Design of Road Embankments Constructed Over Existing Underground Structures

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Abstract:

Due to the development of new highways in metropolitan areas, construction of the road embankments over existing underground structures is inevitable. Since most of these buried structures are old and assessing the structural integrity is not simple, most project agreements allow no additional stresses on the existing underground structures. In this situation, precise stress and deformation analysis are required to evaluate the effect of the construction of the new embankments and the effectiveness of the stress reduction measures on the existing underground structures. In this study the use of numerical modeling to analyze and design of road embankments constructed over existing underground structures is investigated. Detailed 2D finite difference analyses were conducted by means of FLAC in order to determine the efficiency of these stress-deformation analyses. Since the use of expanded polystyrene (EPS) blocks in road embankments is one of the most common methods to reduce the loads on existing underground structures and also the construction time, a light weight embankment in the new Champlain Bridge Corridor project was considered as a case study to evaluate the effectiveness of the proposed analysis approach in design optimization. The results of the analysis show that using numerical modeling with advanced soil models can effectively optimize the design of the lightweight road embankment and significantly reduces the required EPS volume and consequently the construction costs. The outcome of this study provides new insights into the design and construction of road embankments over the existing underground structures in metropolitan areas.

1 INTRODUCTION

Construction of new highways or expanding the existing ones in urban areas due to growing need of more transportation network is rising. One of the challenges in design and build of new road embankments in the urban area is the presence of existing underground infrastructures. Most of these underground structures are too old or not initially designed to take extra load from the new road embankments. In order to safely construct the new road embankments, structural integrity of the existing structures needs to be accurately assessed. If the assessment shows that the existing structure is not able to carry the additional load from the new road embankments, the existing structure needs to be strengthened, otherwise additional loads should be avoided on it. Protective structure can be constructed over the existing underground structures to carry the extra load from the new embankment. Strengthening the existing underground structures and building protective structures are usually costly and time consuming. Reducing the weight of the road embankment by using lightweight fill materials can be an effective way to reduce or eliminate additional loads on the existing structures. Using the lightweight fill material in highway construction significantly raised during 1990s [1]. One of the light weight fill material which has been introduced to the construction of lightweight embankment is Expanded Polystyrene (EPS). EPS are used in the road embankments in the form of large blocks. These blocks are placed in interlocking configurations with enough earth fill cover [2]. Figure 1 shows an example of use of EPS geofoams in the construction of road embankments.

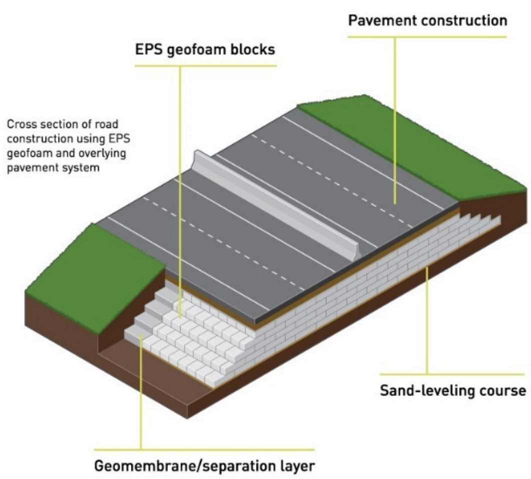

Figure 1. Use of EPS geofoam blocks in road embankment [3]

Since the cost of the EPS blocks per cubic meter is much higher than the regular earth fill material, minimizing the use of EPS in the embankment is always a challenge. The common way of designing the ESP fill embankment is using the conventional analytical solution to estimate the stress on the underground structures and using the existing guidelines. In this study the design and optimization of the EPS configuration in a light weight embankment over an existing underground structure is investigated using numerical modeling due to the complex geometry of the problem. The approach and results of the analyses on a light weight highway embankment over an existing collector as a part of the New Champlain Bridge Corridor will be presented as a case study.

2 CASE STUDY DESCRIPTION

Extension of Highway 15 in Montreal was planned as a part of the New Chaplain Bridge Corridor project. Part of the designed new highway is to be construed over an existing old collector called Saint-Pierre Collector (CSP). Since the structural condition of the CSP was unknown, the pavement and geotechnical design for Highway 15 above the CSP were prepared in such a way that no additional load would be applied on the Collector due to the new construction. This is achieved by compensating fill approach which consists of removing a portion of the existing fill and installing lightweight engineered fill material to the required grade such that the Collector experiences no increase in applied stresses. The lightweight fill used for this purpose is composed of expanded polystyrene (EPS) blocks. In this study the design of EPS fill over 1.2 km of the Highway 15 (presented as segment 2 on the figure below) is investigated.

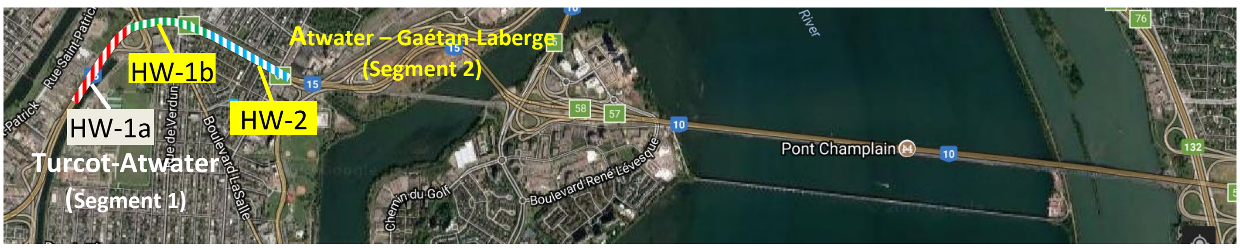

Figure 2. Location of studied area

The existing CSP is a double tube reinforced concrete culvert with two arches with a minimum thickness of 0.4 m. The internal heights of the tubes are 4.27 and 4.88 m. The bottom part of the collector consists of a concrete slab those measures 0.45 m in thickness and 11 m long. Figure 3 shows a typical cross section of the CSP.

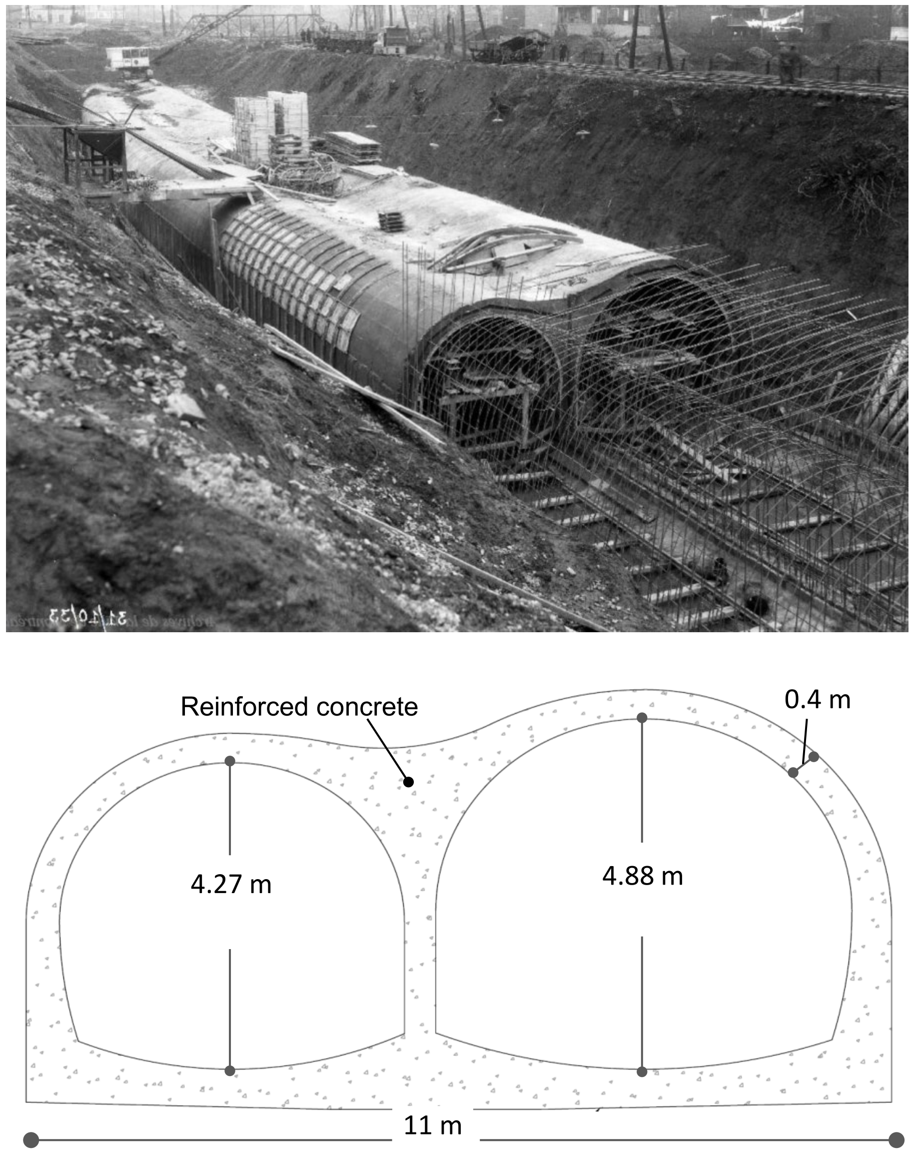

Figure 3: Typical cross section of the Saint-Pierre Collector (CSP)

In the initial design of the EPS fill the influence zone for the CSP were considered as 1H: 1V slope starting at 1 m distance of bottom of CSP, which means any additional load due to the construction of the new highway embankment in this influence zone should be compensated by EPS fill. The goal of this study is to determine the optimum influence zone of the collector using numerical modeling and minimize the use of EPS fill to reduce the cost of the project.

3 NUMERICAL SIMULATION

3.1 Model

The numerical simulations for this study were conducted using finite deference program FLAC 8.0. This 2D finite difference software is a powerful and precise tool for analyzing geotechnical problems. Due to the embankment's geometry and embedded structures, a 2D numerical model is a suitable choice to carry out stress-deformation analyses. Figure 4 shows a general configuration of the model in FLAC. The model is about 100 m long and about 25 m high.

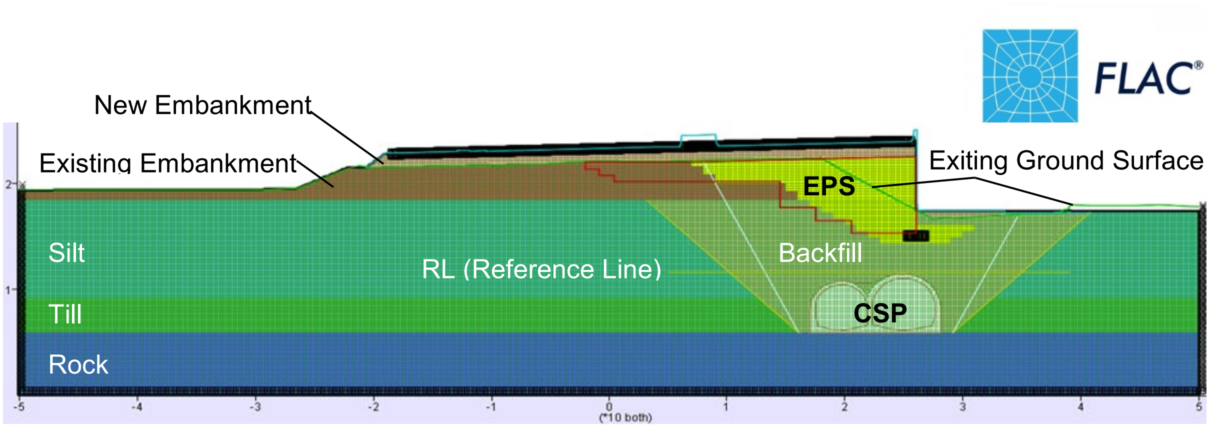

Figure 4: Numerical model of the cross section at the station 33+340 in FLAC 2D

The CSP was simulated as a zone filled with stiff material to represent a solid concrete structure. The backfill zone boundary is considered with a slope of 1H:1V starting at 1 m from bottom corners of the CSP.

Table 1 shows the soil properties used in the numerical models. The soil stiffness parameters were back-calculated from in-situ settlement plate studies. Details of the back-calculation analyses are presented in the following section. As a consequence of the construction stages of the CPS and new fill, the soil models used for some layers must consider unloading and reloading processes. A stress hardening soil model was used to account for soil under unloading and reloading conditions.

The effects of the concrete facing wall on the CSP were taken into account by applying a distributed load at the base of the wall footing. An equivalent load of 76 kN was applied along the 1.2 m length of the footing in order to consider the weight of the wall and the footing. These loads were simulated in the finite difference models as an interior downward load at the elevation of the footing base.

Table 1: Material model and parameters used in the numerical analyses

Material	Model	$\gamma^{(1)}$	E^{ref}_{50}(2)	E^{ref}_{oed}(3)	E^{ref}_{ur}(4)	m(5)	E(6)	ν_{ur}(7)	ν(8)	c(9)	ϕ(10)	ψ(11)	OCR(12)	R_f(13)
		(kN/m³)	(MPa)	(MPa)	(MPa)	(-)	(MPa)	(-)	(-)	(kPa)	(°)	(°)	(-)	(-)
Till	P-H(14)	21	200	200	600	0.5	-	0.2	-	0	38	8	10	0.95
Fill_soil	P-H(14)	17	40	40	120	0.5	-	0.2	-	0	30	0	1	0.95
Fill	P-H(14)	21	50	50	150	0.5	-	0.2	-	0	35	0	1	0.95
Backfill	P-H(14)	17	25	25	75	0.5	-	0.2	-	0	30	0	1	0.95
New fill	M-C(15)	21.5	-	-	-	-	220	-	0.48	0	35	0	-	-
EPS [4]	M-C(15)	1	-	-	-	-	6	-	0.15	10	30	0	-	-
Bedrock	Elastic	27	-	-	-	-	6000	-	0.27	-	-	-	-	-
Collector	Elastic	20	-	-	-	-	1005	-	0.15	-	-	-	-	-

Notes 1: Unit weight
2: Initial stiffness at the reference pressure
3: Tangent oedometer stiffness at the reference pressure
4: Unloading-reloading stiffness modulus at the reference pressure
5: Exponent of the nonlinear modulus model
6: Elastic modulus
7: Unloading-reloading Poisson ratio
8: Poisson ratio
9: Cohesion
10: Friction Angle
11: Dilation Angle
12: Over consolidation ratio
13: Failure Ratio
14: Plastic-Hardening
15: Mohr-Coulomb

To accurately consider the stress and deformation incurred during the construction of the CSP and the existing embankment, staged construction was applied in the numerical analyses. Initial conditions were modeled as three horizontal layers of rock, till and fill-soil before building the CSP. Initial stresses were produced and followed by Plastic-Hardening analysis of the till layer. At the next stage, the collector and the backfill were built and then followed by another Plastic-Hardening analysis. The Plastic-Hardening model is a shear and volumetric hardening constitutive model used for the simulation of unloading-reloading situations. The existing embankment and retaining structures were then applied to the model and consequently stress and deformation analyses were performed by applying a 17 kPa traffic load. At the final construction stage, the planned new fill was analyzed and the final predicted traffic load was applied.

3.2 Back-Calculation of Material Properties

3.2.1 Description of the test site

Field measurement of the settlement was performed at a retaining wall located within the vicinity of the studied segment as shown in Figure 5. Estimating the soil properties to calibrate the numerical models required the results from a settlement plate test. Settlements were measured over time at the location indicated in Figure 5. The available data were recorded during the settlement survey conducted during construction of the MS-1 MSE wall.

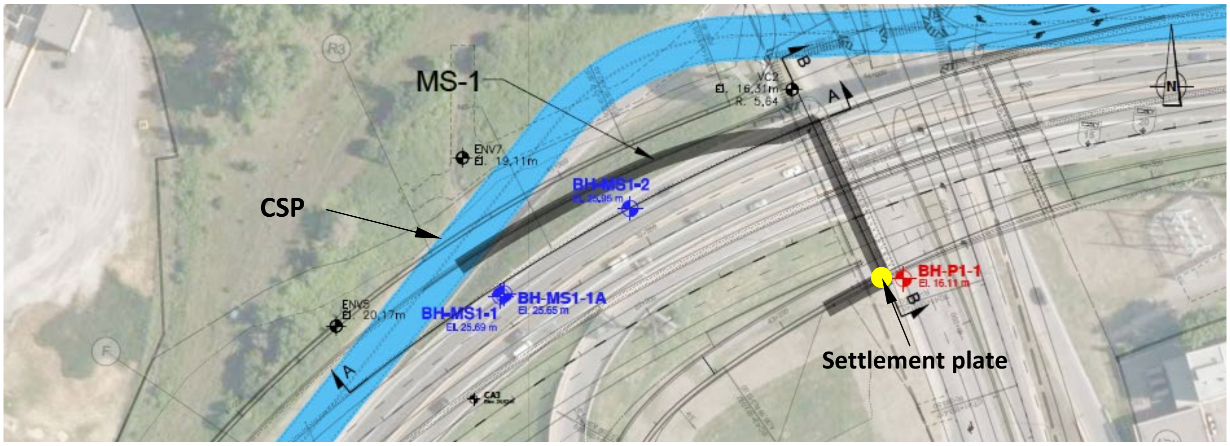

Figure 5 : Location of retaining wall MS-1 at the construction site

3.3 Numerical analysis

In order to determine the nonlinear modulus parameters of the soil, back-calculations were performed using software FLAC with trial and error. Figure 6 depicts a sample model used in the numerical analysis of the MS-1 wall settlement. All stages of the MS-1 wall construction, including the initial condition, the excavation, and the six wall layers were modeled in FLAC using the stage construction technique.

For various sets of nonlinear modulus soil parameters used in the model, settlement values were calculated for five points at the bottom of the wall and then compared with the values measured during the settlement survey of the MS-1 wall. For these calculations, it is assumed that $E_{ur} = 3E_0$. Here E_{ur} is the unloading-reloading modulus and E_0 is the initial elastic modulus. Based on the results of these analyses, soil properties were then back-calculated. Table 1 presents the known (assumed) soil parameters used in the back-calculations.

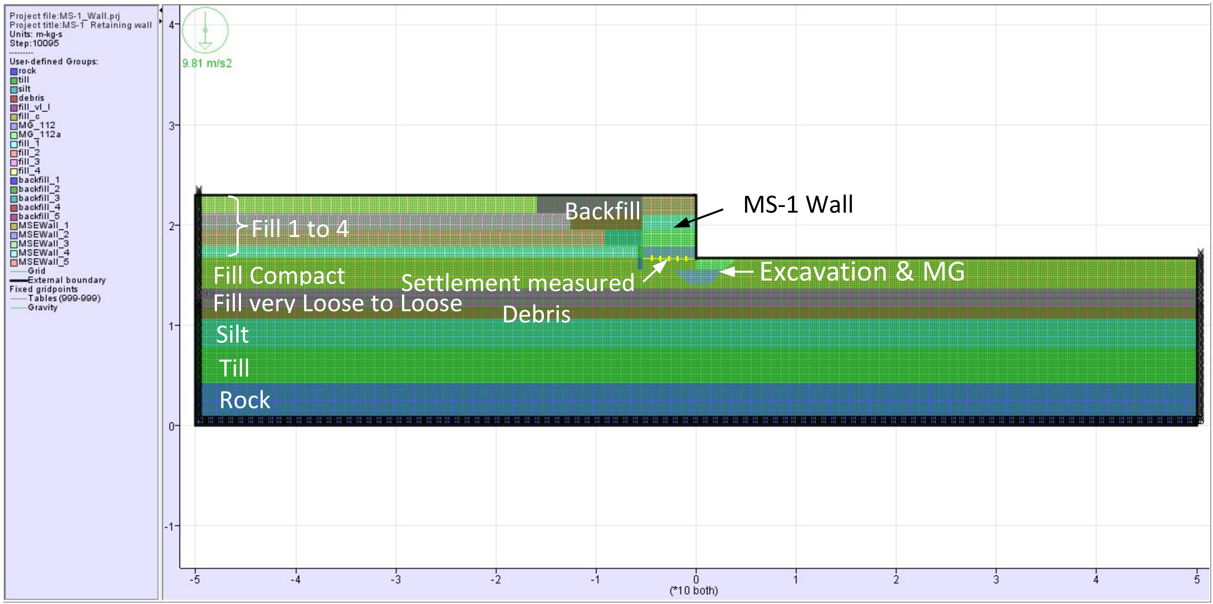

Figure 6: Sample model geometry for settlement in FLAC 8.0

Table 2 : Known (assumed) soil parameters

Layer	Density	Unit weight	Poisson Ratio		Cohesion	Friction Angle	Dilation Angle	OCR(1)	Failure Ratio
	[ρ]	[γ]	[ν_{ur}]	]ν]	[c]	[ϕ]	[ψ]		[R_f]
	(Mg/m³)	(kN/m³)	(-)	(-)	(kPa)	(°)	(°)	(-)	(-)
Rock	2.75	27	-	0.27	-	-	-	-	-
Till	2.14	21	0.2	-	0	38	8	10	0.9
Silt	2.10	20.6	0.2	-	2	35	8	4	0.9
Debris	1.83	18	0.2	-	0	30	30	1	0.9
Fill very Loose to loose	1.73	17	0.2	-	0	30	30	1	0.9
Fill Compact	2.14	21	0.2	-	5	32	32	1	0.9

Note 1: Over consolidation ratio

3.4 Back-calculations outcome

The results of the analyses show that, the examined sets of the soil parameters estimate properly the wall settlements (Figure 7). The average surveyed value of settlement at the bottom of the wall is 14 mm, which accurately matches the average calculated value from the numerical models. The soil parameters calculated from back-calculation analyses are presented in Table 3.

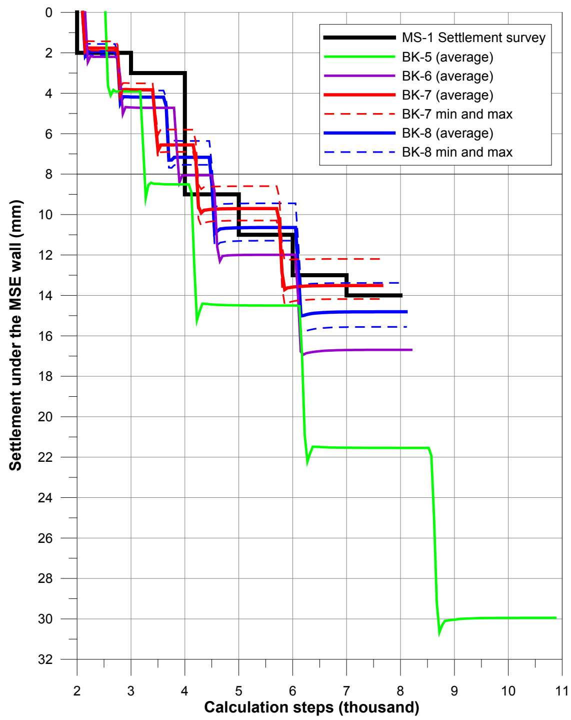

Figure 7. Measured and predicted settlements with back calculated soil properties during the construction of the MSE Wall

Table 3 : Back-calculated soil properties

Layer	Nonlinear Modulus Parameters			
	E^{ref}_{50}(1) (MPa)	E^{ref}_{oed}(2) (MPa)	E^{ref}_{ur}(3) (MPa)	m(4) (-)
Till	200	200	600	0.5
Silt	150	150	450	0.5
Debris	50	50	150	0.5
Fill Very Loose to Loose	20	20	60	0.5
Fill Compact	40	40	120	0.5

Notes 1: Initial stiffness at the reference pressure
2: Tangent oedometer stiffness at the reference pressure
3: Unloading-reloading stiffness modulus at the reference pressure
4: Exponent of the nonlinear modulus model

4 RESULTS

At the reviewing stage, different configurations of EPS blocks were introduced into the model to determine the minimum EPS volume needed for the cross section in order to avoid applying any additional load on the CSP as a result of the new embankment. Hypothetical horizontal reference line RL is defined at 0.2 m above the maximum height of the collector (Figure 5). Vertical stresses (syy) and horizontal stress (sxx) were calculated on RL and on two vertical sections on both side of the CSP to evaluate the effect each EPS configuration would have on inducing stress on the CSP.

In order to evaluate the efficiency of the proposed EPS geometries, the above-mentioned stresses and displacements were compared for three different scenarios: Exiting condition (Existing), Initial proposed EPS configuration (EPS-Initial) and the final geometry proposed in this report (EPS-Revised). The three different studied scenarios are presented Figure 8. Figure 9 presents the vertical stress distribution on RL for Section 32+340 summarizing all three above mentioned scenarios. As in can be seen, the initial design based on approximate analytical solutions was too conservative between points A and B which are not directly on the collector. However, the design seems to be generally conservative, but at point C right on top of the large collector tube, the vertical stress on the collector is much higher than the existing condition which is not allowed (Table 4). The reason for this over stressing at point C is that the analytical solution was not capable of precisely predicting the vertical stress on the collector due to the concrete face wall foundations. The horizontal stresses on the both sides of the collector are presented on Figure 10. As can be seen, the revised EPS configuration using numerical modeling reduces both the vertical and horizontal stresses on the CPS.

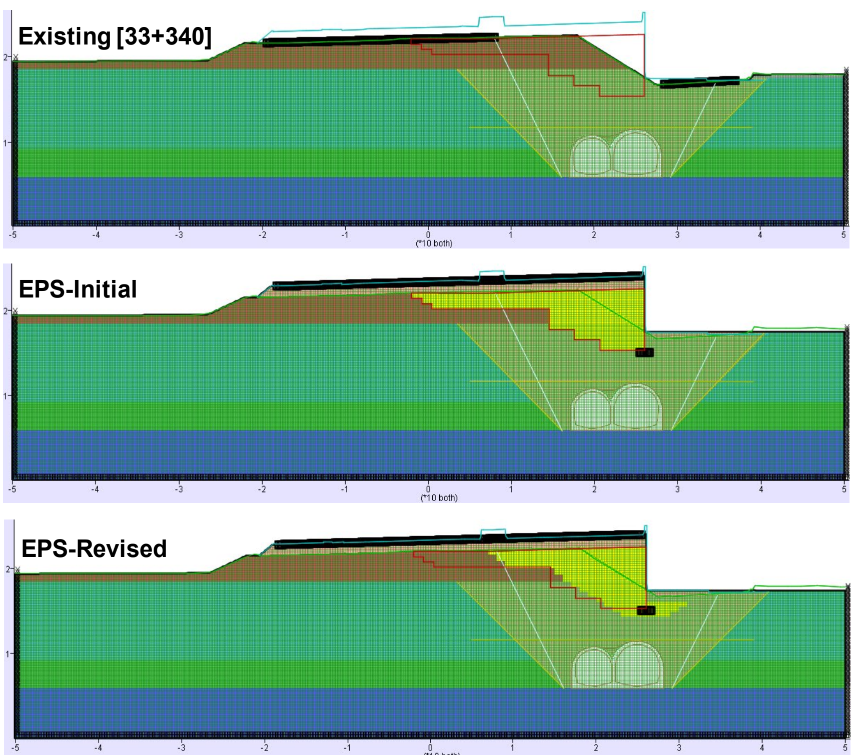

Figure 8. The existing, the Initial EPS configuration and the final proposes EPS configuration simulated in FLAC

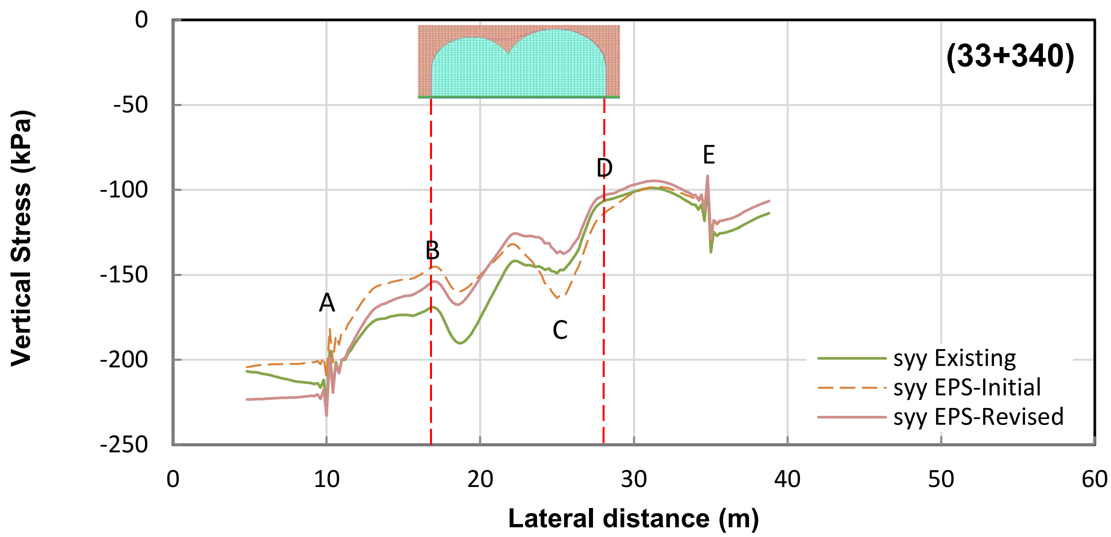

Figure 9. Distribution of the vertical and horizontal stress on RL

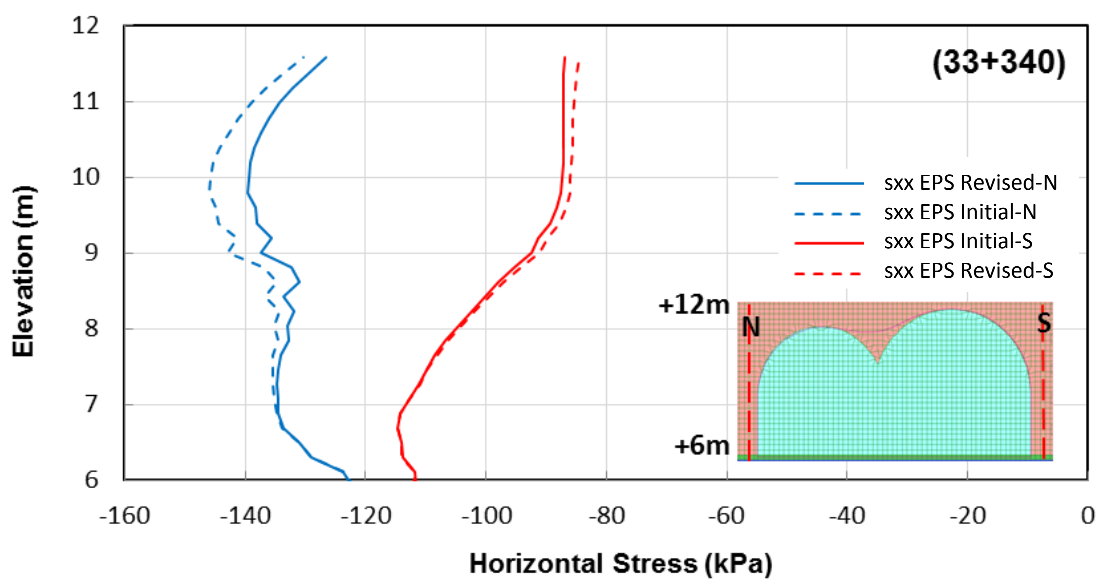

Figure 10. Distribution of the horizontal stress on both side of the collector

Table 4 : Maximum vertical stress at the selected points on the collector

Point	Maximum vertical stress Syy (kPa)			Change EPS-Initial (%)	Change EPS-Revised (%)
	Existing	EPS-Initial	EPS-Revised		
B	170	147	156	-16%	-9%
C	149	162	136	**8%**	-10%

The analyses show that any load applied on the surface outside of the 1H:2V line does not have a significant effect on the horizontal and vertical stress on the collector. This finding reduces the zone of influence from 1H:1V to 1H:2V. Based on the results of the numerical simulations and optimization process, the revised configuration of the EPS at the studied cross section of the highway is presented in the figure below. By removing the unnecessary EPS from the fill, a total volume of about 20 000 m^3 was saved which reduces the material cost of the project by more than 2 M$ over 1.2 km of the highway.

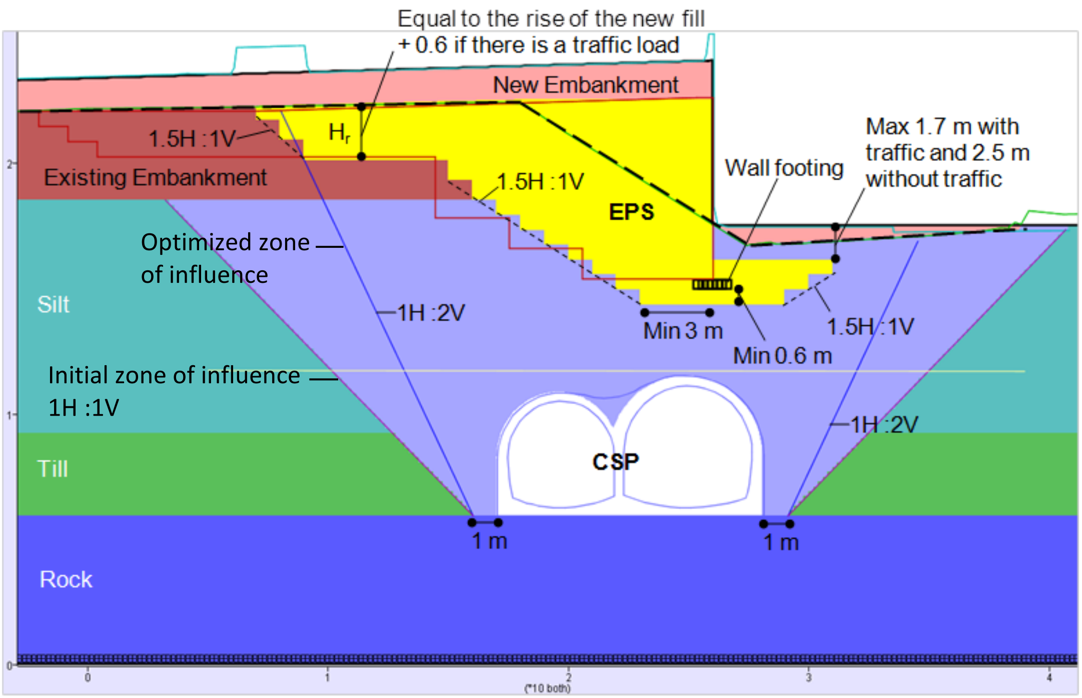

Figure 11. Proposed EPS geometry for the studied cross section

5 CONCLUSION

In this case study, the use of numerical modeling with advanced soil models practiced optimizing the conventionally designed light weight highway embankment. This study shows that using analytical solution in the design of road embankment over existing underground structures can provide too conservative or even not safe design specially when a complex geometry exists. On the other hand, using numerical simulation, provides a detailed information on the stress and deformation analysis of the cross section and help the designers to reach a safe and optimized design, especially with complex geometry problem.

6 BIBLIOGRAPHIE

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