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# Complex foundation analysis of network arch bridge and adjacent embankments founded on grouted columns

## Analyse d'un système de fondation complexe d'un pont en arc suspendu et de remblai fondé sur des colonnes de mortiers

O. Mitrosz
*Technical Office, Keller Polska Sp. z o.o., Gdansk, Poland*

T. Skóra, R. Buca
*Technical Office, Keller Polska Sp. z o.o., Gdansk, Poland*

**ABSTRACT:** The presented 120 m span arch bridge, passing Lyna River, is a landmark point along the new express route built in north-eastern part of Poland. The region is known for its difficult ground and water conditions with a significant presence of soft organic soils (peat, gyttja), underlain by impermeable soft silty clays and aquifer hard sands. The confined aquifer causes high water pressure with potentiometric surface up to 10 m above foundation level. After multivariate design concepts a floating type of foundation was introduced. The paper presents deep soil improvement with large diameter grouted columns forming massive blocks that transfer loads from the structure down to the silty clays and prevent from uncontrolled leakage of artesian waters. The paper also describes design details including finite element method analysis and compares the calculated deformations with on-site measured values. Finally, the authors share conclusions of successful application of grouted columns as the cost-effective and technically reasonable solution for bridge and adjacent embankments foundations.

**RÉSUMÉ:** Le pont en arc suspendu de 120 m de portée, traversant la rivière Lyna, est un point de repère sur la nouvelle voie rapide construite dans le nord-est de la Pologne. La région est connue pour ses conditions de sol et d'hydrologie difficiles avec une présence significative de sol organique mou (tourbe, gyttja), reposant sur des argiles limoneuses et un aquifère sableux dur. L'aquifère confiné engendre des surpressions interstitielles avec une surface d'équilibre des pressions jusqu'à 10 m au-dessus du niveau des fondations. Après de multiples concepts de dimensionnement, des fondations de type flottantes ont été sélectionnées. Cet article présente une amélioration de sol en profondeur avec des colonnes de mortier formant un bloc massif qui transfère les charges de la structure vers les argiles limoneuses en évitant des fuites incontrôlables provenant de la nappe artésienne. L'article décrit également les détails du dimensionnement incluant des calculs aux éléments finis et compare la déformation calculée avec les mesures sur site. Finalement, les auteurs partagent leurs conclusions sur l'utilisation avec succès de colonnes de mortiers comme une solution peu couteuse et techniquement adaptée pour le pont et les fondations de remblai adjacent.

**Keywords:** arch bridge; soft soil, artesian waters; grouted column; floating foundation;

# 1 INTRODUCTION

The construction of southern bypass of Olsztyn, the capital of Warmia-Masuria Province, is an important element of the strategy for the development of the road transport network in Poland. The task of the newly built route, planned mostly in the new track, is to take over the transit traffic carried out by the city of Olsztyn, improve transport accessibility to the industrial areas and improve communication with neighboring towns. The extensive transport system of the city and its outskirts will also affect the region's tourist attractiveness and enable further economic development. One of the significant elements of this investment is the road bridge being built over the Lyna River, recognized as 'MS-15' object.

# 2 GEOLOGICAL AND HYDROGEOLOGICAL CONDITIONS

The area within the presented bridge is located in the Vistulian Glaciation zone. In the substrate, below the surface layer, there are quaternary sediments in the form of peat and gyttja lake-river genesis with very high compressibility (a gel-like consistency) and thickness reaching up to 11 m, as well as glacial silty clays, from medium to high degree of plasticity. Under the cohesive soils, fine and medium sands of medium dense to hard aquifer sands are deposited.

The top groundwater is shallow, coarse waters with water table slightly pressured by the overlying cohesive layers. This level stabilizes at a depth of 0.5 ÷ 1.5 m below ground level. These waters occur in upper layers and depend on seasonal fluctuations in the water level in the Lyna River. The main groundwater level is found in bottom aquifer hard sands. The confined aquifer takes on an artesian form and causes high water pressure with potentiometric surface up to 10 m above foundation level. Therefore, not only low strength and deformation parameters of organics and cohesive top layers were a challenge for the foundations of the bridge (Table 1), but also potential real threats resulting from uncontrolled outflows of artesian waters.

*Table 1. Generalized stratigraphy, strength and deformation parameters*

| **Layer** *(stratum)* | $\gamma_{sat}$ (kN/m$^3$) | $\phi$ (°) | **c** (kPa) | $E_{oed}$ (MPa) |
|---|---|---|---|---|
| **PEAT/ GYTTJA** (*I*) | 10.5 | 3 | 3 | < 0.35 |
| Silty **CLAY / SILT** (*IIa*) | 20.0 | 13 | 12 | 8-10 |
| Silty **CLAY** *(IIb)* | 21.0 | 15 | 19 | 15 |
| Fine/Medium **SAND** (*III*) | 17.5-19.0 | 31 | – | 62-120 |

In the early 1990's a history of well drilling, located approximately 1200 m from the designed bridge, is remembered. After penetrating the top of the confined aquifer, there was a rapid outflow of pressured waters, which washed out significant volume of fines from the aquifer. The nearby area began to settle, which threatened the stability of surrounding objects.

There is also known a failure of a residential building foundations in the area of Lyna River (Damicz et al. 2007). During execution of the well, the sensitive layer was punctured causing spurting up through the hole the trapped grounwater, reaching about 10 m above the ground level. As a result of an uncontrolled outflow with a significant expense, the water flow removed large quantities of material from the aquifer. That resulted in the formation of a sinkhole which destroyed the ready part of the building.

There are also documented cases of foundations of bridge structures in similar ground and water conditions (groundwater under pressure), which confirmed the possibility of uncontrolled intensive outflows of artesian waters. One of the examples was a foundation failure of the bridge structure on the A2 motorway (Wojtasik and Różański 2011). The single-span structure was founded on 17 m long precast driven piles, which punctured the layer of cohesive soils isolating from the artesian groundwater. Unfortunately,

during pile installation an uncontrolled outflow of water occurred. Another analogous case, a bridge object located in the Kwisa River Valley, faced an uncontrolled outflow of waters from the bottom confined aquifer (Świeca and Walczak 2007). The loosening of the ground created a real risk of foundations' stability and a special repair treatment was needed.

One shall mention a case of bridge construction over the Huron River in Michigan in US (Byrum 2008) where the confined aquifer was punctured causing huge technical problems and delays on contract. This project was related to similar ground and water conditions as the presented MS-15 bridge. The only difference was the top layer, represented by loose grain soils instead of non-bearing organic soils. Already at the initial design stage the lightweight bridge construction was adopted and the deep foundations basing on long piles was rejected as dangerous (Figure 1).

The aforementioned examples of failures indicate a great difficulty in the geotechnical problem recognition when designing foundation solutions in such complex ground and hydrological conditions. Threats in locating the MS-15 bridge in the swamp - peat area of the Lyna River were, therefore, to be considered in terms of at least two challenges. The first of them was a non-bearing subsoil (peat, gyttia) with low strength and deformation parameters. The second, far more difficult, forcing greater caution in approaching the adopted foundation system was the presence of artesian waters and large hydraulic gradients.

## 3 ADOPTED SUPERSTRUCTURE AND FOUNDATIONS OF THE BRIDGE

At every stage of the design, geotechnical and bridge experts expressed similar opinions on the various foundation concepts of the MS-15 and emphasized that any method of foundation system requires special attention and caution. The general recommendations were to design a "light bridge" structure to transfer relatively low pressure to the ground and not to interfere with the confined aquifer.

Finally, after multivariate design concepts, a 120 m single-span bridge was introduced. The superstructure was made up of two steel arches with a tie, transversely braced. The deck was designed as a concrete slab combined with steel girders working as a tie. The hanging system was built of pre-stressed hangers forming a network. The abutments of the bridge were designed as massive reinforced concrete (Figure 2).

Taking into account the highlighted threats, it was decided to place the engineering object on the massive blocks of improved soil, made of grouted columns with a diameter of 2.4 m. The authors introduced a floating foundation system, suspended in low-bearing soils (*clay IIb*), without puncturing the artesian layer. Therefore, the risk of uncontrolled outflow was significantly mitigated. The adopted method allowed to improve the mechanical parameters of the soil as well as its tightness. The technology was non-impact and vibration-free, which eliminated the possibility of 'cracks' of impermeable soils (clay), and thus their hydraulic puncture and uncontrolled outflow of artesian waters.

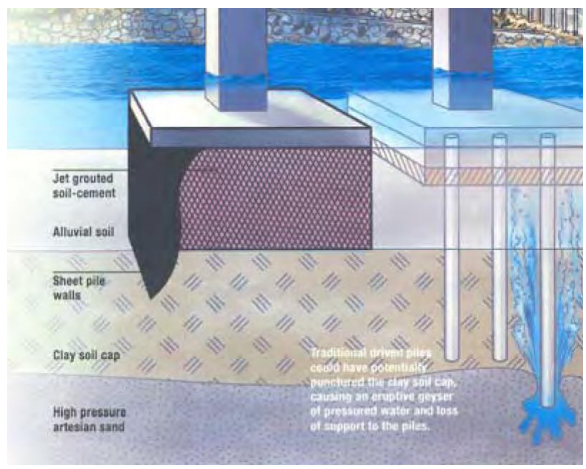


*Figure 1. The applied soil improvement under the foundations of the bridge over the Huron River and an exemplary illustration of the failure resulting from the hydraulic puncture of artesian waters (Byrum 2008).*

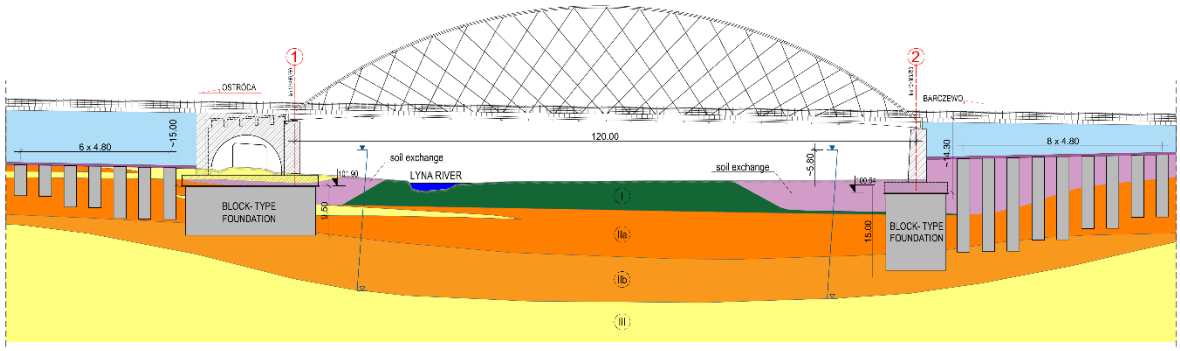


*Figure 2. Schematic longitudinal cross-section of the MS-15 with highlighted scope of non-bearing soil exchange*

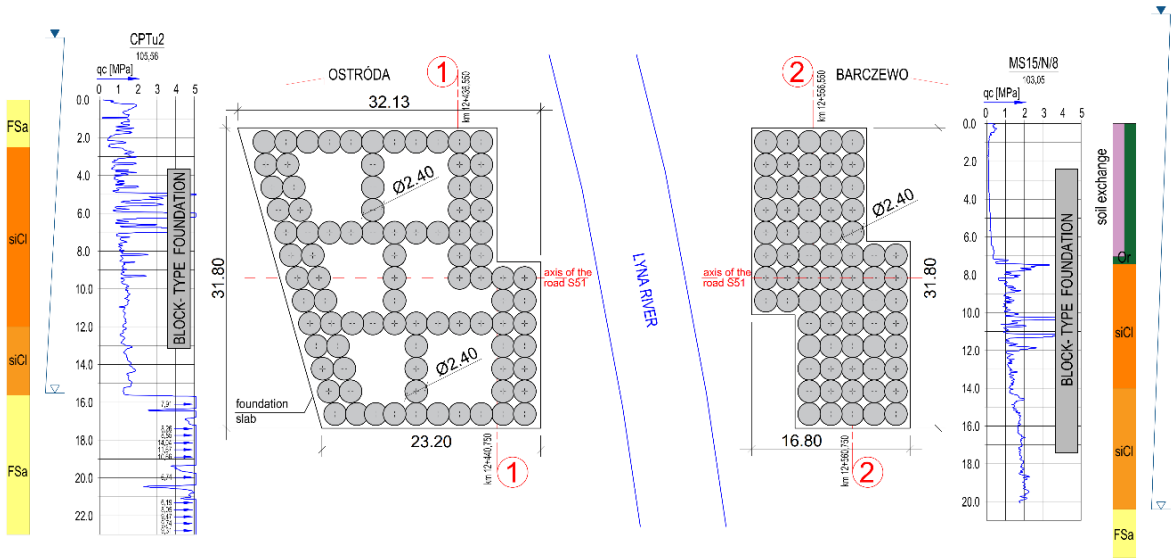


*Figure 3. Layout of grouted columns with a typical geological cross-section*

# 4 DESIGN STAGE

## *4.1 Calculation model*

For each support, finite element method (FEM) models in the longitudinal section were prepared in Plaxis with simulation of subsequent stages of superstructure and adjacent embankments installation. For the purpose of comparative analysis with monitored values on real structure, the results of calculations are presented in three crucial construction phases (Figure 4). Thus, the total displacements in the representative A-A section (Figure 4a) were estimated as following: **3.5 cm** after execution of engineered fill to level 107.0 m A.S.L. (Figure 4b), **6.8 cm** after execution of span and engineered fill to level 112.0 m A.S.L. (Figure 4c), **9.9 cm** after final leveling to 115.0 m A.S.L. and application of service loads (Figure 4d). In separate calculations, the value of residual settlements resulting from consolidation of compressible soils was determined to be 3-5 cm in 12 months after completion of works.

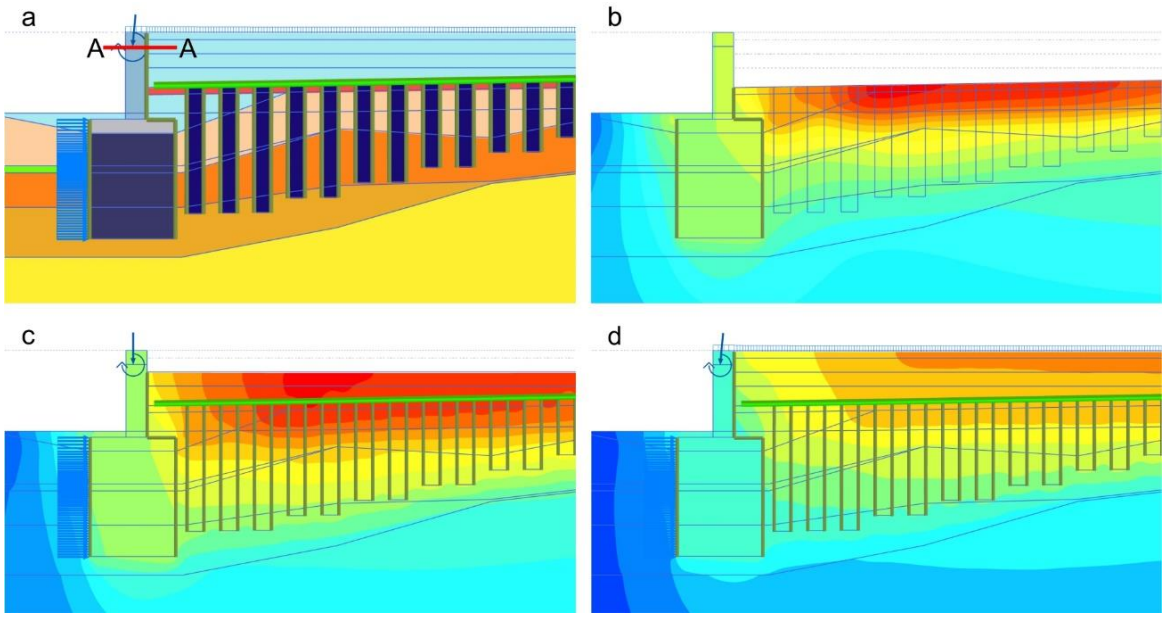


*Figure 4. Plane strain model in Plaxis 2D software (**a**). Map of vertical displacements after execution of: Phase 1 - engineered fill to level 107.0 m A.S.L. (**b**), Phase 2 - span and engineered fill to level 112.0 m A.S.L. (**c**), Phase 3 - engineered fill to final level 115.0 m A.S.L. and application of service loads (**d**)*

## *4.2 Structural (internal) bearing capacity of the grout*

In order to determine the required grout compression strength, the maximum force acting on grouted column was calculated. The analysis was based on the design force values coming from the most critical load combinations. Thus, the maximum design stress in the grouted column 0.804 MPa was obtained and compared with the design compression strength ($f_{cd}$). Hence, in accordance with formula (1), the minimum required characteristic compression strength ($f_{ck}$) was estimated 1.42 MPa. Local experience in implementation of grouted techniques indicated that the grout compression strength can be in the range of 1.5-3.0 MPa (Table 2). Finally, to keep the high safety level, restrictive 2.0 MPa as the minimum strength value and 3.0 MPa as the average grout strength were determined in the design.

$$f_{cd} = 0{,}85\,\frac{f_{ck}}{\gamma_m} \qquad (1)$$

Where 0.85 is a reduction factor due to possible long-term effects, resulting in deduction of grout strength, $\gamma_m$ is a partial safety factor ($\gamma_m = 1{,}5$ for permanent and variable loads and $\gamma_m = 1{,}3$ for exceptional loads).

*Table 2. Average characteristic compression strength of grout after Topolnicki (2018)*

| **Type of soil** | **Compression strength, $f_{ck}$ [2)] (MPa)** | **Cement content (kg/m$^3$)** |
|---|---|---|
| Peat [1)] | 0.5 – 1.0 | 350 |
| Organic mud [1)] | 0.7 – 1.5 | 320 |
| Clay | 1.0 – 2.0 | 280 |
| Clayey silt | 1.2 – 2.0 | 260 |
| Silt | 1.5 – 3.0 | 220 |
| Sand | 3.5 – 5.0 | 180 |

[1)] increasing content of organic content reduces compression strength
[2)] after 60 days of curing, at the specified cement content

*Table 3. Bearing capacity of the block-type foundations*

| Support in axis | $B_G$ (m) | $L_G$ (m) | $H_G$ (m) | $s_{u,s}$ (kPa) | $s_{u,b}$ (kPa) | $N_{d,k}$ (kN) | $N_{d,b}$ (kN) | $N_d$ (kN) | $R_{d,b}$ (kN) |
|---|---|---|---|---|---|---|---|---|---|
| P1 | 25.61 | 31.8 | 9.0 | 60 | 90 | 208.622 | 197.899 | 406.521 | 470.379 |
| P2 | 13.33 | 31.8 | 14.5 | 70 | 100 | 149.701 | 165.955 | 315.656 | 337.550 |

Where $N_{d,k}$ is a design value of the maximum vertical force in the base of abutment, $N_{d,b}$ is a design value of the block-type foundation dead weight ($N_{d,b} = B_G \cdot L_G \cdot H_G \cdot 20kN/m^3 \cdot 1{,}35$), $N_d$ is a design value of the maximum vertical force in the base of the block-type foundation ($N_d = N_{d,k} + N_{d,b}$).

### *4.3. Geotechnical (external) bearing capacity of the block-type foundation*

In one of the calculation steps, the design load capacity of the block-type foundations formed by the grouted columns was analysed in accordance with formula (2) after Franke (1992). Results of calculations are reported in Table 3.

$$R_{d,b} = [2 \cdot (B_G + L_G) \cdot H_G \cdot s_{u,s} + 5 \cdot (1 + 0{,}2 \cdot B_G/L_G) \cdot (1 + 0{,}2 \cdot H_G/B_G) \cdot s_{u,b} \cdot B_G \cdot L_G]/\gamma_R \quad (2)$$

Where $R_{d,b}$ is a design bearing capacity of the block-type foundation, $B_G, L_G, H_G$ are dimensions of the block-type foundation, $c_{u,s}$ is a mean undrained shear strength of the soil on the shaft of the block-type foundation, $c_{u,b}$ is an undrained shear strength at the base of the block-type foundation, $\gamma_R$ is a partial resistance factor according to EN-1997 ($\gamma_R = 1{,}1$).

## 5 CONSTRUCTION STAGE

### *5.1. Additional soil investigation*

Because the grouting method of soil improvement also created some risk of hydraulic puncture during the execution of works, a wide range of additional soil investigation in the area of bridge supports and adjacent embankments was done. The trustworthy top level of the artesian sands was determined. Then, after analysis of all available geological data, the columns' toe level was designed to keep at least 2 m distance from the top of the confined aquifer, ensuring adequate safety.

### *5.2. Preliminary laboratory tests*

For the purposes of the case study, numerous tests were carried out on the *in-situ* material. Laboratory tests were performed on two separate layers, i.e. pure organic soils with $I_{om} = 19{,}2\%$ and silty clays (Table 4).

*Table 4. Results of preliminary laboratory tests*

| Symbol of the tested soil (cement content, kg/m³) | Average grout strength (MPa) [1] | | | |
|---|---|---|---|---|
| | 7 days | 14 days | 28 days | 60 days |
| siCl [2] (320 kg/m³) | - | 0,97 | 1,61 | 1,71 |
| siCl [2] (320 kg/m³) | - | 1,62 | 2,32 | 2,48 |
| Or [2] (450 kg/m³) | 0,47 | 0,54 | 0,70 | 0,72 |
| Or [2] (600 kg/m³) | 0,65 | 0,98 | 1,05 | 1,87 |
| Or [2] (800 kg/m³) | 0,73 | 1,18 | 1,34 | 2,53 |
| Or [3] (600 kg/m³) | 0,69 | 1,03 | 1,21 | 1,92 |
| Or [3] (800 kg/m³) | 1,01 | 1,61 | 1,73 | 2,46 |
| Or+10% sand [2] (450) | 0,50 | - | - | 0,86 |
| Or+20% sand [2] (450) | 0,60 | - | - | 1,09 |
| Or+30% sand [2] (450) | 0,60 | - | - | 1,21 |

[1] the series included min. 3 samples of grout
[2] Portland cement CEM II/B-V 32,5R
[3] Portland cement CEM I 42,5R

Due to predominant effect of the soil type on the grout strength, the most reliable way to check the binding efficiency of the analysed soil with selected cements and to quantify the correlations are direct tests. Preliminary tests performed on in-situ soil samples mixed with cement content in

the laboratory and appropriately planned tests can significantly reduce the risk of not achieving on site the physical and mechanical designed parameters of the grout. One shall notice the organic content ($I_{om}$) in the tested soil is of great importance. With $I_{om} \leq 15\%$ comparable grout strengths can be achieved, as with the same soil with no organics, but this requires a significant increase in the cement content. After exceeding $I_{om} > 20\%$ strength is low even with significant cement dosing, which determines the rational limit of application of soil cementing technologies in the organic soils.

### *5.3. Dredging and flushing of organic soils*

Due to the fact that the object supports were located in non-bearing soil zone, it was decided to exchange them by dredging. In order to ensure the proper quality of the grouted columns after the organic soil was exchanged, additional control tests were carried out. After all, a map of the remaining organic soil was made (Figure 5). Despite the positive results of the preliminary tests on grout mixtures, prior to soil improvement, it was decided to carry out additional flushing of the remaining organic layers using jet-grouting.

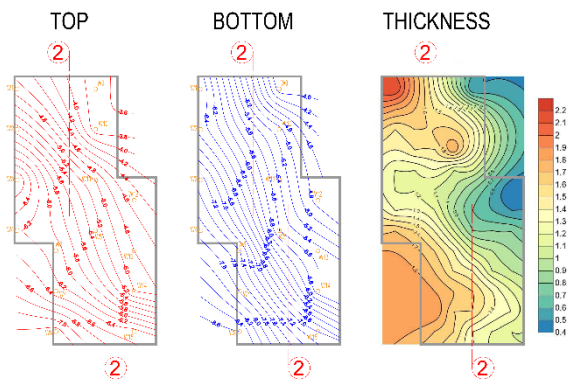


*Figure 5. Range of remaining organic soils confirmed after dredging - P2 foundation*

### *5.4. QC tests of soil improvement*

Basing on the laboratory tests and expertise, the quantity, type and appropriate batching of the binder was selected to ensure the specified grout parameters are met. All strength tests performed on production 114 samples, carried out as part of quality control, confirmed the effectiveness of the adopted grout technological parameters (Table 5).

*Table 5. Results of production quality control tests*

| **Support in axis** | **Min. grout strength (MPa)** | | **Mean grout strength (MPa)** | |
|---|---|---|---|---|
| | 28 days | 60 days | 28 days | 60 days |
| P1 | 2,11 | 3,18 | 3,19 | 3,78 |
| P2 | 2,15 | 3,10 | 2,76 | 3,84 |

### *5.5. Monitoring measurements*

As a part of the design work related to the foundation system, Keller has introduced a detailed guideline for measuring the bridge abutments deformations. The long-term monitoring commenced from foundation execution and is still proceeded. The analysis of settlement curves (Figure 6) indicate good convergence of average results with calculations made in Plaxis software (Figure 4). The last phase of loading the structure took place at the end of October 2018 (Phase 3 - day 311, see Figure 6). Despite systematic and reliable monitoring, too short period after the completion of works does not yet allow to observe the full stabilization of settlements. However, significant pace reduction of monitored values in the last 3 months is a promising result. Monitoring is still continuing and a further slight increase in settlements in coming months is expected resulting from the consolidation of cohesive soils under the deep foundation block.

## 6 CONCLUSIONS

The presented systematic monitoring and its results show a high convergence with the calculations, thus confirming the correctness of the adopted FEM models. The solution presented in the paper is the example of technically and economically successful execution of grouted columns as a deep foundation.

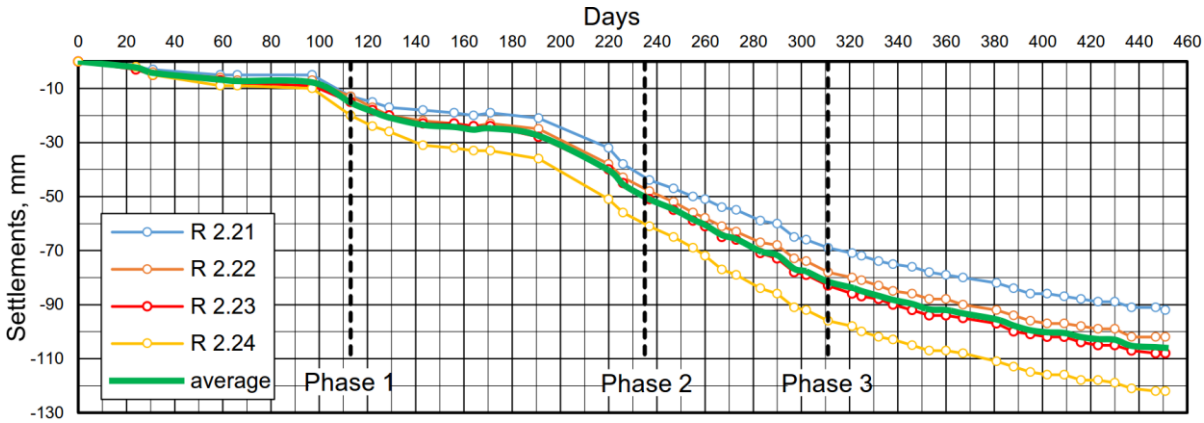


*Figure 6. Settlement curves of monitored benchmarks with highlighted construction phases – P2 foundation*

The introduced floating foundation system in the form of massive blocks of cemented soils is an innovative solution in such complex ground and hydrogeological conditions. Therefore, typical piling solution disturbing the integrity of the impermeable soil layer with a highly pressured waters were not required. It is also worth emphasizing that due to the high soil improvement factor, it was also possible to reduce the overall dimensions and amount of reinforcement in the foundations.

Currently, the completed MS-15 object is a characteristic, sublimated in terms of construction, point of a newly built express route in the north-eastern part of Poland.

*Figure 7. A general view of the bridge - final stage*

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