The D4R7 Reinforced Soil Retaining Walls in Bratislava, Slovakia

Elena Gil, *VSoL® Operations Manager, VSL Construction Systems, Barcelona (Spain); elena.gil@vsl.com*
Carlos Serrano, *VSoL® Sales Manager, VSL Construction Systems, Barcelona (Spain); carlos.serrano@vsl.com*
Carlos Pereira, *Project Manager, VSL Construction Systems, Paço de Arcos (Portugal); carlos.pereira@vsl.com*
Pedro Osso, *Deputy General Manager, VSL Construction Systems, Barcelona (Spain); pedro.osso@vsl.com*
Juan Lima, *Regional Technical Director, VSL Construction Systems, Barcelona (Spain); juan.lima@vsl.com*
Ivan P. Damians, *Ph.D.; Assistant Professor at Universitat Politècnica de Catalunya·BarcelonaTech (UPC); Associate Research Professor at International Centre for Numerical Methods in Engineering (CIMNE), Barcelona (Spain); ivan.puig@upc.edu; VSoL® R&D, VSL Construction Systems, Barcelona (Spain); i.puig@vsl.com*

ABSTRACT: *This paper presents the extensive use of reinforced soil retaining walls in the D4-Motorway and R7-Expressway Project in Bratislava (Slovakia). The D4R7 Project, with a total of 47,100 m² of total facing area among all 60 reinforced soil wall structures involved, corresponds to one of the largest mechanically stabilized earth wall applications in a single project in the last five years in Europe. The VSoL® reinforced soil retaining wall system used in this project were both with polymeric strips and steel ladders reinforcement types, comprising retained earth solutions as simple walls with heights up to 15 meters, with or without backslopes on top, true and false (piled) bridge abutments, back-to-back cases, as well as different facing solutions with precast panel and welded wire mesh stone facing alternatives. The paper provides a general presentation and specific data resulting from the design calculations, materials, production, and construction of the walls, specifically focusing on the true bridge abutments and stone facing alternatives installed. Additionally, this study provides sensitivity analyses in the mechanical performance verifications for both ultimate and serviceability limit states through 2D FEM modeling calculations assuming both polymeric strips and steel ladders reinforcement types.*

KEYWORDS: reinforced soil bridge abutment, polymeric strip reinforcement, numerical modeling, facing displacements.

SITE LOCATIONS: **Geo-Database**

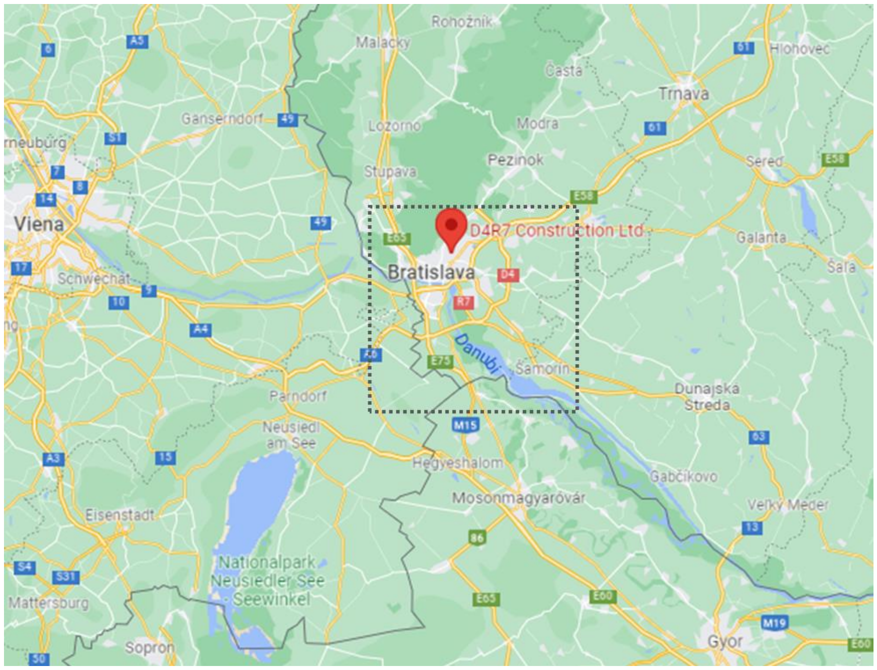
INTRODUCTION: WORK DESCRIPTION

Reinforced soil wall systems are cost-effective and environmentally competent, high-performance retaining wall systems that have shown excellence in a wide variety of applications, providing an eco-friendly and better sustainable alternative as compared to other conventional or traditional retaining earth solutions (Damians et al., 2017a; 2019; 2018). Since its development in the early 1980s, the VSoL® wall system (VSL 2021) has been used extensively worldwide to provide cost-effective and aesthetically appealing retaining wall solutions on a wide range of infrastructure projects. To date, more than 6 million m² of VSoL® walls have been built around the world. The system combines concrete or welded wire mesh facings and soil reinforcement made from steel ladders or polymeric strips, installed into compacted fill to form a coherent retained earth block which resists forces generated within and behind the wall. About 2/3 of constructed VSoL® walls have been designed and executed with polymeric reinforcement and 1/3 have been designed and executed with steel reinforcement. The system is widely used for temporary or permanent projects ranging from general grade separation retaining walls to highway bridge abutments and mining structures.

The D4R7 Project, with a total of 47,100 m² of facing area considering all 60 reinforced soil wall structures involved, is one of the largest reinforced soil wall applications in a single project in Europe in 2020. The D4R7 - Bratislava Bypass is a public private partnership project, defined as a long-term contract between a private party and a government entity. The D4, a new 27 km long motorway between Jarovce and Rača (see Figure 1), is intended to relieve existing routes through and within Bratislava by providing a new high-capacity bypass road to access the city. The R7, a new 32 km long radial expressway

Submitted: 1 July 2021; Published: 1 October 2022
Reference: Gil, E., Serrano, C., Pereira, C., Osso, P., Lima, J., and Damians, I.P. (2022). The D4R7 Reinforced Soil Retaining Walls in Bratislava, Slovakia. International Journal of Geoengineering Case Histories, Volume 7, Issue 2, pp. 13-33, doi: 10.4417/IJGCH-07-02-02

running in a south-easterly direction from the city center (Figure 1), is proposed to alleviate traffic on existing radials through heavily urbanized areas and to eliminate daily traffic jams to the city. Light traffic is expected to go down by 15%, and heavy traffic by 50% (D4R7 2017). This project will most likely improve traffic conditions in the region, but also prove advantageous in the long term by improving opportunities for economic development as well as by boosting employment and the local supply chain.



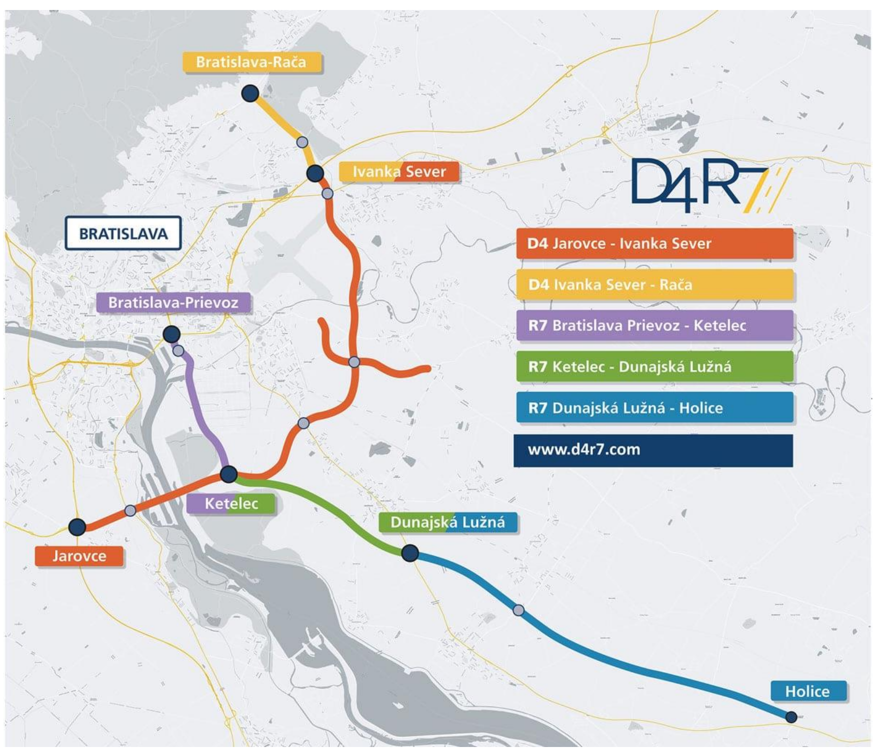


Figure 1. Location of the D4R7 project in Bratislava and related zones detail (from D4R7 2017).

Although the solution has been innovative in Slovakia, the VSoL® reinforced soil retaining wall system was extensively used in this project, both with polymeric strips and steel ladders reinforcement types, as a common geotechnical structure and with a wide range of solutions, from simple walls with or without slopes on top, true and false (piled) bridge abutments, and back-to-back wall cases. Facings of all structures were designed and executed with precast concrete panels, except for one structure with direct abutments designed and assembled with welded wire mesh (WWM) stone facing for aesthetics.

REINFORCED SOIL STRUCTURES

Reinforcement System Features

Two types of reinforcement were supplied for this project, depending on the type of structure. 67% of the total area of the reinforced walls were assembled with polymeric reinforcement (751,850 m of total polymeric reinforcement length) and 33% were assembled with steel reinforcement (292,625 m of total steel reinforcement length).

The polymeric strips reinforcement (see Figure 2a) is made of high-tenacity polyester fiber concentrated in a number of separate bundles (yarns) and coated with a polyethylene sheath using a die extrusion process. The outer surfaces of the reinforcing strip have a knurled finish to ensure an effective frictional surface for interaction with the fill soil particles. Production of strips is monitored throughout all stages of manufacturing to control the mechanical properties, the coating thickness, the strip width, the roll length (100 – 200 m), and the final product weight. Polymeric strip reinforcements have different strength capacities (grades). For this project, the following grades (and proportional quantities) have been used: 27kN (14%), 30 kN (18%), 36 kN (19%), 45 kN (17%), 54 kN (13%), 63 kN (18%), and 70 kN (1%). The nominal-related width (w) and thickness (total) of the used strips vary from 46 to 90 mm, and from 1.8 to 2.8 mm, respectively, which depend on the strips' grade and type.

The steel ladders reinforcement system is manufactured from cold-drawn galvanized steel bars, comprising both longitudinal and transverse bars that are fusion-welded to form long and narrow "ladder" elements (see Figure 2b). A connection loop is formed at one end of each longitudinal wire, where the connection to the panel is performed by inserted/precast steel loops. The reinforcement-to-panel connection is generated by matching the loops from both parts and inserting a straight pin through them. The longitudinal bars consist of smooth bars with a diameter ranging from 8 mm to 10 mm. Reinforcement element lengths used varied from 3 to 11 m, with transversal members spacing (d_i) from 0.15 to 0.5 m, and with initial spacings (d_0) from 0.45 m to 1.0 m. The soil-reinforcement interaction strength is provided mainly by the transverse wires of the ladder, where bearing resistance takes place due to the soil interlocking effect, generating a non-negligible dilatancy effect at low confinement scenarios and consequently obtaining a mobilized interface strength which may be up to 2.5 times larger than the frictional strength of the soil at low confinement conditions (Damians 2016). The galvanization of the steel ladder is conducted to protect it against corrosion and hence to ensure the durability of the reinforcing system for a specified design life of the structure. Usually, the ladders are hot-dip galvanized to a certain thickness, which depends on code and project specifications, e.g., minimum thicknesses of 70 µm according to EN 14475 (BS EN 2006) with an average value of 85 µm (ISO 1461, 2009). However, for this project and due to the galvanizing process, the average thickness of galvanization was above 115 µm for all the steel reinforcements assuring, in addition to the proper fill selection with electrochemical features suitability, an expected good performance during the service life (100 years) of the structures.

Other components/accessories supplied were the panel-to-panel horizontal joint HDPE bearing pads (20,278 units), used mainly to allow the proper distribution of the vertical facing stresses as well as the differential settlements between the facing and the backfill (Damians *et al.*, 2013; 2016; 2017b); galvanized steel straight and L-shape connector pins (31,247 and 2,722 units, respectively); plastic wedges (61,582 units) to ensure a proper and tight installation of the steel ladders with the connection; and geotextile sheets (8,676 m^2), covering all inward panel joints and avoiding backfill loss while allowing water drainage.

Regarding the facing with precast facing panels, a rectangular shape was chosen for this project with general dimensions of 2.25 m (width) × 1.5 m (height) × 0.14 m (thickness), a standard panel type. A total of 13,652 units of panels were required to install all the reinforced soil structures. Precast panels were painted on the inside face to provide additional (precautionary and probably unneeded) protection due to an eventual chemical attack from the backfill. For the WWM stone facing wall, a total of 500 units of welded wire mesh (WWM) with dimensions of 2.25 m (width) × 1.5 m (height) were supplied to complete the assembling of this kind of structure.

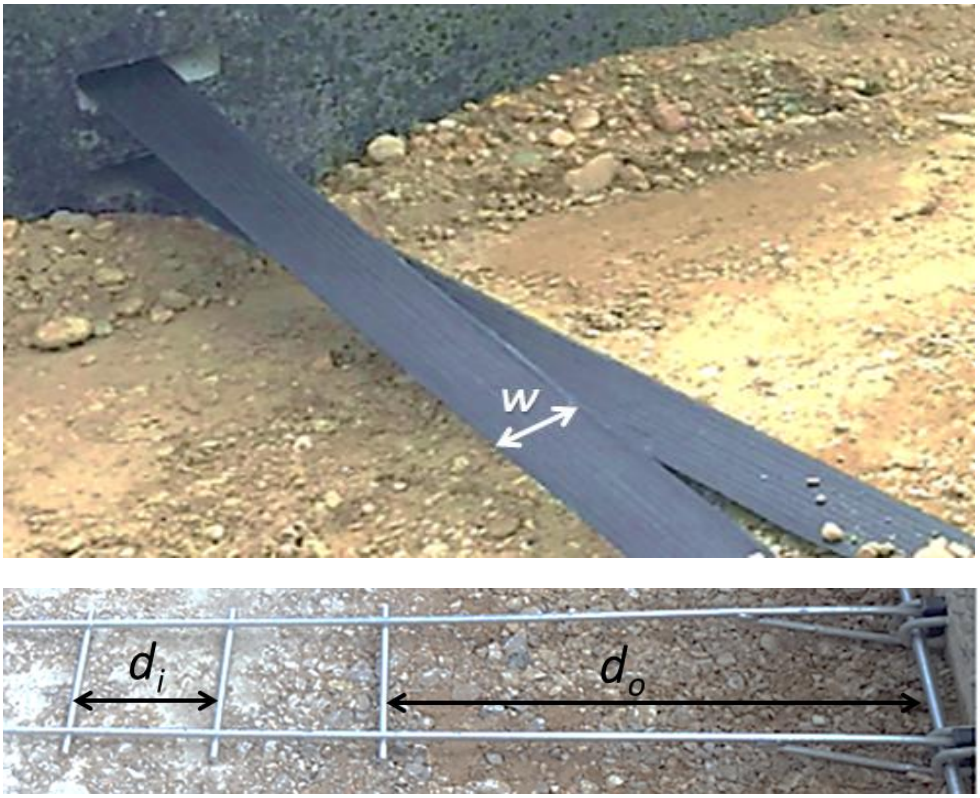


Figure 2. Details of the VSoL® wall reinforcement types used in the D4R7 Project: (a) polymeric strips and (b) galvanized steel ladders and related connection systems (void formed and clevis loops, respectively).

Types of Structures

As aforementioned, different types of reinforced walls were designed and executed for this project, taking into consideration the different types of reinforcement needed for each type of structure (see Table 1).

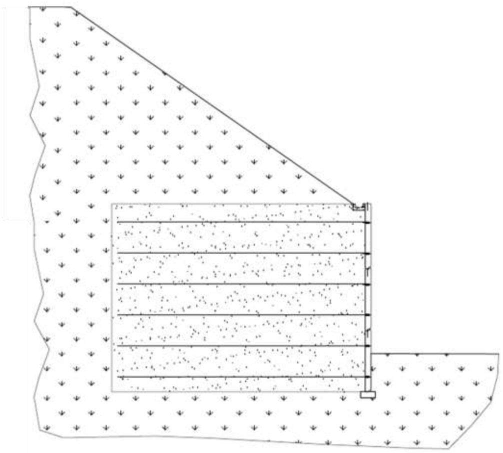
Table 1. D4R7 Project reinforced soil structures inventory.

Wall type:	Total facing surface:		Ranging wall heights (m):	Reinforcement type:
	m^2	%		
Simple walls [(a)]	20 747	44	1.5 – 14.3	Polymeric strips
False (piled) bridge abutments [(b)]	10 857	23	4.2 – 11.3	Polymeric strips
True abutments [(b)]	14 125 + 1 336 [(c)]	33	3.9 – 8.6	Steel ladders

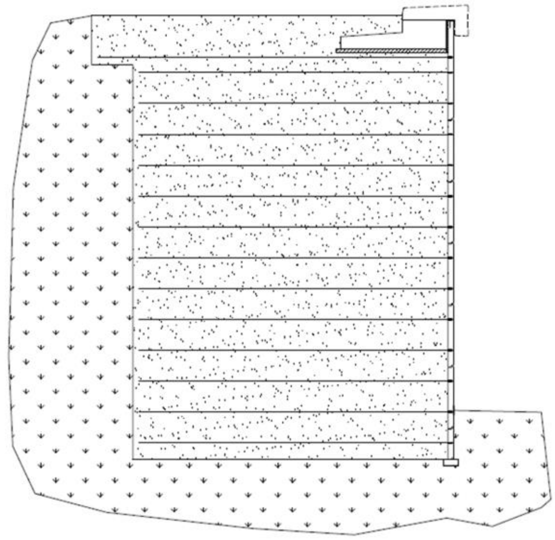
[(a)] including horizontal and backslope on top, and back-to-back wall type;
[(b)] including the lateral wing-walls;
[(c)] welded wire mesh stone facing wall/slope types.

Simple Walls

Simple walls are walls with or without slope on top. A total of 20,747 m^2 of walls were designed and constructed (Figure 3). A simple wall also includes reinforced soil wall specific configurations, as for the the back-to-back wall cases (Figure 4). All these walls used polymeric strip reinforcements. The total reinforced area includes the lateral extension of wing-walls attached to some of the direct and false -piled- abutments. The maximum wall height and the maximum reinforcement length were 14.3 m and 14.5 m, respectively.



a) H = 4.6 m (+ 4.8 m slope) case



b) H = 10.7 m case

Figure 3. Simple reinforced soil wall detailed section examples: (a) with and (b) without slope on top.

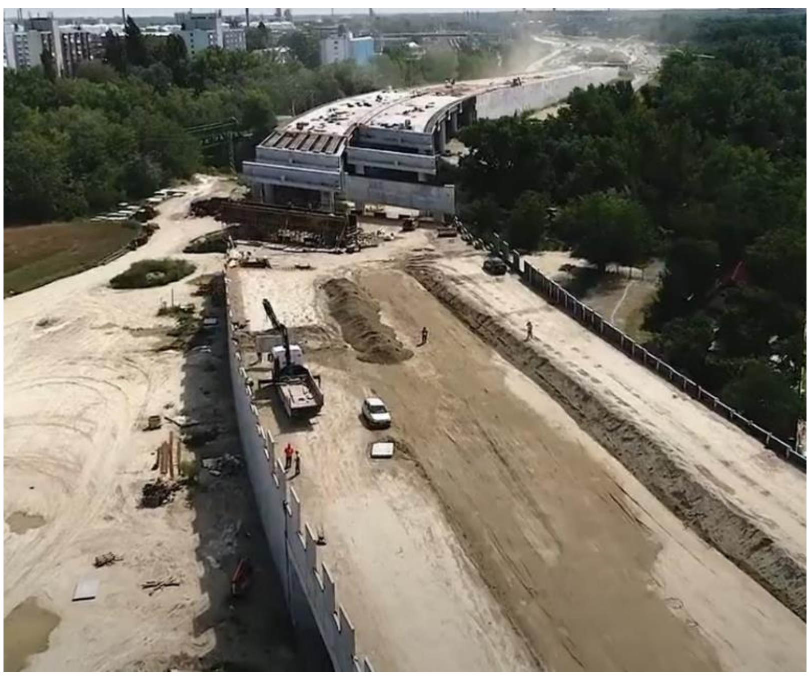
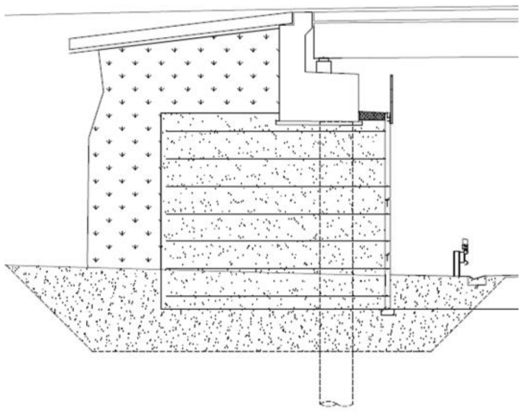


Figure 4. Back-to-back D4R7 reinforced soil walls during construction.

False (Piled) Abutments

False abutments are abutments where the bridge deck is supported on piles embedded in the reinforced soil, traversing the reinforced block and transferring the upper bridge loads to the foundation (as shown in Figure 5). This solution, including the lateral wing-walls, corresponds to 23% of the project, with 10,857 m^2 designed and executed with polymeric strip reinforcements. As previously specified, in some of these false abutment reinforced soil wall solutions, a certain depth of the foundation soil was also replaced by better quality fill material to reduce settlements.

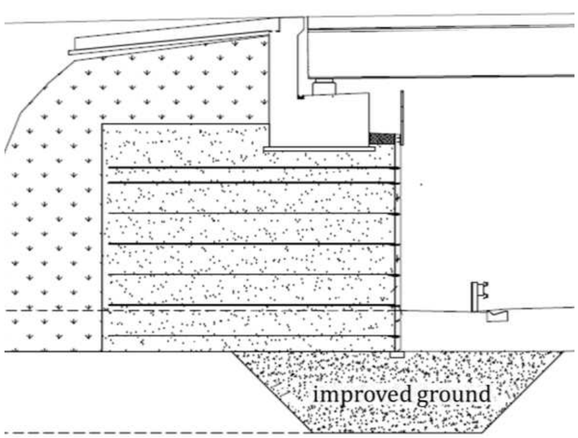


a) H = 5.4 m case b)

Figure 5. False abutment reinforced soil wall type: (a) detailed section and (b) in-construction D4R7 example case.

Abutments

Abutments are reinforced soil walls with a concrete bank seat installed above the reinforced soil block directly bearing the loads from the bridge deck to it (Figure 6). A total area of 14,125 m^2 was designed and installed. Included in this measurement are the lateral wing walls that accompany the bridge abutment (with variable wall heights, as shown in Figure 6b). The maximum wall height between all executed abutment reinforced soil walls was 8.2 m, with a reinforcement length of 9 m and a span length up to 33 m. As shown in the Figure 5a example detail, in some cases an improvement of the foundation soil below the toe of the wall was projected, with replacing a certain portion-depth of the existing foundation by select fill.



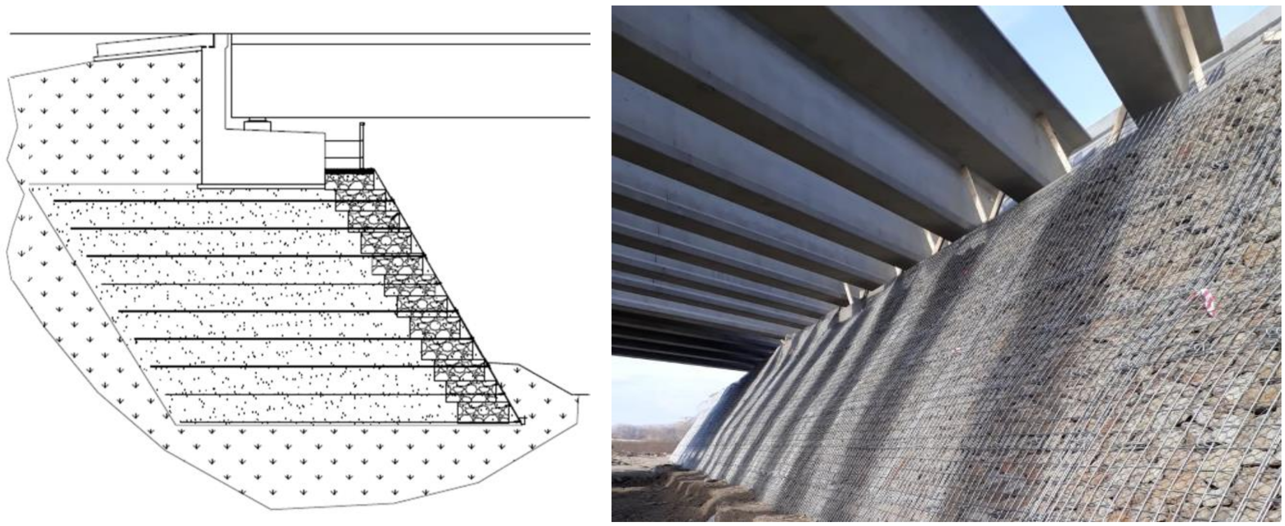
a) H = 5.3 m case b)

Figure 6. Bridge abutment reinforced soil wall type: (a) detailed section and (b) in-service D4R7 example.

The reinforcement used for the true abutments was galvanized steel ladders. However, the alternative solution with polymeric strips was also considered and projected during initial design development phases, demonstrating a good performance in terms of both ultimate limit and serviceability states. However, despite the demonstration of suitability of the polymeric reinforcement performance, this proposed solution was finally rejected by the bridge deck contractor, who was more confident in following the traditional steel reinforcement solution. Numerical sensitivity analyses and comparison results about these two solutions are presented subsequently in the numerical analysis section.

Stone Facing Walls/Slopes

A total of 1,336 m^2 reinforced soil abutments were designed and installed with welded wire mesh (WWM) facing and steel ladders reinforcements (Figure 7a). In this case, the inner 0.8 − 1.0 m (thickness) at the back of the welded wire mesh facing is filled with stones, providing a suitable and aesthetical finish (see Figure 7b). In this WWM facing case, the structure was projected with battered facing, reaching up to 30º with respect to vertical in a particular slope case. The maximum height constructed of this wall type was 7.7 m with a maximum reinforcement length of 8.5 m. The maximum span length was 33.3 m.



a) H = 6.0 m case b)

Figure 7. Bridge abutment reinforced soil slope type with WWM stone facing: (a) detailed section and (b) as-built (D4R7 Section 201-02).

Precast Panels Production, Delivery, and Assembly

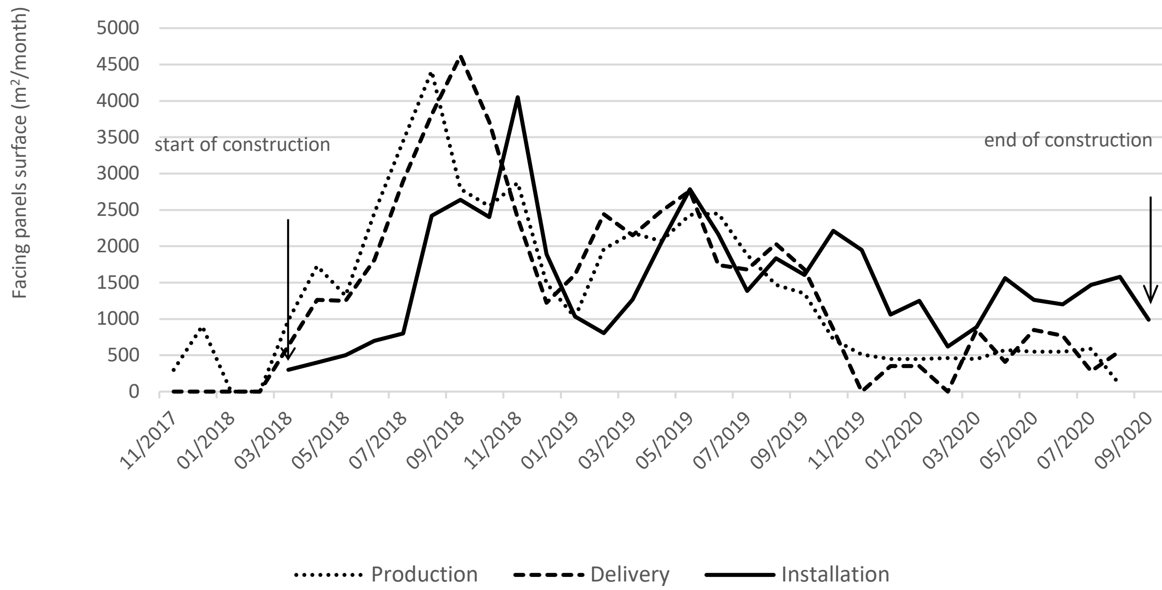
The following details refer to the precast concrete panels. The rest of the materials provided by VSL (reinforcement, equipment, accessories, etc.) were supplied directly to the job site warehouses.

Production began in November 2017 and ended in August 2020, with almost 50% of the total panels produced between July 2018 and February 2019 (see Figure 7a). To be able to supply all the volume of panels by the required deadlines, up to three pre-manufacturers were subcontracted near the job site (two in Slovakia and one in the Czech Republic) during peak production. The monthly average production reached almost 1 400 m^2, including in January and February 2018, without production due to low temperatures, and reached a total of 4,410 m^2/month of panels manufactured in August 2018.

Such a significant quantity of panels for many structures and from up to three prefabrication plants required a great effort in terms of logistics and coordination. These were completed successfully and reduced challenges associated with stock, transportation, and inclement events arising during the project. Thus, the supply of panels began in March 2018 and ended in August 2020, although 40,000 m^2 (85% of the total) were delivered on site between May 2018 and September 2019 (Figure 8a). Considering the 30 months in which the entire production was supplied, the monthly ratio was close to 1,600 m^2/month.

Regarding the installation of the structures, this first began in March 2018 with just 300 m^2, increasing monthly until reaching the maximum of 4,050 m^2/month in November 2018 (Figure 8a). The installation was practically completed in September 2020, although the remaining 150 m^2 were installed in January 2021 due to construction circumstances in one of the structures. Therefore, considering 31 months of installation, the monthly rate exceeded 1,500 m^2 per month, with high activity from August to November 2018, with an average close to 2.900 m^2/month. The historical evolution of all reinforced soil wall installation (in terms of wall facing area) is presented in Figure 8b.

a)



b)

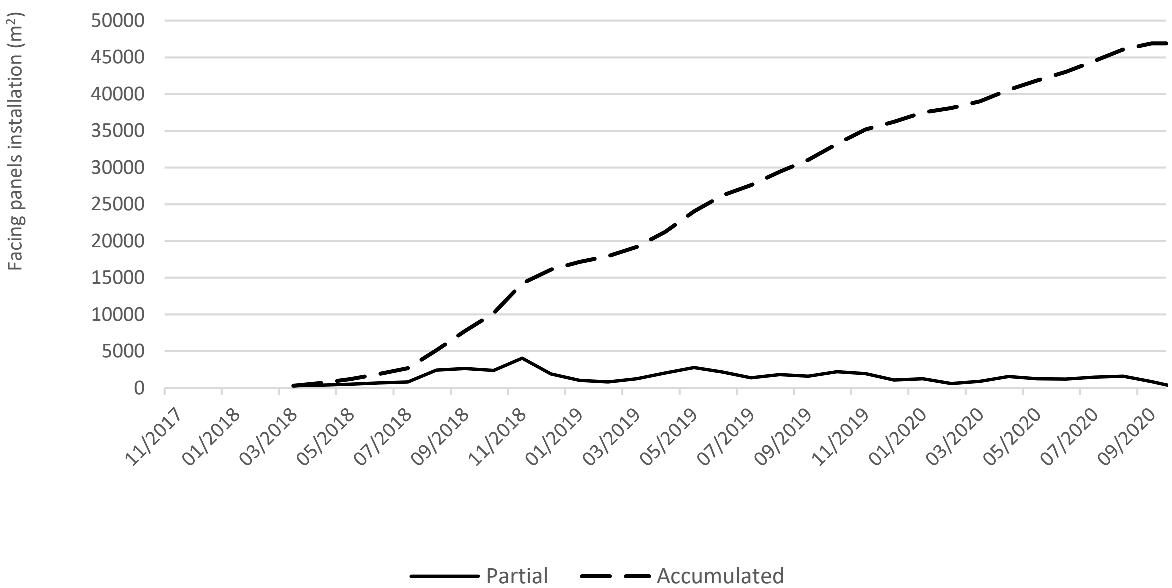


Figure 8. D4R7 Project VSoL® wall construction evolution: (a) precast panels monthly production, delivery, and installation quantities, and (b) total wall facing construction evolution.

Construction Sequences

The first step of construction is to generate a leveling pad that will serve as a flat level working surface for the erection of precast concrete facing panels. After the leveling pad has cured, the first row of panels can be erected using adequate temporary propping devices and bracing systems. The next step is to fill in and compact (achieving a soil density of 95% of the Modified Proctor) the backfill reaching the first level of connectors. After having the first layer of backfill correctly leveled and compacted, the reinforcement can be placed (polymeric strips or steel ladders). Then it is necessary to repeat the process with entwined panel placement, backfilling, and reinforcement placement until the top of the wall is reached. Finally, depending on the type of structure, the slope or bridge deck can be placed. Figure 9 shows a construction sequence for a particular reinforced soil bridge abutment structure of the D4R7 Project.

a) b) c) d) e) f)

Figure 9. D4R7 Project VSoL® reinforced soil bridge abutment construction stages: (a) 1st panel row installation, (b) backfilling, (c) polymeric strips placement, (d) frontal embedment detail, (e) frontal view, and (f) bridge deck arrangement.

FINITE ELEMENT ANALYSIS OF PARTICULAR ABUTMENT CASES

2D finite element method (FEM) analyses were carried out with PLAXIS (2008) to verify the performance of the 5-205 and 201-02 bridge abutment reinforced soil structures for the long-term (serviceability) limit state. FEM model strategies, if calibrated, have demonstrated the ability to reasonably predict the behavior of reinforced soil wall structures under different load and boundary conditions (Damians et al. 2014; 2015a; 2020). In this case, the FE model analysis was able to confirm the design validation following the British Standard 8006-1 for strengthened/reinforced soils and other fills (BSI 2016), and to verify that the structure performance was in line with the BS strain limits at serviceability states. The global wall abutment behavior, in addition with the reinforcement loads, facing displacements, and bank seat movements, were considered in order to check the wall performance and confirm the suitability of the assumed design and system component features according to the EN 14475 (BS EN 2006).

Structure 5-205: Precast Panel Reinforced Soil Wall Facing

Figure 6a presents the 5-205 cross-section considered for the FE 2D model generation. The 5-205 corresponds to a 5.3 m-high facing wall supporting a 3.2 m-high bank seat and related bridge deck loading on top, becoming a bridge abutment wall type. The reinforced soil wall is compounded by 7 polymeric reinforcement layers separated by about 0.75 m (vertical spacing). The facing consists of discrete precast concrete panels of 2.25 m (width) and 1.5 m (height) (as per project specifications detailed below), comprising 3 reinforcement connections per layer, which results in 6 strips per layer (i.e., continuous V-shape strips unrolling).

Figure 10 presents the main model dimensions, the FE model mesh details, and the model boundary conditions assumed for the base case (Figure 10a) and the additional geometry features used for final—and probably more illustrative—performance representation (Figure 10b). Surcharge loading features (with magnitude and location detail) are presented in Figure 11.

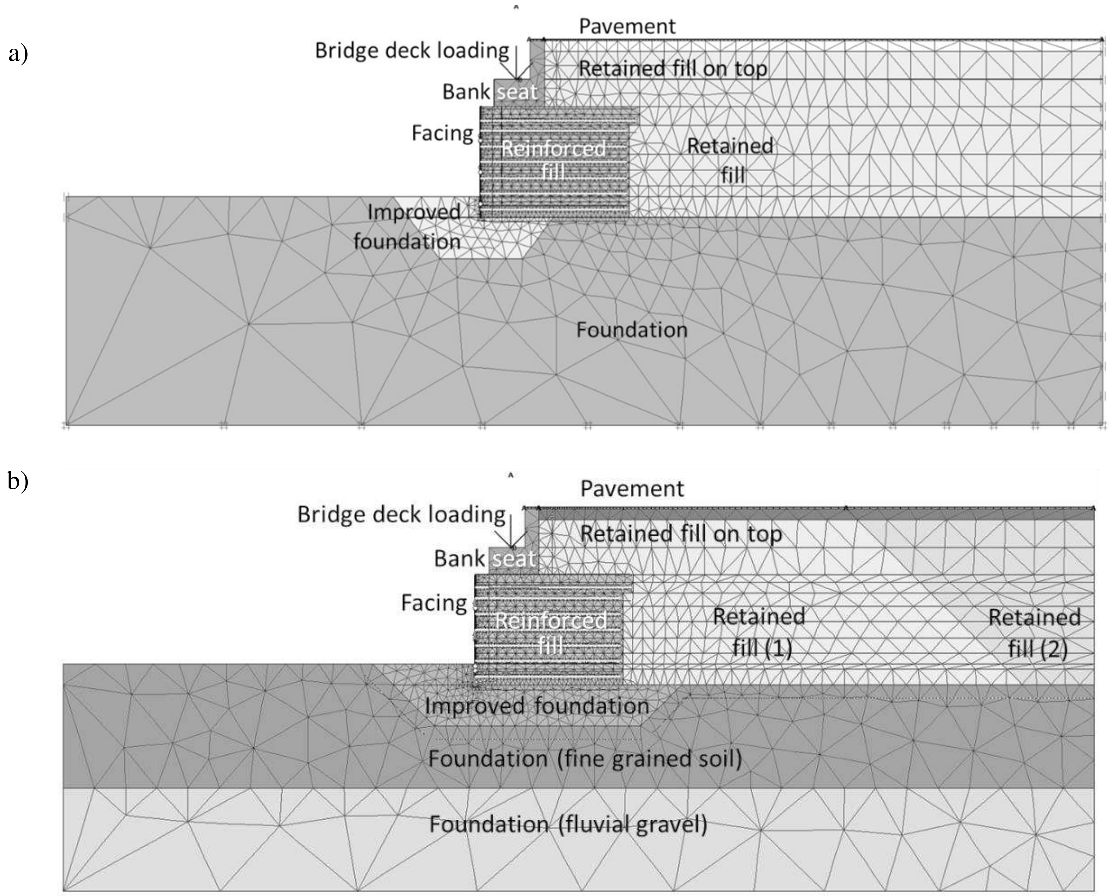


Figure 10. FE 2D model mesh generated: (a) Base case and (b) additional cases geometry.

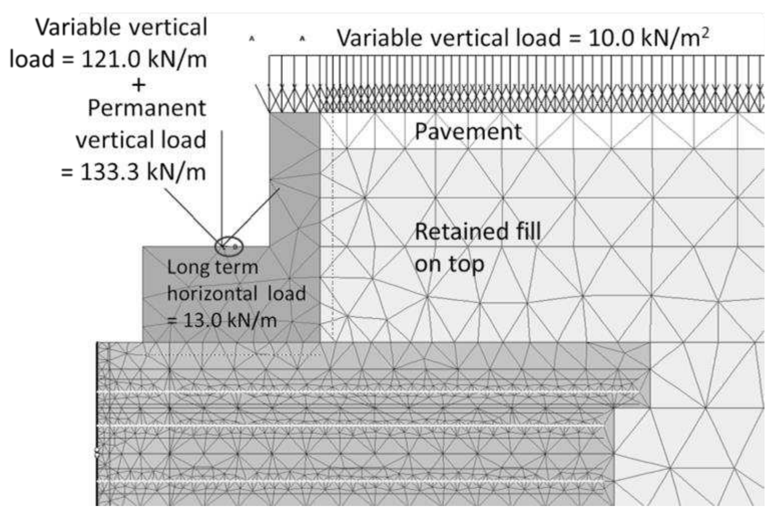


Figure 11. FE 2D model: bank seat loading features detail.

Table 2 shows the initial soil material properties (base case). Soil materials were modeled assuming linear elastic-plastic behavior with Mohr-Coulomb failure criterion, with the cut-off tensions option (no tensile load development allowed). Structural components were modeled as linear elastic. Soil strength properties are assumed as being under triaxial conditions. This is a conservative assumption, as more realistic soil properties to be numerically analyzed would be the actual plane strain conditions (Bathurst and Hatami, 2006). The ratio between the peak friction angle from triaxial (ϕ_{tx}) and plane strain (ϕ_{ps}) tests is usually from about 1.12 to 1.2 (Kulhawy and Mayne, 1990). However, in the current study, a conservative triaxial condition was assumed to affect all backfill materials.

Table 2. Base case soil material properties: elastic-plastic Mohr-Coulomb failure criterion.

Parameters [a]:	Reinforced fill: <1 m from facing [b]	Reinforced fill: >1 m from facing	Retained fill	Foundation: Fluvial gravel	Foundation: Improved zone: soil exchange [c]	Pavement
Unit weight, γ (kN/m^3)	20		20	20		25
Friction angle, ϕ_{tx} (deg.)	35		35	30	35	35
Cohesion, c (kPa) [d]	0.1		1	5	1	35
Dilatancy angle, ψ (deg.)	5		5	0	5	5
Elastic modulus, E (MPa)	50	100	50	50	100	50
Poisson's ratio, ν (-)	0.3		0.3	0.3		0.3

[a] all soil materials assumed to be non-tension materials (i.e., cut-off tension);
[b] less stiffness assumed at reinforced soil near the facing to take into account light compaction equipment efforts at those locations;
[c] improved foundation zone under the facing as per figure detail;
[d] no zero cohesion values are assumed to avoid numerical instability (however, non-tension soil performance is assumed as previously specified).

Tables 3 and 4 present the facing (panels and elastomeric bearing pads, modeled with a hinge connection between them) and reinforcement component properties, respectively. All structural material properties (i.e., facing components, reinforcements, as well as the bank seat and related elastomeric bearings) had to be properly transformed from a 3D to 2D plane strain model representation.

Table 3. Base case model properties of facing "plate" elements: linear elastic.

Parameters	Precast concrete panels [a]	Bearing pads (HDPE) [b,c]
Axial stiffness, EA (MN/m)	4800	0.8
Bending stiffness, EI (kN/m^2/m)	9000	1.4
Poisson's ratio, ν (-)	0.15	0.4

[a] based on 2.25 m-width × 0.14 m-thick concrete panel geometry, where the modeled plate elements thickness is defined as $\sqrt{12EI/EA}$;
[b] 2 units per 2.25 m panel joint;
[c] 20 mm-thick pads.

Table 4. Base case axial stiffness of reinforcement "geogrid" elements: polymeric strips.

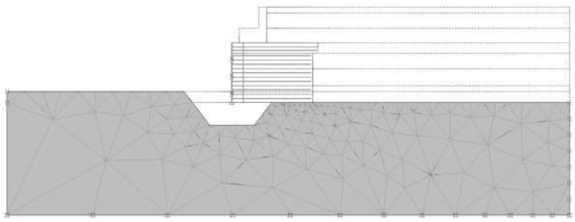
Reinforcement layers	Strip grades (T_{char})	Connections per 2.25 m-width panel	Axial stiffness [a] (kN/m)
7th (top layer)	70	3	1400.0
1st (bottom layer) to 6th	63	3	1555.6

[a] short-term characteristic reinforcement strength (T_{char}) reached linearly at about 0.10-0.12 strain for all strip grades (Damians et al., 2015b).

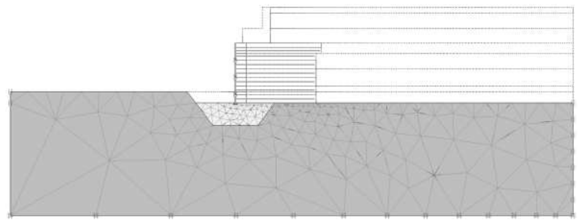
Regarding the reinforcement strips' polymeric stiffness, it is worth mentioning that with linear elastic constitutive criteria, because the stress-strain curve of such materials is not purely linear even in short-term performance tests (stiffer at low strains), a more accurate value can be obtained through an iterative process from the obtained results, with a proper selection of a representative stiffness after calculating the achieved maximum reinforcement strains of each layer.

Staged construction was assumed in model development to match field stress-strain performance conditions (18 steps) more realistically. Figure 12 presents the construction stages up to the bank seat installation (12 steps). Figure 13 presents construction stages after bank seat installation (post-construction stages; 6 steps).

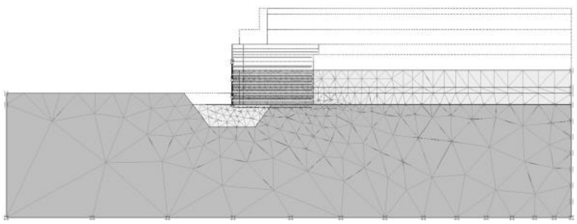
a) Excavation:



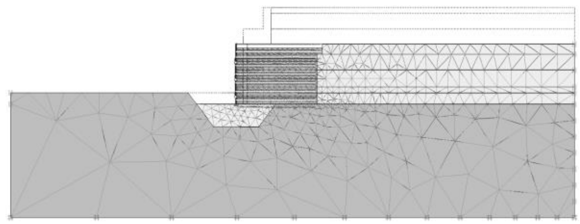
b) Improved foundation zone (gravel):



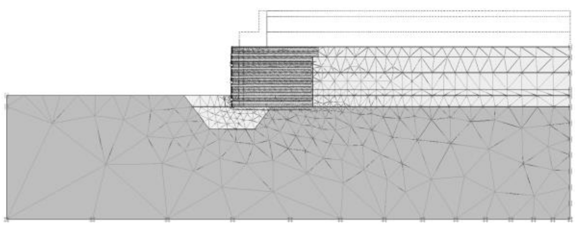
c) 1st to 4th reinforcement layers (4 stages):



d) 5th to 7th reinforcement layers (3 stages):



e) Top of the wall and facing embedment (2 stages):



f) Bank seat installation:

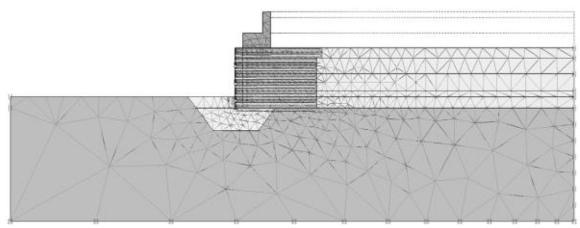
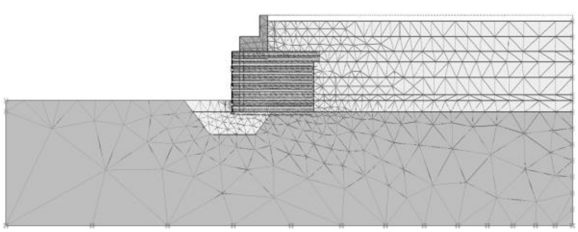
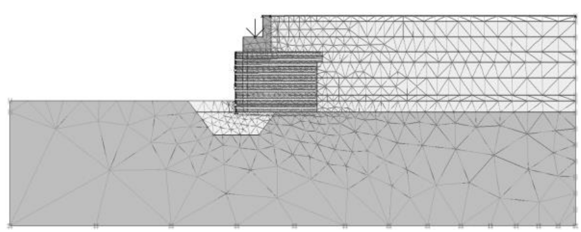


Figure 12. Construction stages up to bank seat installation.

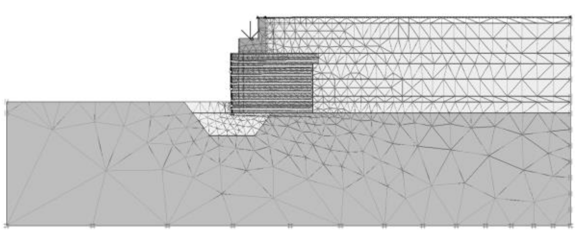
a) Retained fill layers on top (2 stages):



b) Bridge deck (133 kN/m permanent load surcharge at bank seat bearing center) and pavement (2 stages):



c) Live loads (121.0 kN/m variable vertical load application at the bank seat bearing center, and 10.0 kN/m load on pavement surface):



d) Live loads (13.0 kN/m long-term horizontal load applied at the bank seat bearing center):

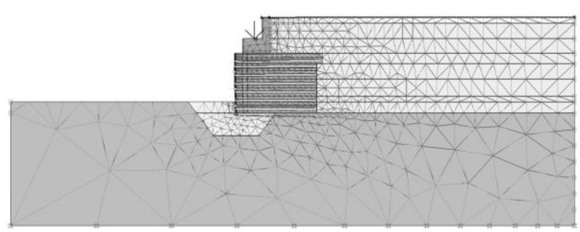


Figure 13. Construction stages after bank seat installation.

Precast Panel Reinforced Soil Wall Model Deformation

Figure 14 shows the post-construction model deformation and shear strains' development after permanent and variable surcharge load application on the bank seat bearing center and on top of the retained fill/pavement (serviceability stage). As it is also confirmed in next section, the shear strains generated within the reinforced soil (Figure 14b) start from the facing toe at the bottom reinforcement layer up to the edging-corner of the bank seat at the top layer. This is in line with the theoretical location of the maximum tension loads through the reinforcement layers (BSI 2016).

a) Deformed mesh (amplification factor: ×10): b) Shear strains (ranging from 0 to 5%):

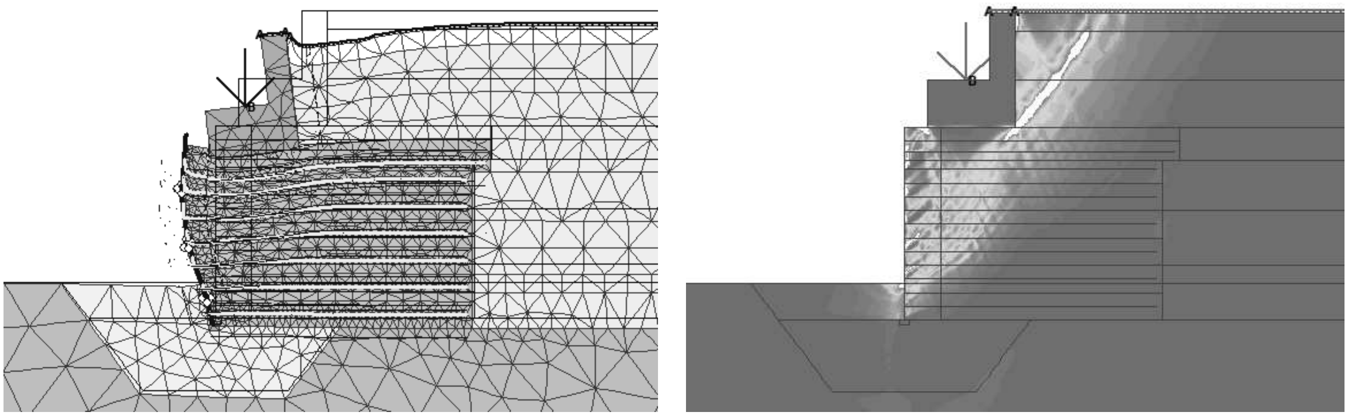


Figure 14. Deformed mesh (a) and shear strains (b) at post-construction serviceability stages. Base case.

Precast Panel Reinforced Soil Wall Reinforcement Loads

Figure 15 presents the reinforcement axial/tensile load distribution for each reinforcement layer. The average reinforcement axial loads at the end of construction (EoC) and at the serviceability stages are presented. About 43 kN/m is the highest maximum tensile load value developed at the serviceability stage (see 3rd, 4th, and 5th reinforcement layers). This value relates to about T_{max} = 16 kN/strip (i.e., 43 kN/m × 2.25 m-panel width / 6 reinforcement strips per panel). Considering the characteristic short-term tensile load (T_{char}) of the strip reinforcement grades at those layers (i.e., T_{char} = 63 kN) and the required factor which takes into account the long-term creep reduction for serviceability limit state ($RF_{CR(CLC)}$ = 2.0), the related tensile load for design becomes T_{CS} = 31.5 kN. Thus, the resultant maximum tensile load from the model results in a capacity demand ratio of about 2 (i.e., 31.5 kN / 16 kN = T_{CS} / T_{max}), confirming the conservative reinforcement design. The obtained post-construction strain limits between both scenarios (i.e., the EoC and serviceability stages) are also included for each reinforcement layer.

Considering the reinforcement layer developing the highest post-construction strain values (see 5th and 6th reinforcement layers, at 3.415 m and 4.185 m elevation, respectively), about $\Delta\varepsilon$ = 0.4% axial strain was reached. British Standard BS 8006-1 (BSI 2016) prescribes the post-construction strain limit as $\Delta\varepsilon^*$ = 0.5% for bridge abutment cases under permanent structural loading. Thus, the model is in line with this value (and on the safe side) for serviceability limit states, with a related coverage demand ratio of 1.2. As already noted in Figure 14, the geometric-theoretical location of the maximum reinforcement tension loads are in agreement with the shear strain development from the facing toe up to the corner of the bank seat on top of the reinforced fill (i.e., at about 3 m at the top surface, which relates to about 2.5 m at the 7th reinforcement layer).

Precast Panel Reinforced Soil Wall Additional Cases and Facing Displacements Comparison

To analyze the different foundation properties and reinforcement stiffness conditions, additional cases have been analyzed in order to clarify both the facing and bank seat bearing point displacements at post-construction stages (i.e., bank seat installation, retained fill on top placement, bridge deck surcharge, and live load application). These additional cases correspond to the following:

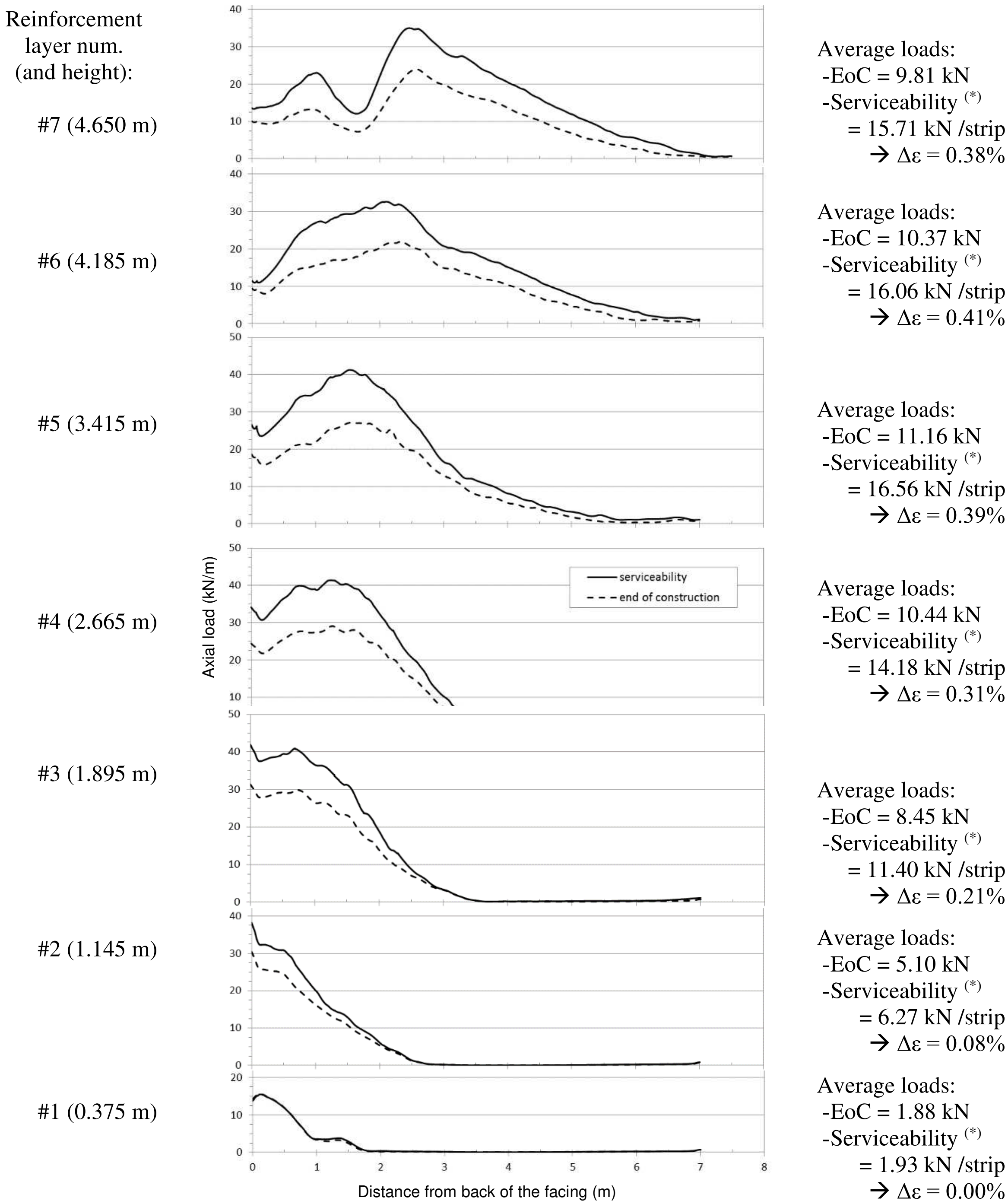


Figure 15. Reinforcement axial load (kN/m) distribution at the end of construction and under serviceability long-term scenario ((*) 100% of the LL's application). Base case.

i) *Modified soil properties*. This case represents the more realistic/original scenario, including an additional foundation soft layer, enlarged dimension of the foundation gravel - improved region (see Table 5), and maintaining the strip grades of the original case (i.e., reinforcement Grade 63 kN, with the exception of Grade 70 kN at the top layer). The improved zone materials were assumed also with a higher (more realistic) frictional strength, and the retaining fill zone was divided to take into account construction backfilling.

ii) *Polymeric strip reinforcement with increased strength*. In this case, Grade 100 kN was assumed in all layers, with the same modified soil properties than in the previous case (i).

iii) *Steel reinforcement* (also assuming the modified soil properties from Table 5), in order to identify a limit case in terms of wall deformability. The steel reinforcement (assumed as a purely inextensible reinforcement) was modeled with Young's modulus of 200 GPa, and thus predicts very small facing outwarddisplacements.

Table 5. Modified soil material properties.

Parameters:	Reinforced fill <1 m from facing	Reinforced fill >1 m from facing	Retained fill (1)	Retained fill (2)	Foundation: Fluvial gravel	Foundation: Fine grained soil	Foundation: Improved zone: soil exchange	Pavement
Unit weight, γ (kN/m^3)	20		20		20	20	20	25
Friction angle, ϕ_{tx} (deg.)	35		35		30	25	45	35
Cohesion, c (kPa)	0.1		1		0.1	5	0.1	35
Dilatancy angle, ψ (deg.)	5		5		0	0	15	5
Elastic modulus, E (MPa)	50	100	50	30	50	10	100	50
Poisson's ratio, v (-)	0.3		0.3		0.3	0.3	0.3	0.3

Figure 16a presents the facing outward phase displacement at post-construction stages for the base case. The construction stage generates a high outward displacement corresponding to the live load surcharge application, with about 2.5 mm horizontal displacement. It should be noted that the straight discontinuities generated at about 0.75 m, 2.2 m, and 3.75 m elevation are related to the panels' joint locations where the 20 mm-thick bearing pads are placed, and slight displacement and rotation movements are allowed as in the real facing case.

Figures 16b-d presents the facing outward displacements at post-construction wall stages for the additional three cases analyzed (i-iii). The modified soil strength and stiffness properties case results in poorer performance of the wall behavior, with about 3 cm (+43%) mismatch of the facing displacement at the top compared to the original case, reaching about 10 cm outward maximum displacement at the top of the wall. As shown in Figure 16c, the increase of the polymeric strip strength to Grade 100 kN (i.e., increasing the capacity on the reinforcement layers of the base case by 40 − 60%) reduces displacements in a global manner, falling into the same range than in the original case with non-modified soil strength properties (about 7 cm maximum outward displacement at the top of the wall facing). It can be concluded that the consideration of a worse soil material is compensated by an increase in the reinforcement strip stiffness. As seen (and expected), clearly minor displacements are obtained for the steel reinforcement case, reaching a maximum outward displacement value of 2.3 cm at the facing top.

Precast Panel Reinforced Soil Wall Bank Seat Displacements

Figure 17 presents the displacements obtained at the bank seat bearing center point (i.e., where modeled loads are applied; see Figure 11 detail) at post-construction wall stages, in addition with the displacements of the facing toe (i.e., at the bottom of the first panel; in hinge contact with the modeled leveling pad). Therefore, the displacements can be separated with regards to the total and relative displacements with or without considering the facing toe movement. Both vertical (i.e., settlement) and horizontal outward displacements are generated during a permanent and variable surcharge loading application after/above bank seat construction. Horizontal displacements are in line with the ones previously presented at facing. Again, the live load application generated the higher horizontal displacement value; however, the larger settlement value is reached due to the bridge deck loading application.

The modified soil properties of the additional cases (i-iii, with lower foundation stiffness) increase the facing toe displacements: about 1 cm-outward and 2.3 cm-settlement in the 3 cases. The results reached at the base case were 0.25 cm-outward and 0.8 cm-settlement due to the stiffer foundation, significantly lower than the modified case. According to this, the relative final outward-settlement displacements (i.e., taking into account the facing toe displacements) obtained at the bank seat bearing center point are about 9 cm, 6 cm, and 1.5 cm for the modified soil properties, polymeric strip Grade 100 and steel reinforcement cases, respectively (Figures 17b-d). The relative vertical displacements values (i.e., settlement) reached are about 5.5 cm, 3.5 cm, and 1 cm for the modified soil properties, polymeric strip Grade 100, and steel reinforcement cases, respectively.

The total maximum displacements at the bank seat bearing point for each case are (horizontal/vertical): a) 6/4 cm, b) 10/8 cm, c) 7/6 cm, and d) 2.7/3.5 cm. These displacement values may be significantly reduced if movements due to the retained backfill on top are not considered, because this does not affect the deck joint closure (the deck would be installed after this movement happens). The bank seat can be realistically poured in a different position considering this movement. By doing this, the differential movements between the bank seat and the bridge deck due to the retained backfill on top may be neglected (1.0/1.0 cm, 1.2/1.8 cm, 0.9/1.6 cm, and 0.0/1.2 cm for the 4 cases analyzed). Furthermore, it can also be seen how the contribution of the Live Load to the total displacements is quite important (the dashed line in previous Figure 17). However, in the Serviceability Limit State (SLS), the displacement evaluation does not consider 100% of the live load, but rather reduces it with a combination factor. This factor varies from 0.00 to 0.75, depending on the load type (point or distributed) and load combination (characteristic, frequent, or quasi-permanent). According to the BS 8006-1, only dead loads without partial load factors are considered for SLS (Combination C; BSI 2016). Thus, by applying a factor (say, 67%) to the Live Load application, a more realistic (and still conservative) theoretical movement of the bank seat could be obtained.

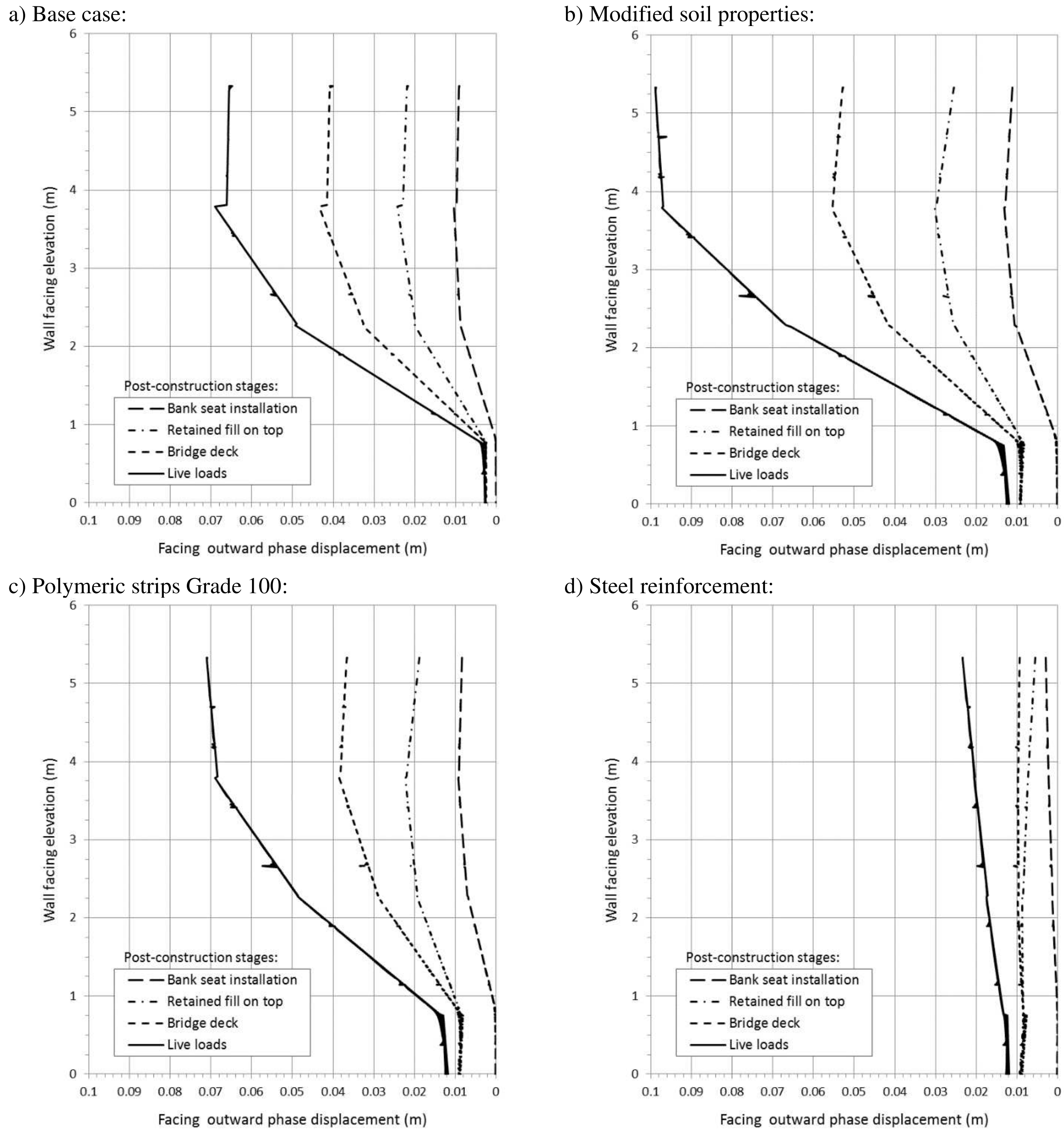


Figure 16. Facing outward displacements at post-construction wall stages: (a) base case, (b) modified foundation soil properties, (c) assuming polymeric strips Grade 100, and (d) assuming steel reinforcement case. Note: (c) and (d) cases also assume modified soil properties as in b-case.

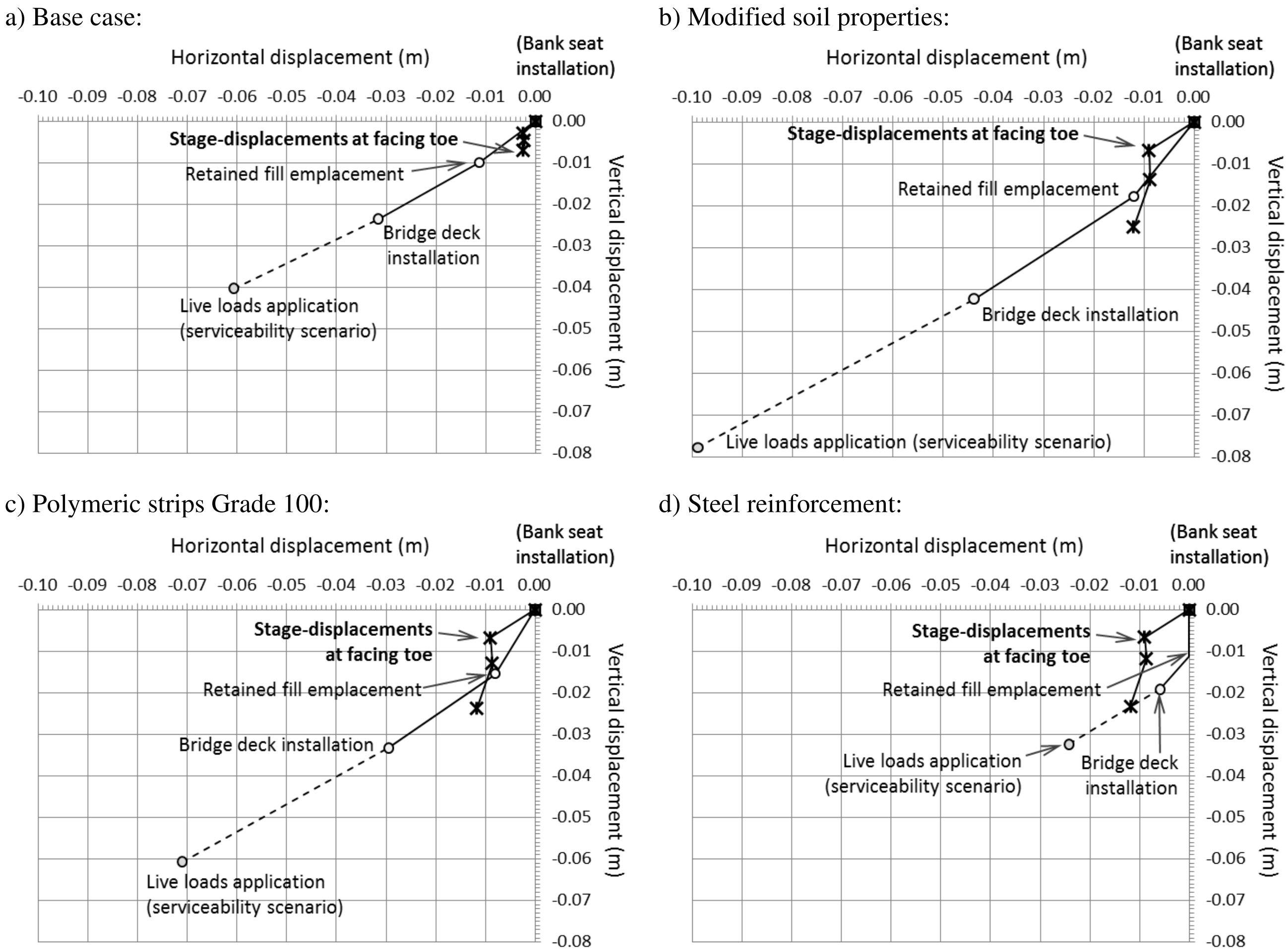


Figure 17. Displacements of the bank seat bearing center point at post-construction wall stages: (a) base case, (b) modified foundation soil properties, (c) assuming polymeric strips Grade 100, and (d) assuming steel reinforcement case. Note: (c) and (d) cases also assume modified soil properties as in b-case.

STRUCTURE 201-02: Welded Wire Mesh (WWM) Facing Slope

The 201-02 cross-section shown in Figure 6a was used for the FEM 2D model generation. The 201-02 is a bridge abutment type that has 6 m-high wall-slope supporting a 2.9 m-wide × 3.5 m-high bank seat and related bridge deck loadings on top. The reinforced soil slope is compounded by 8th steel ladder reinforcement layers, each with 0.75 m vertical spacing and a length of 8 m. The facing is formed by discrete L-shaped welded wire mesh units that are 2.40 m wide (0.15 m horizontally overlapped) and about 0.90 m high (0.2 m vertically overlapped). The wire mesh is composed by 8 mm and 12 mm nominal wire diameter in horizontal-vertical directions, with center-to-center distances in both directions of 100 mm and 150 mm. Each facing unit (i.e., 2.25 m wide) entails 4 steel ladder units with a 10 mm bar-diameter. Coarse granular material is included just behind the facing WWM units, generating a stone facing wall type. Figure 18a presents the main model dimensions, the FE mesh detail, and the model boundary conditions assumed. Surcharge loading features (magnitude and location detail) are presented in Figure 18b.

Table 6 lists the soil material properties, and Table 7 lists the facing WWM units and steel ladder reinforcement component properties. Soil materials were modeled assuming linear elastic-plastic behavior with Mohr-Coulomb failure criterion, with a cut-off tensions option (no tensile loads development allowed). Structural facing and soil reinforcement components were modeled as linear elastic. Similar to the previous case, staged construction was assumed in model development to more realistically match stress-strain performance site conditions (18 steps). See previous Figures 12 and 13 for reference.

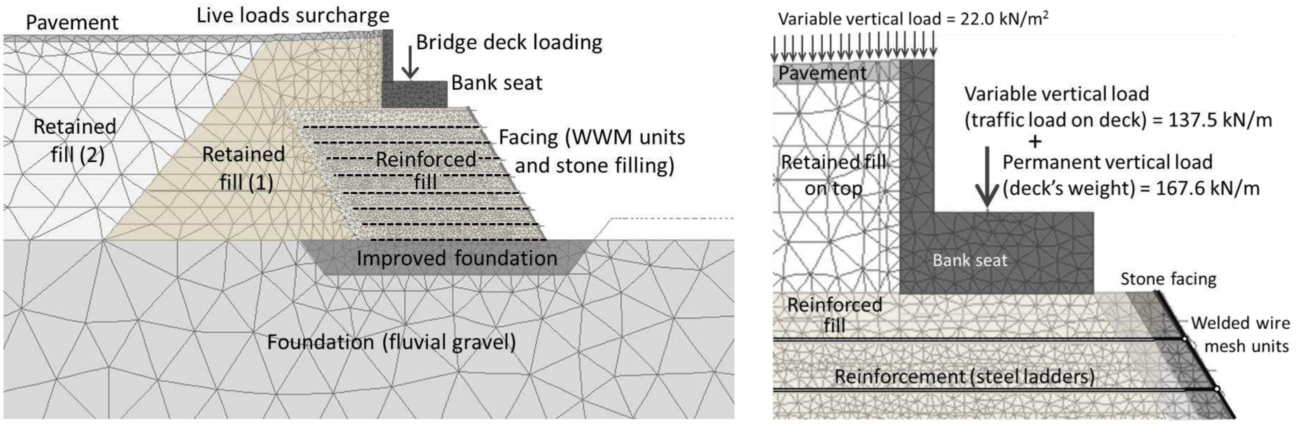


Figure 18. FEM 2D model mesh generated: (a) general model, and (b) bank seat with loading features detail.

WWM Model Deformation and Facing Displacement Results

Figure 19 shows the post-construction model deformation and shear strains development after a permanent and variable surcharge load application on the bank seat bearing center and on top of the retained fill/pavement (serviceability stage). Figure 20 presents the facing outward phase displacement at post-construction stages. As can be observed, the construction stage generating the high outward displacement corresponds to the live load surcharge application (frequent load scenario, corresponding to 67% of the LL's application), with about 2.0 mm horizontal staged displacement.

Table 6. Soil material properties [(a)]: elastic-plastic Mohr-Coulomb failure criterion.

	Reinforced fill		Stone facing	Retained fill		Foundation		Pavement
Parameters:	<1 m from facing [(b)]	>1 m from facing		(1)	(2)	Fluvial gravel	Improved: soil exchange [(c)]	
Unit weight, γ (kN/m^3)	20		20	20		20	20	25
Friction angle, ϕ_{tx} (deg.)	35		45	35		35	38	35
Cohesion, c (kPa) [(d)]	1		1	1		0.1	0.1	5
Dilatancy angle, ψ (deg.)	5		15	5		5	8	5
Elastic modulus, E (MPa)	50	100	100	50	30	50	100	35
Poisson's ratio, ν (-)	0.3		0.3	0.3		0.3	0.3	0.3

[(a)] all soil materials assumed to be non-tension materials (i.e., cut-off tension);
[(b)] less stiffness assumed at reinforced soil near the facing to take into account light compaction equipment efforts at those locations;
[(c)] improved foundation zone under the facing as per figure detail;
[(d)] no zero cohesion values are assumed to avoid numerical instability (however, non-tension soil performance is assumed as previously specified).

Table ***7****. Model properties of facing and reinforcement elements: Linear elastic.*

Parameters	Welded wire mesh facing [(a)]	Steel ladders [(b,c)]
Axial stiffness (MN/m)	158.3	44.8
Bending stiffness (MNm^2/m)	1.4×10^{-3}	-
Poisson's ratio (-)	0.2	-

[(a)] equivalent properties from actual L-shape welded wire mesh facing unit materials and related bar diameters/spacing;
[(b)] equivalent properties assuming 8.74 mm-diameter − 4 steel ladder units each 2.25 m facing column-width (i.e., 3.56 longitudinal bar units each 2D plane-strain model meter);
[(c)] axial stiffness according to 8.74 mm bar-diameter, which corresponds to 10 – 1.26 mm-diameter due to the sacrificial thickness required for service Limit States 100 years design.

a) Deformed mesh (amplification factor: ×200): b) Shear strains (ranging from 0 to 0.5%):

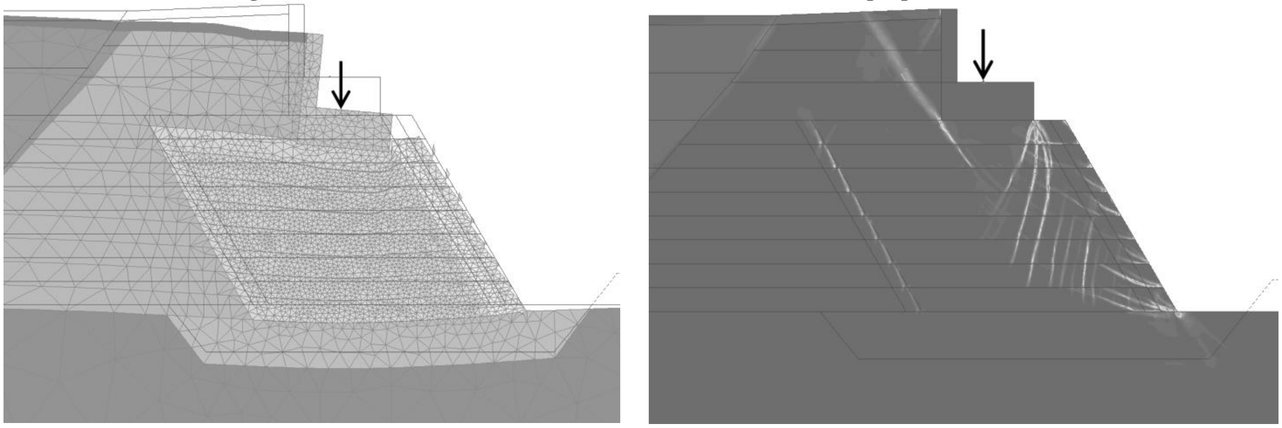


Figure 19. Deformed mesh (a) at bridge deck construction stage and (b) shear strains at frequent loads combination scenario.

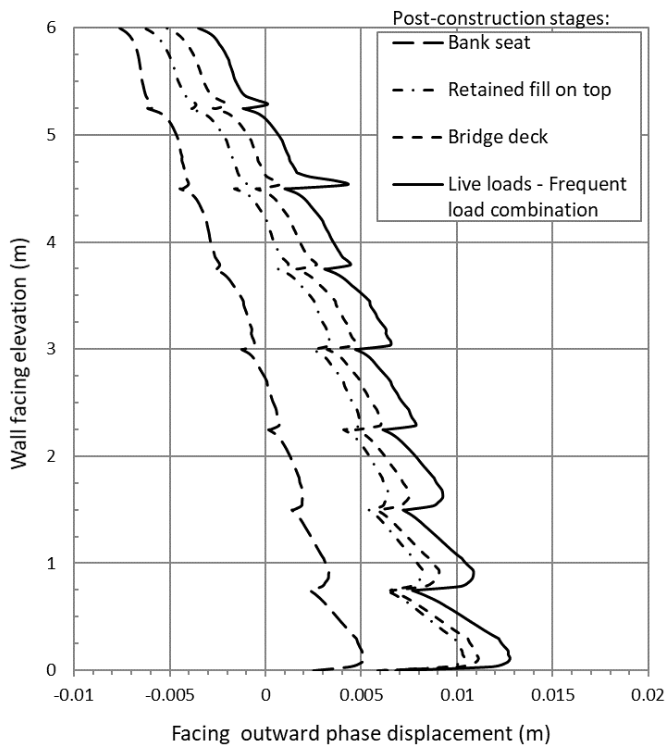


Figure 20. Facing outward displacements at post-construction wall stages.

WWM Bank Seat Displacement Results

Figure 21 presents the bank seat bearing point movement at the post-construction stages. As noticed, both vertical (i.e., settlement) and horizontal displacements are generated during permanent and variable surcharge loading applications after/above the bank seat construction and top retained fill emplacement behind. Horizontal displacements are in line with the ones previously presented at facing. As previously identified, the live load application generated the higher horizontal displacement value; however, despite the characteristic load application (100% of the LL's), the larger settlement value is reached due to the bridge deck loading application.

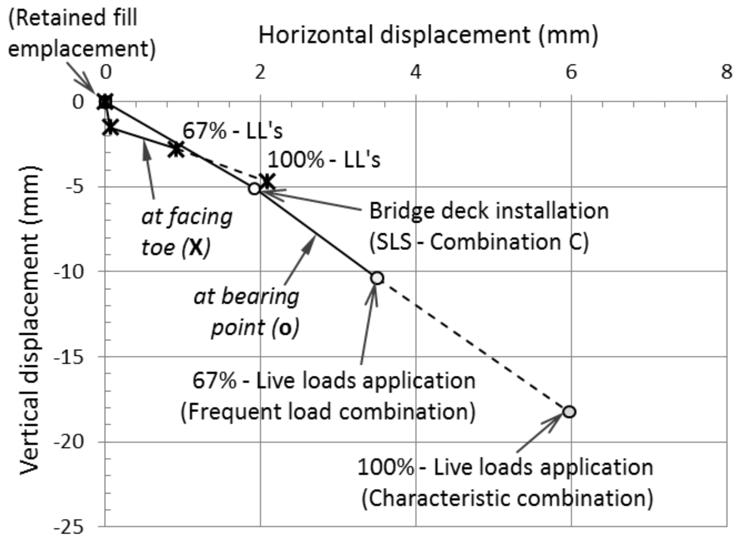


Figure 21. Displacements of the bank seat bearing center at post-construction wall stages.

CONCLUSIONS

The study presented the materials/components required and the panel manufacturer/supplier for the D4R7 Project, which involved about 60 VSoL® system reinforced soil wall structures with a total of 47,100 m^2 of wall facing area. The required combination of logistics and coordination is presented as being successful and did result in project completion while addressing challenges related to stock, transportation, and inclement events arising during the project, with significant production and installation monthly ratios thanks to all involved elements.

The finite element analysis results demonstrate that the design of the bridge abutment reinforced soil wall met the British Standard 8006-1 requirements. Different scenarios regarding the reinforcement stiffness as well as foundation properties were assumed for the reinforced soil precast facing abutment case, confirming the expected response: the stiffer the reinforcements are, the lower facing movement is, compensating in some cases for the soft conditions of the foundation properties. The calculated movements of the facing and the bank seat bearing point showed that the higher effect (i.e., higher displacement) is due to the serviceability loading scenario assumptions (variable surcharge loading application). However, this loading case is probably too conservative and not realistic, as it assumes that the variable loading is applied on the entire bridge deck and also above the retaining fill/pavement behind/above the abutment. The analyses performed for the welded wire mesh facing reinforced soil abutment type demonstrated good performance of the solution, assuming a factor application for the live loads of 0.67. As shown, the designed structure satisfies the BS 8006-1 requirements for Serviceability Limit States (SLS - Combination C: only dead load application), being below the prescribed post-construction strain limits given in the code. Furthermore, the resulting reinforcement tensile loading distribution was conservative with enough coverage demand ratios even for the most critical reinforcement layers. According to this, the methodology and parameters selected in the design process can be stated to be appropriate and sufficiently conservative.

REFERENCES

Bathurst R.J., and Hatami K. (2006). "Physical to computational modeling of reinforced soil walls under static loading." *International Conference on Physical Modeling in Geotechnics*, ICPMG 2006, Hong Kong, China.

British Standard Institution (BSI). (2016). *Code of practice for strengthened/reinforced soils and other fills (BS 8006-1:2010+A1:2016)*, Milton Keynes, UK.

British Standard Institution (BSI). (2006). *Execution of special geotechnical works - Reinforced fill (BS EN 14475 (2006))*, European Committee for Standardisation.

Damians, I.P., Bathurst, R.J., Olivella, S., Josa, A., and, Lloret, A. (2020). "3D modeling of strip reinforced MSE walls." *Acta Geotechnica*.

Damians, I.P., Bathurst, R.J., Lloret, A., Josa, A., and, El Mansouri, D. (2019). "Assessment of earth retaining wall sustainability: Value functions and stakeholder weighting sensitivity." *Proceedings of the 3rd GeoMEast - International Congress and Exhibition*, Egypt 2019 on Sustainable Civil Infrastructures. Cairo, Egypt.

Damians, I.P., Bathurst, R.J Adroguer E., Josa, A., and, Lloret, A. (2018). "Sustainability assessment of earth retaining wall structures." *ICE Environmental Geotechnics*.

Damians, I.P., Bathurst, R.J Adroguer E., Josa, A., and, Lloret, A. (2017a). "Environmental assessment of earth retaining wall structures." *ICE Environmental Geotechnics*.

Damians, I.P., Bathurst, R.J., Josa, A., Lloret, A., and, Lima J. (2017b). "Analysis of vertical facing panel-joint gap in reinforced soil walls." *Proceedings of the 3rd African Regional Conference on Geosynthetics (GeoAfrica 2017)*. Marrakech, Morocco.

Damians, I.P., Bathurst, R.J., Josa, A. and, Lloret, A. (2016). "Vertical facing panel-joint gap analysis for steel-reinforced soil walls." *ASCE International Journal of Geomechanics*.

Damians, I.P. (2016). *Mechanical performance and sustainability assessment of reinforced soil walls*, Ph.D. Thesis, School of Civil Engineering (ETSECCPB), Department of Civil and Environmeantl engineering (DECA), Universitat Politècnica de Catalunya·BarcelonaTech (UPC).

Damians, I.P., Bathurst, R.J., Josa, A. and, Lloret, A. (2015a). "Numerical analysis of an instrumented steel reinforced soil wall." *ASCE International Journal of Geomechanics,* 15(1), 04014037.

Damians, I.P., Bathurst, R.J., Lima J., Lloret, A., and, Josa, A. (2015b). "Numerical study of the use of actively-tensioned polymeric strips for reinforced soil walls." *Proceedings of the XVI European Conference on Soil Mechanics and Geotechnical Engineering (XVI ECSMGE)*, Geotechnical Engineering for Infrastructure and Development. Edinburgh, Scotland, 3833-3838.

Damians, I.P. Bathurst, R.J., Josa, A. and, Lloret, A. (2014). "Numerical study of the influence of foundation compressibility and reinforcement stiffness on the behavior of reinforced soil walls." *International Journal of Geotechnical Engineering*, 8(3), 247-259.

Damians, I.P., Bathurst, R.J., Josa, A., Lloret, A. and, Albuquerque, P.J.R. (2013). "Vertical facing loads in steel reinforced soil walls." *ASCE Journal of Geotechnical and Geoenvironmental Engineering,* 139(9), 1419-1432.

D4R7 (2017). D4R7 Construction s.r.o. Zero Bypass Limited, Odborárska 21, 831 02 Bratislava, Slovakia.

International Organization for Standardization (ISO). (2009). *ISO 1461. Hot dip galvanized coatings on fabricated iron and steel articles - Specifications and test* methods. (ISO/TC107/SC4), Vernier, Geneva, Switzerland.

Kulhawy F.H., and, Mayne P.W. (1990). *Manual on estimating soil properties for foundation design. Report EL-6800*, Electric Power Research Institute (EPRI). Palo Alto, California.

PLAXIS (2008). *Reference Manual, 2D: Version 9.02. PLAXIS B.V.,* Delft University of Technology, Netherlands.

*Vorspann System Losinger (*VSL International). (2021). *VSoL® Retained earth Walls*, Bouygues Construction.