded:

- increased settlement of supports (in relation to analytically determined settlements) exceeds the values considered acceptable and does not guarantee safe, compatible use of the structure.

- the analysis of static calculations proved that with the preservation of the observed clay soil parameters at the foundation level, the adopted direct foundation was rationally and technically justified and there was no risk of exceeding the ultimate and serviceability limit states, i.e., increased settlement of supports,

- the phenomenon of increased settlement is the result of several reasons having their source in both design and execution stages, e.g., the lack of unambiguous solutions for "separating" the soil at the foundation level from the influence of groundwater, the use of technology deteriorating the properties of the soil (sheet piling).

It should be emphasized that the necessity to carry out additional geotechnical control tests, before and during construction, is very often treated by investors as an attempt of the general contractor to create additional costs or to justify a delay in the design or construction cycle, this is why it is often refused.

References

1. Rybak, J., Stilger-Szydło, E. (2009). Rozpoznanie podłoża gruntowego przy posadowieniach obiektów infrastruktury transportu lądowego. Górnictwo i Geoinżynieria, 33(1), pp. 531-544.
2. Ustawa z dnia 9 czerwca 2011 r. Prawo geologiczne i górnicze (Dz. U. z 2016 r., poz. 1131).
3. Rozporządzenie Ministra Środowiska z dnia 8 maja 2014 r. w sprawie dokumentacji hydrogeologicznej i dokumentacji geologiczno-inżynierskiej (Dz. U. z 2014 r., poz. 596 z późn. zm.).
4. Ustawa z dnia 7 lipca 1994 r. Prawo budowlane (Dz. U. z 2016 r. poz., 290).
5. Rozporządzenie Ministra Transportu, Budownictwa i Gospodarki Morskiej z dnia 25 kwietnia 2012 r. w sprawie ustalania geotechnicznych warunków posadawiania obiektów budowlanych (Dz. U. z 2016 r., poz. 672).
6. Rozporządzenie Ministra Transportu, Budownictwa i Gospodarki Morskiej z 25 kwietnia 2012 r. w sprawie szczegółowego zakresu i formy projektu budowlanego (Dz. U. z 2012 r., poz. 462). (Dz. U. z 2016 r., poz. 672). (Dz. U. z 2012 r., poz. 462).
7. PN-EN 1997-1:2008 Eurokod 7 – Projektowanie geotechniczne – Część 1: Zasady ogólne.
8. PN-EN 1997-2:2009 Eurokod 7 – Projektowanie geotechniczne – Część 2: Rozpoznanie i badanie podłoża gruntowego.
9. Stilger-Szydło, E. (2005). *Posadowienia budowli infrastruktury transportu lądowego. Teoria - Projektowanie - Realizacja.* Wrocław. Dolnośląskie Wydawnictwo Edukacyjne, 2005.
10. Tejchman, A., Krasiński, A. (1992). *Zastosowanie presjometru w badaniach gruntu i projektowaniu fundamentów*. Inżynieria Morska i Geotechnika. 4, pp. 163–170.
11. Wytyczne badań presjometrycznych. *Instrukcja ITB 231/1980*. (1980). Warszawa. ITB
12. PN-83/B-02482 Fundamenty budowli. Nośność pali i fundamentów palowych.
13. Gorączko, A. (2017). *Fundamenty na gruntach ekspansywnych*. Inżynier budownictwa. Available online https://inzynierbudownictwa.pl/fundamenty-na-gruntach-ekspansywnych/, access date 15.01.2023.
14. Wytyczne techniczne projektowania pali wielkośrednicowych w obiektach mostowych. *Wytyczne IBDiM 1993.* Warszawa. IBDiM.
15. PN-85/S-10030 Obiekty mostowe. Obciążenia.
16. PN-81/B-03020 Grunty budowlane. Posadowienie bezpośrednie budowli. Obliczenia statyczne i projektowanie.
17. PN-83/B-03010 Ściany oporowe. Obliczenia statyczne i projektowanie.
18. PN-BS-02205: 1998. Drogi samochodowe - Roboty ziemne - Wymagania i badania
19. PN-91/S-10042 Obiekty mostowe. Konstrukcje betonowe, żelbetowe i sprężone. Projektowanie.
20. PN-EN 206:2014-04 Beton -- Wymagania, właściwości, produkcja i zgodność.
21. PN-82/B-02001 Obciążenia w obliczeniach statycznych. Obciążenia stałe i zmienne.
22. General department of standardization, technical regulations, and certifications, 1994, SNiP 10.01.94, System of normative documents in construction.

23. Gosk, F. (2022). The use of cohesive soils improved with lime as an example of circular economy in earthworks. *Inżynieria Bezpieczeństwa Obiektów Antropogenicznych, 3*, pp-pp. 10-20, DOI: https://dx.doi.org/10.37105/iboa.146
24. Sobczyk, K. et al. (2022). Preparation of the non-cohesive soil sample and calibration of the pneumatical launcher in the dynamic soil test SHPB. *Inżynieria Bezpieczeństwa Obiektów Antropogenicznych, 1*, pp-pp. 16-27, DOI: https://dx.doi.org/10.37105/iboa. 129
25. Obolewicz J., Baryłka A. (2021). Life cycle engineering of a construction object. *Inżynieria Bezpieczeństwa Obiektów Antropogenicznych, 3*, pp-pp. 11-20, DOI: https://dx.doi.org/10.37105/iboa.115