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Diaphragm wall heavy foundation of railway open-spandrel arch bridge

Utilisation de parois moulées comme fondations d'un pont ferroviaire en arche

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

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

ABSTRACT: Pomeranian Metropolitan Railway has been one of the most important and largest transport projects in northern Poland in the recent years, connecting Gdansk 'Lech Walesa Airport' with TriCity and providing transport services for over one million residents. The arch 80m span open-spandrel bridge is a part of double-track route and allows operation of trains overpassing one of the main roads in the region. The paper presents geotechnical design of deep diaphragm wall foundations for massive box-shaped abutments and discusses main challenges of transferring significant horizontal forces to the ground, affecting restrictive serviceability limits in all construction phases. The paper describes finite element method of modelling in geotechnical software compared with structural analysis based on sophisticated methods. The results are reported, analysed and collated with measured displacements in the operational phase. Complex construction works phasing of bridge elements combined with adjacent embankments and appropriate mobilisation of passive earth pressure is listed. Finally, the authors share conclusions of successful application of diaphragm wall as the cost-effective and technicaly reasonable solution for arch bridge foundations.

RÉSUMÉ: La voie ferrée Pomeranian Metropolitan a été l'un des principaux projets d'infrastructure de ces dernières années dans le nord de la Pologne. Elle relie l'aéroport international Lech Walesa de Gdansk avec les villes de la Métropole des TriCity, offrant ainsi un service de transport à plus d'un million d'habitants. Le pont en arche d'une portée de 80m fait partie de la double voie et permet le passage des trains au-dessus d'une des principales routes de la région. Cet article présente la conception des parois moulées comme fondations profondes des culées du pont et discute des principaux défis liés au transfert des charges horizontales dans le sol, ainsi que leurs effets sur les états limites de services durant toutes les phases de la construction. L'article compare les méthodes de calculs aux éléments finis du modèle géotechnique à celui du modèle d'analyse structurelle. Les résultats sont présentés, analysés et mis en relation avec les mesures effectuées sur site lors de la phase d'exécution. Les différentes phases de construction du pont sont listées et prennent en compte les remblais adjacents ainsi qu'une mobilisation appropriée des pression passives des terres. Enfin, les auteurs partagent leur conclusions sur une application efficace et raisonnable économiquement des parois moulées comme fondation de ponts en arches.

Keywords: railway bridge; arch bridge; diaphragm wall; ground improvement; deep foundations;

1 INTRODUCTION

The Pomeranian Metropolitan Railway has been one of the most important and largest transport projects in the northern Poland in recent years. The new 19.5 kilometre section of the railway connects Gdansk ‚Lech Walesa Airport‘ with TriCity and other parts of the Pomeranian Voivodship providing easy access to transport services for over one million residents. A part of the newly designed railway covers a historical trail of the Kokoszkowska Route connecting Kashubian area with Gdansk in 1914 - 1945. The form of the engineering structure named WK-11 (Figure 1) inscribes into the existing landscape as a kind of a symbolic gateway to Gdansk (Łucki et al. 2016).

Figure 1. A general view of the railway viaduct WK-11 (photo by Tadeusz Brzozowski)

2 GEOLOGICAL CONDITIONS

The area within the mentioned viaduct is located in the region of the Eastern Pomeranian Lake District and forms part of the upland of the Kashubian Lake District. In the substratum, below the surface layer of the uncontrolled fills there is a layer of silty clay and cohesive-organic soils i.e. dark grey silty coal. In addition, non-cohesive soils have been found in the form of medium-compacted and compacted silty and fine sands and gravels. Locally, below the embankments, there were found thin lenses of organic soils in the form of peat.

It is worth emphasizing that the abutments of the new viaduct were deliberately located in the place of high embankments of the historical trail that has caused pre-consolidation stress in the soils. From a geotechnical point of view, pre-consolidation stress has great importance because it separates elastic and reversible deformations from inelastic and only partially irreversible deformations and marks the starting point of high compressibility.

Table 1. Generalized stratigraphy, strength and deformation parameters

Layer *(stratum)*	γ_{sat} (kN/m^3)	ϕ (°)	**c** (kPa)	E_{oed} (MPa)
FILL (*I*)	19,0	27	–	–
Silty **COAL** (*II*)	19,5	16	25	30
Silty **CLAY** *(III)*	21,0	18	32	40
Fine/Medium **SAND** (*IV*)	19,0-20,0	30	–	50-125
Sandy **GRAVEL** (*V*)	20,0	39	–	>150

In order to determine the parameters of individual layers of the subsoil, CPT tests were used. Distinguished layers (Table 1) fit the soil classification adopted to local conditions after Robertson et al. (1986) and Schmertmann (1978), respectively for sand deposits and sensitive silts.

$$M_0 = E_{oed} = \alpha_m \cdot q_c \qquad (1)$$

The constrained modulus (M_0) for main stratum was derived using formula (see Eq. 1) including cone resistance (q_c) multiplied by a coefficient (α_m) appointed after Senneset et al. (1989) adapted to regional conditions, with a coefficient equal 8 for clays and 5 for sands.

3 STRUCTURE CHARACTERISTICS

The load-bearing structure of the bridge is a reinforced concrete arch with a top running 80 m span. The construction of the arch consists of two parabolic girders with variable cross-section heights supported by abutments by means of hinged bearings. Topping structure forms a reinforced concrete slab joined in the central part with main girders and based on them by means of hinged columns on the remaining sections. The slab is terminated with concrete end beams (Figure 2). The object supports were designed in the form of massive reinforced concrete box-shaped abutments. Arched girders and the topping slab are based on their front walls. The rear wall together with openwork side walls formed a support element for the embankment and the top slab.

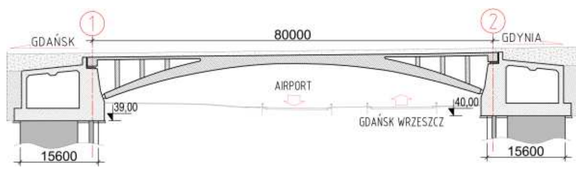

Figure 2. Overall scheme of longitudinal section of the railway viaduct structure

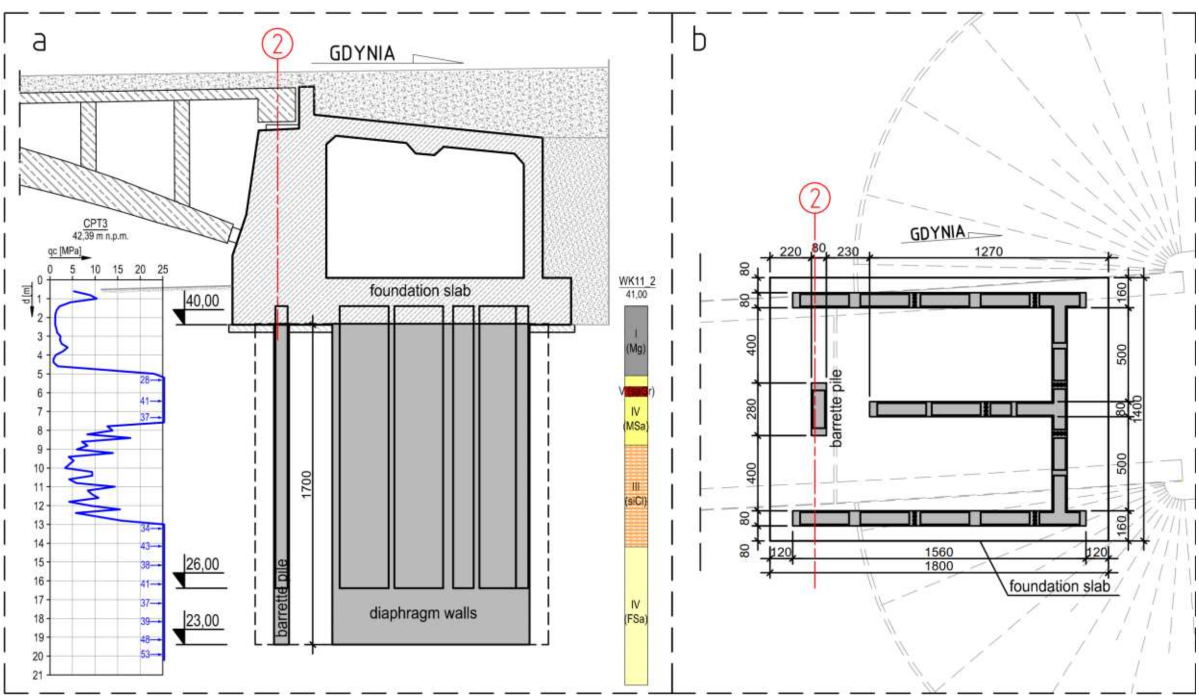

*Figure 3. Deep foundations of abutment in axis 2: longitudinal section (**a**), D-wall layout (**b**)*

Foundations of the bridge structure were designed in the form of deep diaphragm walls (D-wall) with a thickness of 80 cm, forming an „E“ letter shape (Figure 3b). Basing on geological conditions and static analysis, depth of the foundation elements was determined to be 19,0 m and 17,0 m, respectively for support in axis 1 and axis 2. The D- wall sections included straight sections, T-shaped sections and a separate D-wall section on the span side, the-so-called barrette (Figure 3b).

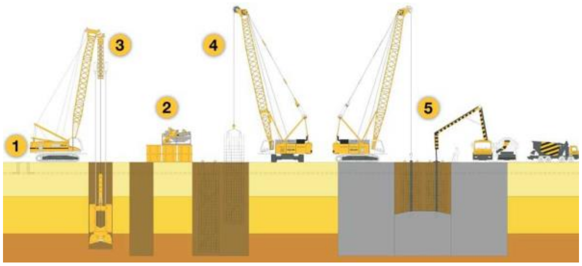

Figure 4. Stages of diaphragm wall execution

4 DESIGN STAGE

The decision to implement heavy foundation diaphragm walls was made to offer the WK-11 object appropriate stiffness of the foundation system and let the effective transfer of significant horizontal (over 95 MN) and vertical design loads from the structure to the ground. Therefore, the very crucial design issue was to estimate the proper interaction of deep foundations, bridge structure and adjacent, high embankments.

4.1 Calculation phases

As a part of static analysis, four representative construction phases were distinguished (Figure 6). These phases were implemented in the further sophisticated numerical modeling. For the initial phase (‘0‘), at-rest earth pressure coefficient ($K_0 = 0{,}63$) was determined according to Polish code (PN-83/B-03010) based on the formula:

$$K_0 = [\, 0{,}5 - \xi_4 + (0{,}1 + 2 \cdot \xi_4) \cdot (5 \cdot I_S - 4{,}15) \cdot \xi_5] \tag{2}$$

Where I_S is a density factor of engineered fill, ξ_4 is a coefficient dependent on material type of fill ($\xi_4 = 0{,}1$), ξ_5 is a coefficient dependent on technology of fill compaction ($\xi_5 = 0{,}9$).

For the subsequent construction phases of the object, the earth pressure values were determined in correlation with Plaxis 3D software analysis based on the most realistic deflections of the structure (Figure 5). The result of such correlation resulted in final coefficients $K_1 = 0{,}70$, $K_2 = 1{,}50$ and $K_3 = 2{,}20$, set for phases 1, 2 and 3, respectively (Figure 6).

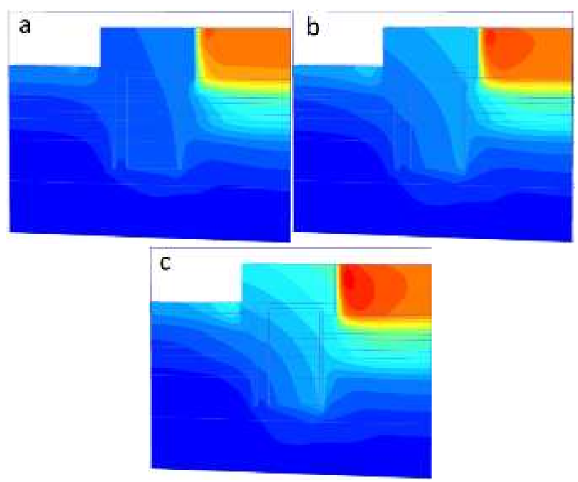

*Figure 5. Total displacement results for the abutment in axis 2: phase 1 (**a**), phase 2 (**b**) and phase 3 (**c**)*

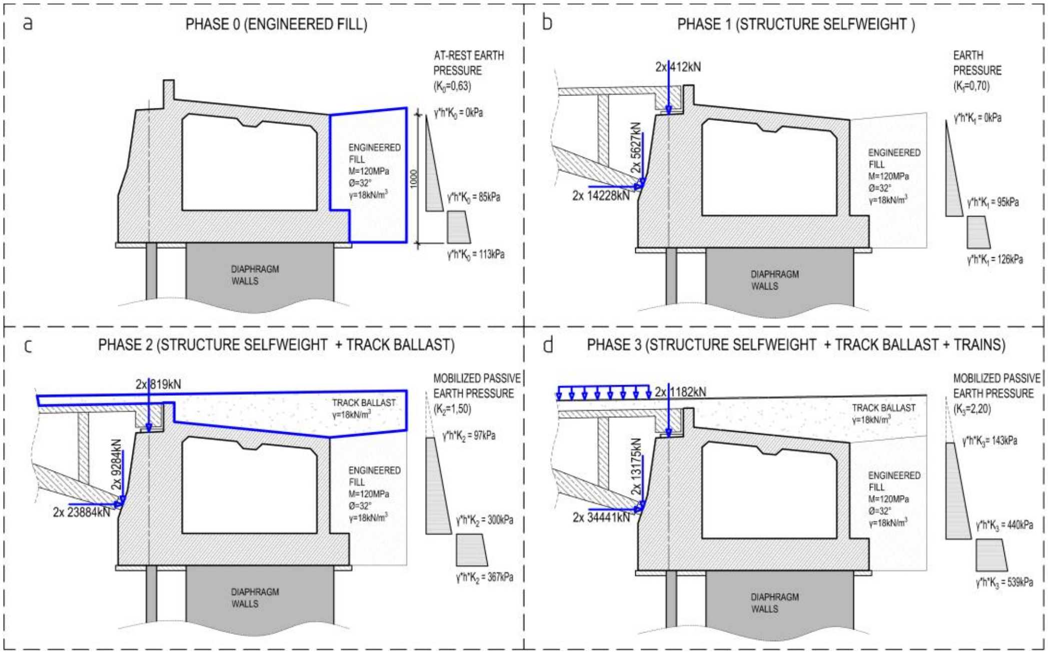

*Figure 6. Construction phases with characteristic loads schemes: phase 0 – engineered fill installation (**a**), phase 1 – loading with structure selfweight (**b**), phase 2 – additional loading with track ballast (**c**), phase 3 – additional loading with trains (**d**)*

4.2 Structural model

The high complexity of described interactions and deep foundations consisting of concrete spatial elements, required three-dimensional calculation models. Internal forces in diaphragm walls were determined based on the results of static calculations in Autodesk Robot Structural Analysis (*ARSA*). Due to the highly advanced multitask, the calculations were cross-checked by external experts using independent finite element method (*FEM*) models in the *SOFiSTiK* environment. The numerical model in *ARSA* software was built using three-dimensional shell elements - classes e2, p3. Between the individual panels, which simulate real sections of diaphragm wall, linear (vertical) hinged joints were introduced. The interface of D-walls in the ground was provided by vertical and horizontal elastic springs. Their stiffness was estimated based on method after Kosecki (2006). As a result, the values of the soil horizontal stiffness modulus (K_x) were determined on the basis of formula:

$$K_x = n_0 \cdot n_1 \cdot n_2 \cdot S_n \cdot \kappa \cdot \phi \cdot E_0 \tag{3}$$

Where n_0 is a corrective coefficient concerning pile diameter effect ($n_0 = 1{,}0$), n_1/n_2 are coefficients concerning spacing of piles in a group arranged in the plane, respectively perpendicular and parallel to a direction of horizontal load ($n_1 = n_2 = 1{,}0$), S_n is a coefficient concerning method of pile embedment ($S_n = 0{,}9$), κ is a coefficient concerning three-dimention soil response, dependent on pile cross-section (κ = 1,0), ϕ is a coefficient concerning long-lasting or cyclic loads effect ($\phi = 0{,}45$), E_0 is deformation modulus of the soil.

It is worth mentioning that the characterstics of horizontal elastic springs for D-walls were diversified for the front and rear diaphragm walls taking into account real effect of the weight of adjacent embankments (Figure 7).

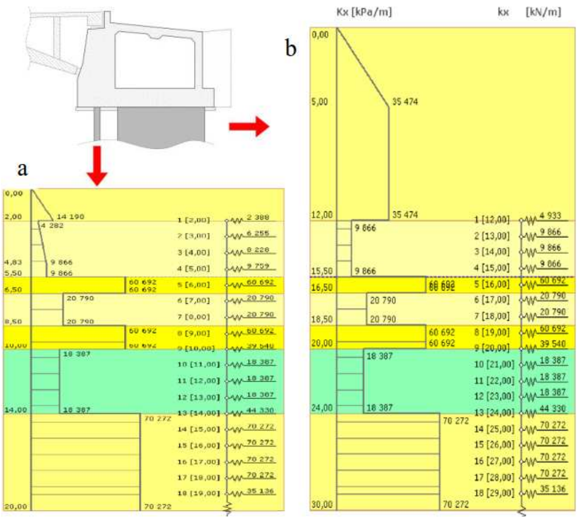

*Figure 7. Abutment in axis 2. Stiffness of horizontal springs in the parallel direction to horizontal loads: front D-wall (**a**), rear D-wall (**b**)*

4.3 Bearing capacity of diaphragm walls

In the presented case, the Bustamante and Gianeselli (1982) design method based on *in situ* soil parameters from CPT tests was chosen to derive the ultimate bearing capacity of D-wall base (see Eq. 4) and shaft (see Eq. 5).

$$Q_L^b = k_b \cdot q_{ca} \cdot A_b \tag{4}$$

Where Q_L^b (kN) is a base capacity, k_b is a coefficient dependent on soil type and piling technique (for D-walls in dense sands $k_b = 0,3$), q_{ca} (MPa) is an arithmetic mean value of cone resistance over a length of ±1,5 times D–wall thickness, A_b (m²) is an area of D-wall base.

$$Q_L^s = \sum\left(\frac{1}{k_{si}} \cdot q_{ci} \cdot A_{si}\right) \tag{5}$$

Where Q_L^s (kN) is a shaft capacity, k_{si} is a coefficient dependent on soil type and and piling technique (for D-walls in dense sands $k_s = 150$, in hard clays $k_s = 60$), q_{ci} (MPa) is a cone resistance over a length of certain i layer, A_{si} (m²) is an area of D-wall shaft.

Thus, the value of allowable design load that can be transfered through D-wall to the ground, taking into account partial safety factors after Bustamante and Gianeselli (1982) is:

$$Q_w = \frac{Q_L^s}{2} + \frac{Q_L^b}{3} \tag{6}$$

In the presented case, for the foundations of abutment in axis 2, the estimated design load capacity of the diaphragm wall per linear meter of the element was $Q_w = 4967\ kN/lm$ (Figure 9), whereas the maximum design load was $F_z = 3910\ kN/lm$ (Figure 8c).

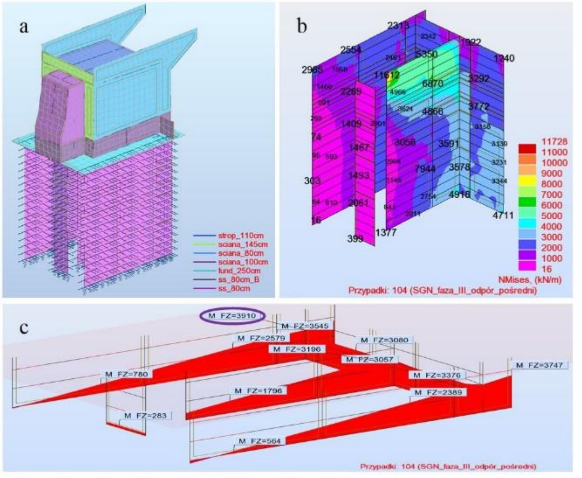

*Figure 8. Abutment in axis 2. FEM ARSA model of abutment integrated with D-wall foundation (**a**), membrane forces in D-walls (**b**), vertical design reactions at base of D-walls (**c**)*

Technology =>	D-WALL	GROUP IA	Concrete class =>	C30/37	Reinforment type =>	Cage	LCPC method	KELLER
Length of D-wall section	L =	17,0	m	Circumference O_s=	2,000	FOS^s=	2,0	- shaft
Width of D-wall section	D_p=	0,8	m	Area A_b=	0,800	FOS^b=	3,0	- base

	Layer	bottom level [m]	L_z [m]	q_c [MPa]	k_c	k_s	$Q_s=q_c/k_s{*}O_s{*}L_z$ (shaft) [kN/m]	$Q_b=k_c{*}q_c{*}A_b$ (base) [kN/m]	D-wall design bearing capacity		
1.	Fill	3,50	3,50	0,0	0,00	60	0	0			
2.	Gravel	4,50	1,00	20,0	0,30	150	267	0	$Q_L^s = \Sigma Qsi$ =	4333	kN/m
3.	Fine Sand	6,50	2,00	25,0	0,30	150	667	0			
4.	Gravel	8,00	1,50	20,0	0,30	150	400	0	Q_L^b = Qb =	8400	kN/m
5.	Silty Clay	12,00	4,00	5,0	0,45	60	667	0			
6.	Medium Sand	20,00	5,00	35,0	0,30	150	2333	8400	Q^w =	4967	kN/m

Figure 9. Abutment in axis 2. Calculation of design vertical load bearing capacity of D-wall

5 CONSTRUCTION STAGE

5.1 Horizontal load test of D-wall section

In order to verify the design assumptions, a horizontal load test of the barrette was carried out in the area of abutment in axis 2. The test was performed up to the load $H_{test} = 1460\ kN$. Horizontal deflection of the barrette on last load step (H_{test}) was 1.8 mm at a base level and 2.8 mm 100 cm above the base level. Thus, the barrette rotation angle could be estimated as $0{,}97 \cdot 10^{-3}\ (rad)$.

The test result confirmed a significantly higher horizontal stiffness of the separated diaphragm wall section than the one assumed in the design. This leads to conclusion that either D-walls will transfer greater part of horizontal loads from the abutments to the ground than expected, or they will work with greater bearing capacity reserve, what increases safety level of the applied geotechnical solution.

5.2 Construction of engineered fill behind the rear wall of abutment

In static calculations it was assumed that vertical loads from the abutments are fully transferred by the diaphragm walls to the ground. However, to ensure safe foundations for the railway viaduct, transfer of significant horizontal loads (over 95MN) and consequently overturning moments, remained a critical issue. Therefore, iterative analysis of displacements and stresses was done as well as sophisticated correlation of structure behaviour in various software, presented in detail in chapter 4.

However, it was necessary to prepare the instructions of proper engineered fill installation behind the rear wall of abutment, meeting the rigorous design assumptions. Therefore, the minimum QC criteria for the fill were set as following: well-graded gravel (uniformity coefficient $C_u > 5$, fines $f < 5\%$), friction angle ($\phi > 32°$), density factor ($I_S \geq 1{,}0$), deformation modulus from Plate Load Test ($E_2 \geq 120\ MPa$).

For additional safety reasons, soil friction under the foundations of the abutments was neglected due to the high possibility of block-like type of displacement of the whole foundation system, including soil mass enclosed within the diaphragm walls.

To control the process of engineered fill compaction, 12.0 m wide area behind the rear wall of the abutment was designated and monitored (Figure 10). The fill was installed layer by layer, supervised and tested. Finally, after its full erection, the quality of compaction was reverified by means of series of dynamic probings performed 2.0 m behind the structure.

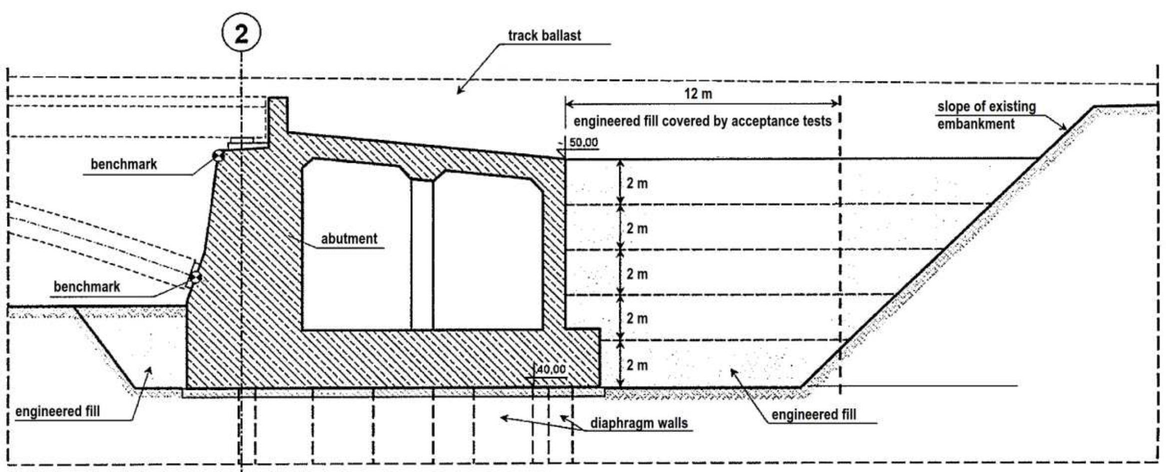

Figure 10. Abutment in axis 2. Scheme of fill execution behind the rear wall

6 CONCLUSIONS

The presented case of diaphragm wall heavy foundation was implemented for railway open-spandrel arch bridge in the restrictive conditions. The applied system was veryfied with the full-scale load test with trains that validated the design. Benchmark installed on the object were monitored in all construction phases and the reported measurements proved the results are fully convergent with the calculations performed.

The exact location of the new abutments situated in the place of high embankments of the historical trail and consequently preconsolidated soils with improved geotechnical parameters, has brought additional benefits for the serviceability limits of the new structure.

The presented concept of diaphragm wall foundations offers extreme horizontal stiffness and is highly recommended for all structures where deflections shall be limited to minimum.

Before choosing any geotechnical solution, the geo-engineer has to consider a variety of components: the applicability of certain technology and its limits, type of structure, type of applied loads, structure sensitivity to deflections and type of soil, or history of preloading the soils. It is also highly recommended to perform field tests prior to the commencement of works, to set appropriate QA/QC procedures and monitor the real life of the structure in order to verify the implemented solution, maintain the high quality of work and mitigate any potential risk. The applied solution also needs to fit the construction timetable and finally has to be economically viable. Thus, geotechnical engineering has to face many challenging demands.

The presented case is an example of successful application of diaphragm wall foundations as the cost-effective and technicaly reasonable solution for arch bridge foundations. Currently, the WK-11 bridge has become a distinctive point and a landmark object being a characteristic point of the modern Gdansk infrastructure.

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