On the numerical modeling of buried structures with compressible inclusion

GEOQuébec 2015
Challenges from North to South
Des défis du Nord au Sud

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ABSTRACT
Expanded polystyrene (EPS) geofoam has been successfully used as a lightweight fill material in geotechnical engineering applications due to its low density and high compressive strength. In this study a 2D nonlinear finite element analysis is conducted to investigate the role of embedding a layer of geofoam within the backfill soil around a buried rigid box on the earth loads reaching the box walls. The earth pressure distribution acting on the box is investigated for varying several parameters including geofoam density, thickness, width and location. The numerical model is first validated by comparing the calculated pressures with experimental data and then used to understand some soil-structure interaction aspects of the problem. Conclusions are made regarding the effectiveness of placing the EPS inclusion on the earth pressure distribution around the structure as well as the resulting soil movement near the soil surface.

RÉSUMÉ
Le Géofoam à base de polystyrène expansé (EPS) est utilisé comme remblai léger dans les applications d'ingénierie géotechnique en raison de sa faible densité et de sa haute résistance à la compression. Dans cette étude, une analyse non linéaire par éléments finis 2D est menée pour étudier le rôle de l'intégration d'une couche de Géofoam dans le sol de remblai autour d'une boîte rigide enterrée sur les charges de terre atteignant les parois de la boîte. La répartition de la pression de la terre agissant sur la boîte est étudiée pour faire varier plusieurs paramètres y compris la densité du Géofoam, l'épaisseur, la largeur et l'emplacement. Le modèle numérique est d'abord validé en comparant les pressions calculées avec des données expérimentales et ensuite utilisé pour comprendre certains aspects sol structure.interaction du problème. Les conclusions sont faites quant à l'efficacité de placer l'inclusion des EPS sur la répartition de la pression de la terre autour de la structure, ainsi que le mouvement du sol résultant près de la surface du sol.

1 INTRODUCTION

The induced trench installation (also called imperfect ditch or ITI method) has been used for several decades to reduce the vertical earth pressure on rigid culverts. It is well known that the earth pressure on deeply buried culverts is affected by soil arching. The method involves installing a compressible layer immediately above the culvert to generate positive arching in the overlying soil.

The Canadian highway bridge design code (CHBDC; CAN/CSA-S6-06, CSA 2006) and the AASHTO LRFD bridge design specifications (AASHTO 2007) provide guidelines for estimating earth loads on positive projecting culverts, but not for culverts installed using induced trench method. This construction method has been an option used by designers to reduce earth pressures on rigid culverts buried under embankments. However, recent doubts toward the induced trench method has left many designers uncertain as to the viability of induced trench construction (McAffee and Valsangkar 2008).

The ITI method of installation for rigid culverts buried under embankments has been used since the early 1900s. Several researches studied the relevant soil-structure interaction using experimental testing and field instrumentation (Sladen and Oswell 1988, Vaslestad et al. 1993, Sven and Liedberg 1997, McAffee and Valsangkar 2008, Sun et al. 2011, and Oshati et al. 2012), as well as numerical modelling (Kim and Yoo 2002, Kang et al. 2008, McAffee and Valsangkar 2008, Sun et al. 2009, and McGuigan and Valsangkar 2010 and 2011) to help better understand the method and to address uncertainties with the design method. However, the majority of these studies have been focused on circular or near circular sections and little work has been done to evaluate the effectiveness of the method for box culverts.

The objective of the present study is to examine the role of geofoam properties in reducing earth pressure on a rigid box culvert. This is achieved using numerical analysis that allows for the effects of several parameters, including EPS density, thickness, width and location to be evaluated. The numerical results are first validated using experimental data and then used to provide a new insight into the interaction between the three different elements (backfill, geofoam and culvert) of the system.

2 BACKGROUND

The imperfect trench installation generally involves placing a compressible material (e.g. EPS, sawdust, shredded rubber tires or very loose fill) directly above a buried structure installed under embankment loading. Figure 1 shows a comparison between the mechanisms of positive projecting and the induced trench conditions for a box culvert.

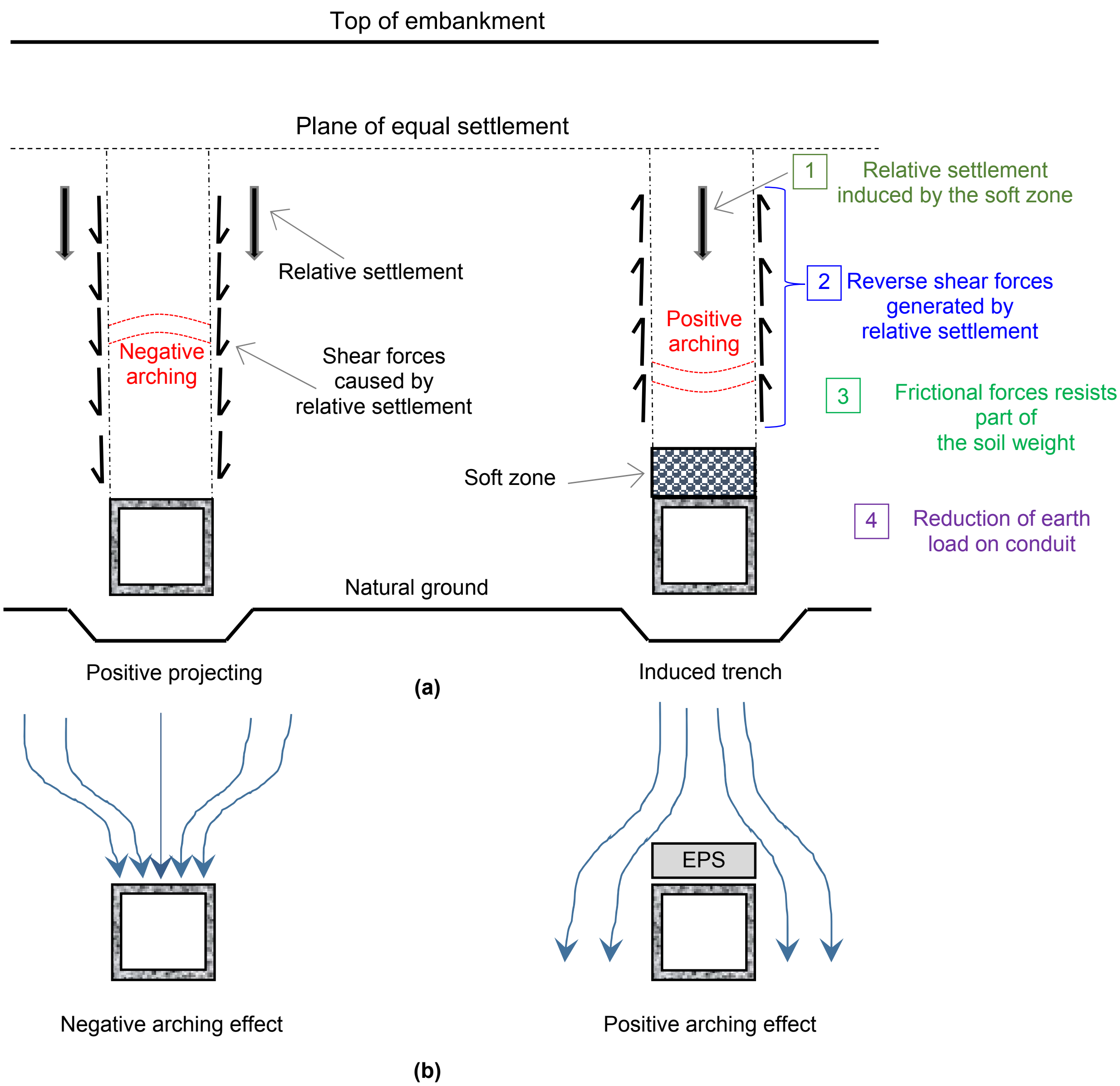

Figure 1. Embankment installations: a) positive projecting versus induced trench, b) negative vs. positive arching

For the case of positive projecting, no compressible zone above the conduit and therefore the walls of the structure experience earth loads that could exceed the overburden pressure. This pressure increase is attributed to the fact that the soil prism above the culvert deforms less compared to the adjacent soil, causing '*negative*' arching. On the contrary, the imperfect ditch installation allows the compressible zone to induce relative settlement between the soil directly above the culvert and the adjacent backfill leading to upward shear stresses (*positive* arching) that reduce earth loads on the culvert.

The ITI method was originally proposed by Marston in the early 1900s and modifications were made by Spangler in 1950 to establish 'Marston-Spangler theory'. For several years the Marston-Spangler (M-S) theory was considered the basis for the design of induced trench installations in many international design standards (e.g. CSA 2006 and AASHTO 2007) and no significant update is made since the publication of the original work. In recent years, limitations of the ITI method have been reported and questions were raised regarding the long-term sustainability of the load reduction process. AASHTO LRFD bridge design specifications (AASHTO 2010) has recognized the ITI method as one of the acceptable methods of installation. However, there are no guidelines or procedure provided to use the method. Instead of relying on the M-S theory, AASHTO suggested the use of accepted test methods, soil-structure interaction analyses, or past experience to determine the earth load on the culvert (Oshati et al. 2012).

3 CONTACT PRESSURE DISTRIBUTION

In this study, the experimental results obtained by Ahmed et al. (2013) are used to validate the numerical model used throughout this study. The experimental work involved the measurement of earth pressure on a box section placed in a rigid chamber overlain by EPS block under an increased surface loading. The numerical analysis is performed using ABAQUS software that allows for the modeling of the response of different materials to be captured. A brief description of the experimental work followed by the details of the numerical investigation are provided in the following sections.

3.1 Experimental Study

The soil chamber (shown in Figure 2) is 1.4 m in length, 0.45 m in width and 1.2 m in height. A square hollow structural section (HSS) with dimensions 25 cm x 25 cm x 43.5 cm and 10 mm in wall thickness is used as a buried structure and instrumented to capture the contact pressure distributions on its walls using a flexible sensing technology.

The backfill material consisted of dry sandy gravel (77% gravel and 23% sand) with unit weight of 16.3 kN/m^3. The soil has a peak friction angle of 47^o as obtained from direct shear tests. The backfill was placed in stages starting with a well compacted bedding layer of 25 cm in height followed by the placement of the HSS box, the side and the top backfill up to the desired height of 0.5 m above the structure.

Surface pressure of up to 140 kPa (equivalent to approximately 8 m of overburden pressure) was applied (with constant displacement rate of 1.3 mm/min) using air bag to insure uniform distribution of stresses.

A total of eleven experiments were conducted including three benchmark tests (no EPS inclusion or positive projection condition) with only the instrumented HSS box inside the backfill and then two sets of four tests are performed for two types of geofoam with different densities (EPS15 with density of 14.4 kg/m^3 and EPS22 with density of 21.6 kg/m^3).

For the tests conducted using EPS inclusion (induced trench condition), EPS block 25 cm in width (equal to the conduit width, B), 43.5 cm in length (equal to the conduit length, L) and 50 mm in thickness is used. Throughout the experiments, the EPS block was located immediately above the culvert.

3.2 Numerical Analysis

The numerical models, for both the positive projecting and induced trench configurations, have been developed such that they follow the geometry and test procedure used in the experiments. A two-dimensional plane strain model was used to investigate the role of geofoam density, thickness, width and location on the changes in earth pressure on the buried structure.

The finite element mesh that represents the geometry of the experiment, the boundary conditions, and the different soil densities around the HSS section is shown in Figure 3. The mesh size was adjusted around the

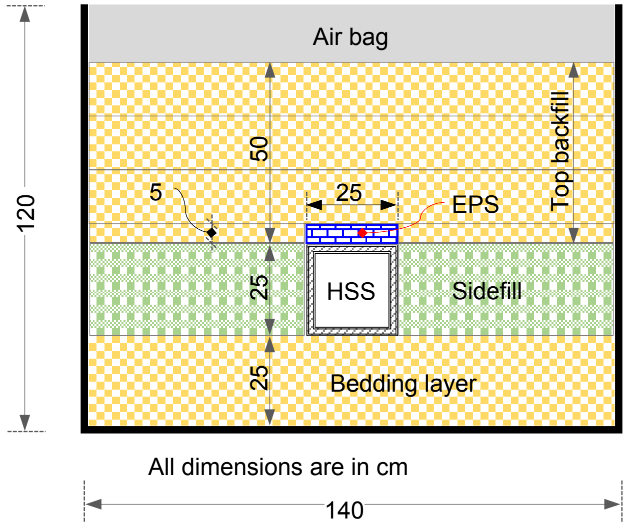

Figure 2. Experimental setup geometry and dimensions

structure to provide sufficient resolution and accuracy within the studied area. The complete mesh comprises a total number of 1962 linear plane strain elements (CPE4) and 2282 nodes.

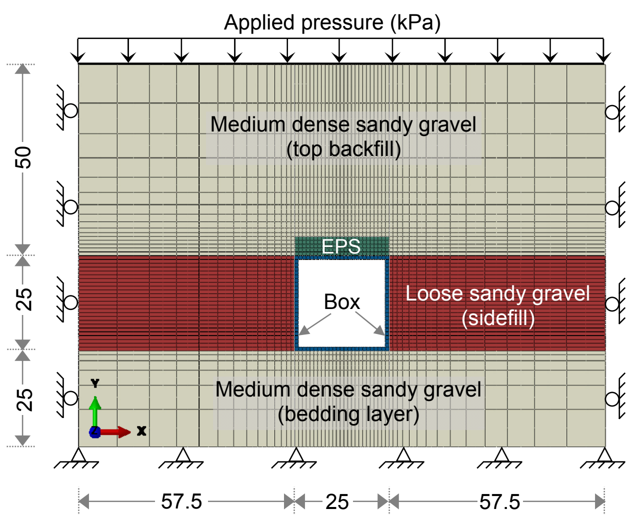

Figure 3. The finite element mesh (dimensions are in cm)

Two types of soils are used in the analysis to represent the compaction conditions of the backfill material as shown in Figure 3. The soil is modeled using Mohr-Coulomb failure criteria with non-associated flow rule. The input parameters as listed in Table 1 below.

The HSS is treated as linear elastic material with density of 7850 kg/m^3 with Poisson's ratio and Young's modulus of 0.3 and 200 GPa, respectively.

Three types of EPS are modeled in this study to examine the effect of geofoam density on the earth load transferred to the structure.

The EPS is modeled as nonlinear elasto-plastic strain hardening material. The elastic properties of the three EPS types are summarized in Table 2.

Table 1. Soil input parameters used in the FE model

Density (kg/m^3)	E (MPa)	Poisson's ratio, ν	$\phi°$	$\psi°$	cohesion (MPa)
Bedding and top backfill layers					
1628	150	0.3	47	15	1E-05
Sidefill layer					
Density (kg/m^3)	E (MPa)	Poisson's ratio, ν	$\phi°$	$\psi°$	cohesion (MPa)
1400	20	0.2	30	5	1E-5

ϕ = friction angle, ψ = dilation angle, E = Young's modulus

Figure 4 shows the data used to specify the hardening rule used in the EPS plasticity model. This model is validated by comparing compression test results conducted on 125 mm cubes with numerical values. Due to space limitations, the details of such analysis are not included in this study.

Table 2. Input parameters for the elastic model of EPS

Geofoam type	Density (kg/m^3)	E (MPa)	Poisson's ratio, ν
EPS15	14.4	4.20	0.1
EPS22	21.6	6.91	0.1
EPS39	38.4	17.8	0.15

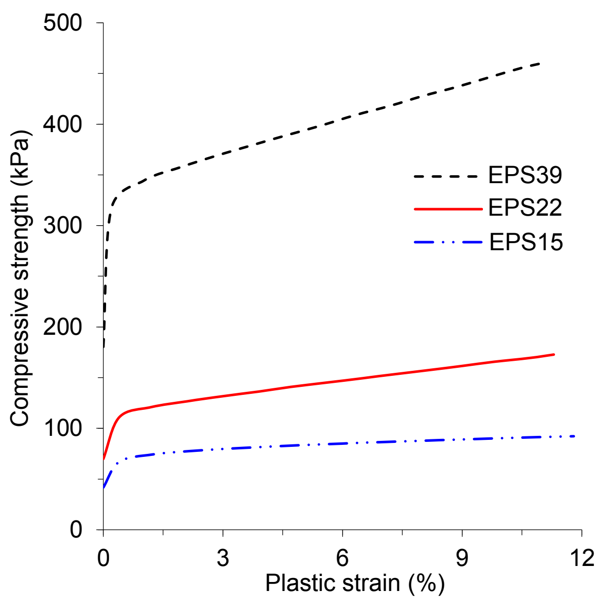

Figure 4. Hardening rule for the EPS plasticity model

Three different contact conditions are considered in this study; namely, i) Soil-EPS interaction, ii) Soil-BOX interaction and iii) EPS-BOX interaction. These interactions are simulated using the surface-to-surface, master/slave contact technique available in ABQUS. Contact formulation in 2D space covers both tangential and normal directions.

In the tangential direction, Coulomb friction model is used to describe the shear interaction between the geofoam, the structure, and the surrounding soil. This model involves two material parameters- a friction coefficient (μ), and a tolerance parameter (E_{slip}). The shearing resistance (τ) is considered as a function of the shear displacement that represent the relative movement between the two contacted parties. On the other hand, a 'hard' contact model is used to simulate the contact pressure in the normal direction. The parameters used to describe these interface conditions are given in Table 3.

Boundary conditions were defined such that nodes along the vertical boundaries of the mesh may translate freely in the vertical direction but are fixed against displacements normal to the boundaries (smooth rigid). The nodes at the base are fixed against displacements in both directions (rough rigid) as shown in Figure 3.

After the model is generated, the initial geostatic stress condition is established by applying soil gravity and incrementally introducing the surface overburden pressure to achieve a gradual response curve.

Table 3. Input parameters for the interface model

Soil-EPS-Culvert interface parameters		
Interface	Friction coefficient (μ)	E_{slip}
Soil-EPS	0.6	0.005
Soil-BOX	0.45	
EPS-BOX	0.3	

4 RESULTS AND DISCUSSION

In this section, the effect of introducing a compressible EPS block on the earth load transferred to the buried structure is examined. A parametric study is, then, presented to evaluate the role of EPS density, width, thickness and location on the response of the system.

4.1 Validation of the Numerical Model

The numerical results are first validated by comparing the calculated pressures with the measured values for three cases a) benchmark test with no geofoam (positive projecting), b) using EPS15, and c) using EPS22.

To facilitate such comparison, the average pressure on a section near the centre of the structure with effective width equal to that of the sensing pads (around 0.65B, B is the box width) is calculated. As shown in Figure 5, the numerical model generally captured the pressure change with a reasonable accuracy particularly for the upper and side walls for the benchmark test (Figure 5a and 5c). However, the model slightly underestimated the contact pressure under the lower wall for the case of EPS15 (Figure 5b).

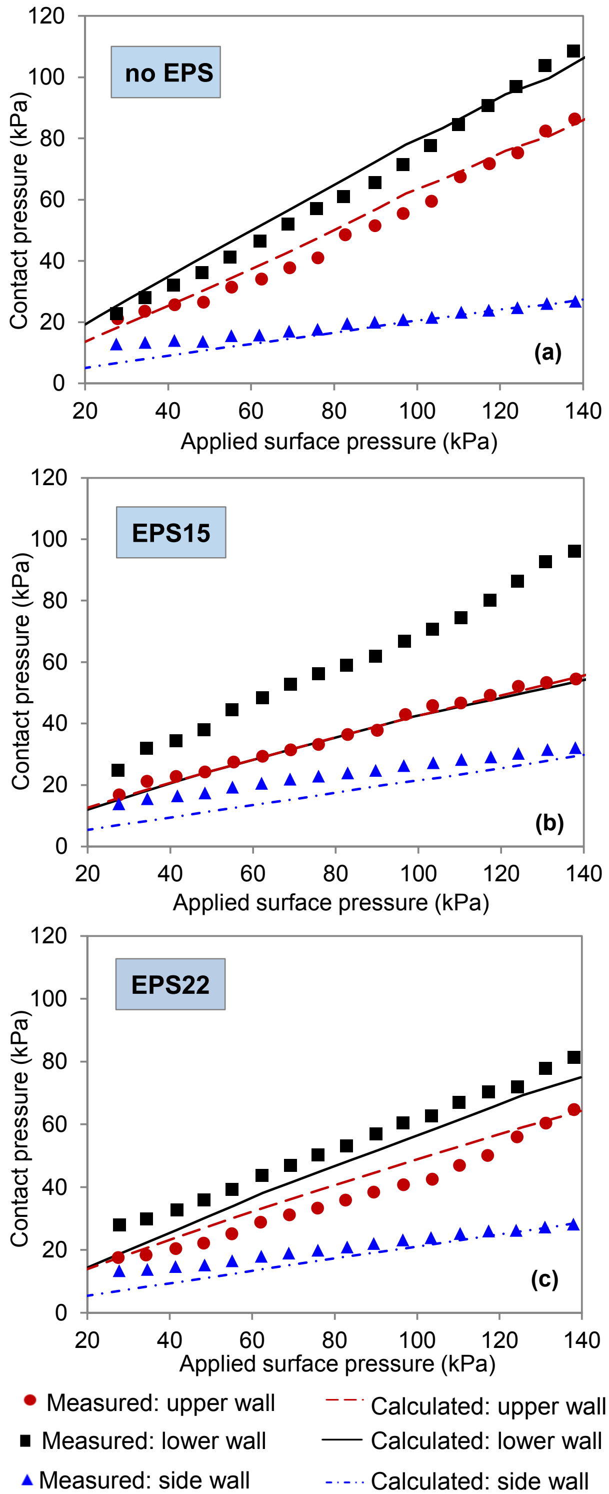

Figure 5. Model validation for the cases of a) no EPS, b) EPS15 and c) EPS22

4.2 Parametric Study

Upon the verification of the FE model, a parametric study is carried out using the proposed model with uniform soil properties and maximum applied pressure of 300 kPa to allow for the behavior of the system to be investigated at high stress levels.

4.2.1 Effect of EPS density

The effect of EPS density is examined by comparing the calculated pressure at the investigated locations (upper, lower and side walls) for three different EPS materials, namely, EPS15, EPS22, and EPS39 (properties are given in Table 2). The EPS width and thickness were kept constant (similar to those used in the experiment). A surface pressure that allows for 1% EPS deformation to be achieved is used throughout this study. The results of the analysis are presented in Figure 6. For comparison purposes, the calculated pressure for each case is also compared with the benchmark analysis (with no geofoam). The vertical axes in Figure 6 represent the contact pressure ratio normalized with respect to the benchmark case.

For the upper wall (Figure 6a), the EPS density was found to have a significant impact on the earth pressure acting on the wall. Compared to the benchmark, the lowest contact pressure is calculated for the case of EPS15 (density = 14.4 kg/m^3). The pressure reduction for different applied surface pressure is presented in Table 4.

Table 4. Pressure reduction and allowable surface pressure for 1% strain of EPS at top

Geofoam type	Density (kg/m^3)	Allowable surface pressure @ 1% (kPa)	Pressure reduction (%)
EPS15	14.4	113	65
EPS22	21.6	129	54
EPS39	38.4	144	23

The pressure reduction ratios for the lower wall (Figure 6b) were found to be 28%, 25% and 14% for EPS15, EPS22 and EPS39, respectively. These effects are considered to be significantly smaller as compared to that calculated for the upper wall. Similar trends were found for the contact pressures on the side wall (Figure 6c) with pressure reduction ratios of 34%, 28% and 15%.
It is worth noting that, due to the linear nature of the calculated responses, the above reduction ratios are expected to apply for other surface pressures as long as the maximum strain in the EPS does not exceed 1%.

4.2.2 Effect of EPS width

To evaluate the effect of geofoam width on the contact pressure acting on the walls of the HSS box, the EPS thickness and density are kept constant and the width is incrementally increased from one to two times the width of the HSS section. The results are summarized in Figure 7. It can be seen that increasing the width of the EPS from 1B to 2B led to 12% increase in contact pressure at the upper wall (Figure 7a). For the lower and side walls, however, the contact pressure decreased by about 10%. This is considered to be insignificant improvement given that twice the geofoam volume (from 1B to 2B) was used.

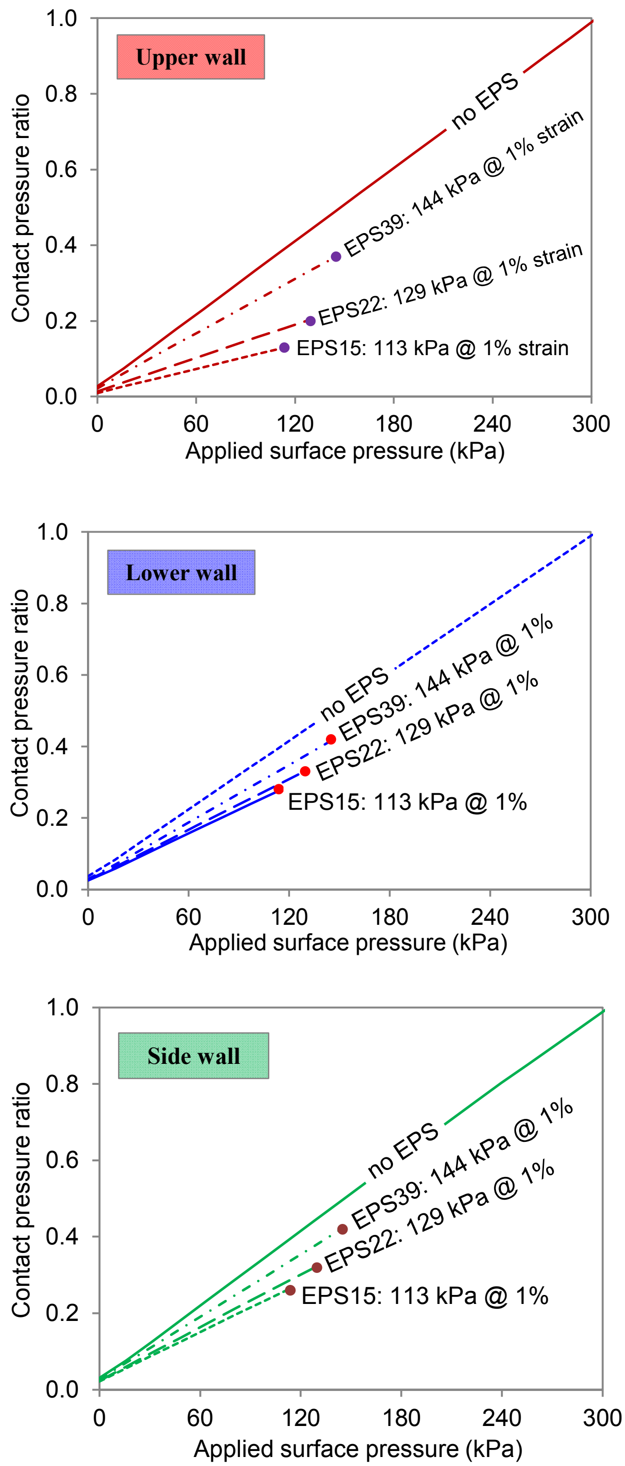

Figure 6. Effect of EPS density on the change of earth pressure on the culvert walls for EPS strain of up to 1%

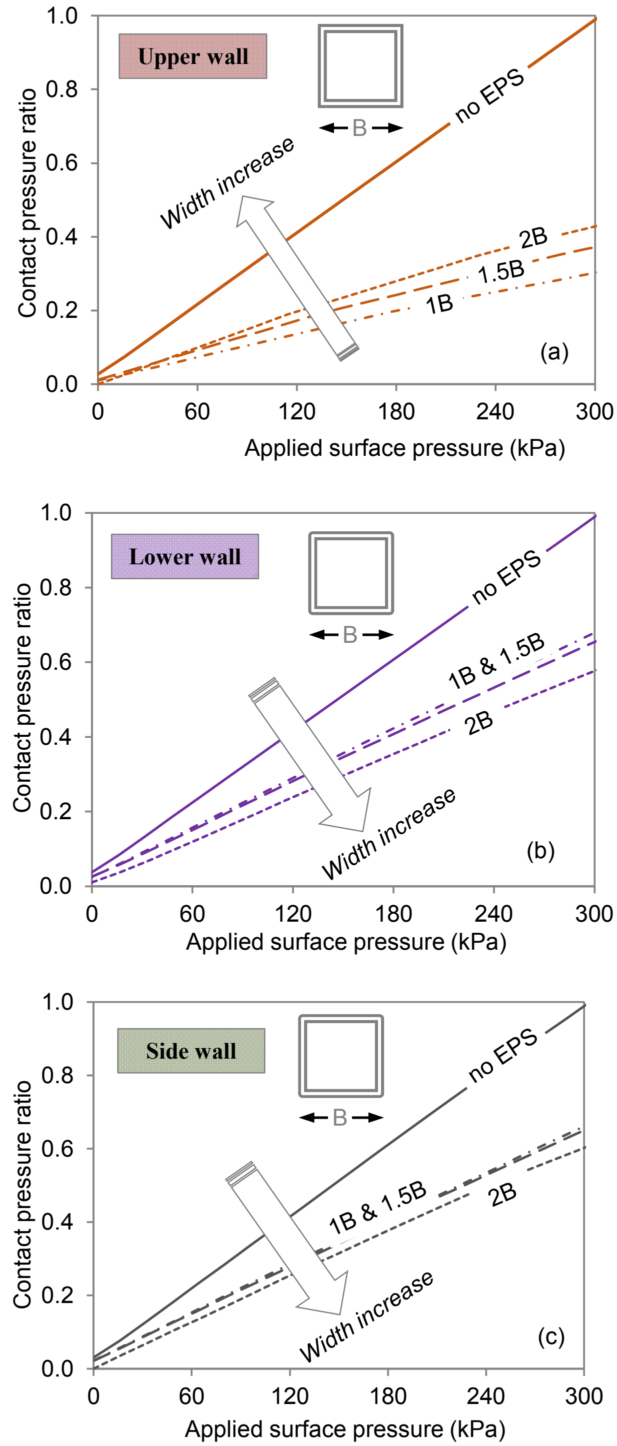

Figure 7. Effect of EPS width on the change of earth pressure on the culvert walls for EPS strain of up to 3%

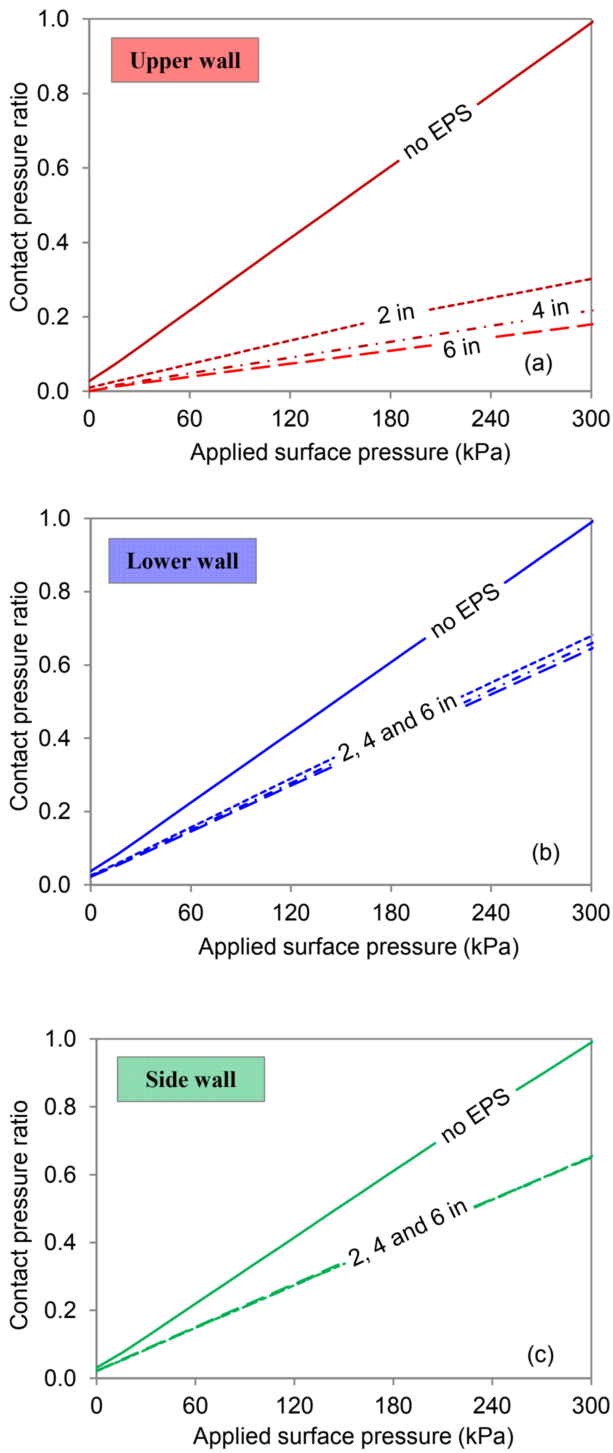

Figure 8. Effect of EPS thickness on the change of earth pressure on the culvert walls for EPS strain of up to 3%

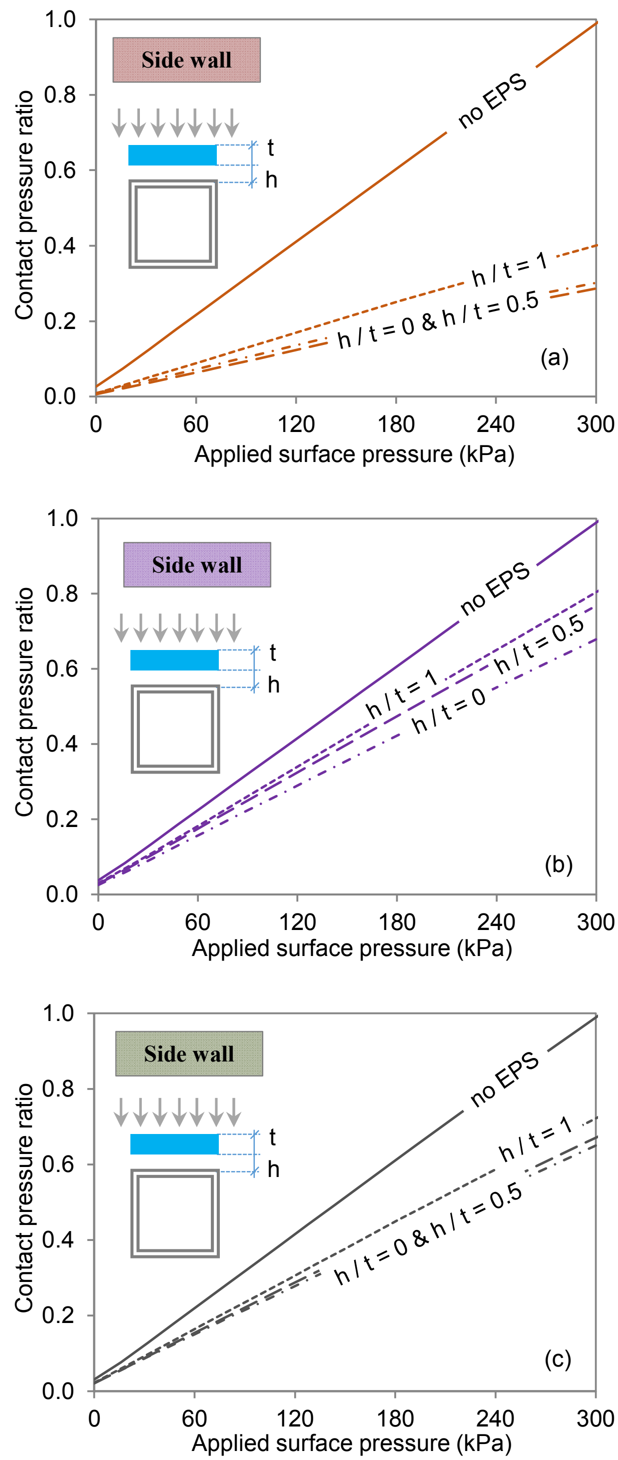

Figure 9. Effect of EPS location on the change of earth pressure on the culvert walls for EPS strain of up to 3%

4.2.3 Effect of EPS thickness

The effect of the EPS thickness is examined in Figure 8 for EPS15. The EPS width was chosen to be equal to that of the HSS section (width = 1B). The thickness is increased incrementally from 2-inch to 6-inch and the contact pressure is calculated for each case around the HSS box. At the upper wall (Figure 8a), increasing the thickness of the EPS block from 2-inch to 6-inch resulted in pressure decrease of about 18%. No significant change was found for the lower and side walls as shown in Figures 8b and 8c.

4.2.4 Effect of EPS location

Figure 9 illustrates the effect of EPS block location with respect to the upper wall on the pressure transferred to the buried structure. The modeled geofoam block is EPS15 with 2-inch in thickness (t) placed at three different locations (h) such that h/t = 0, 0.5 and 1. It can be seen (Figures 9a and 9b) that moving the EPS block by 2 inches (1t) led to a reduction in contact pressure at the upper and lower walls of about 10%. No significant change in pressure was found for the side walls (Figure 9c) as a result of the change in EPS location.

5 SUMMARY AND CONCLUSIONS

In this paper a series of plane strain finite element analyses was carried out using ABAQUS software to study the role of EPS inclusion above a buried box conduit in reducing the earth pressure on the walls of the structure. The developed model was used to investigate a case study of an instrumented HSS section (with and without EPS) that was placed within a rigid steel container backfilled with sandy gravel material and loaded incrementally with a vertical pressure using an air bag.

A parametric study was conducted to examine the effect of the EPS density, width, thickness and location on the earth pressure acting on the HSS section. The only factor that was found to have a significant impact on the changes in earth pressure is the material density. For the investigated range of parameters, results showed that the EPS width and location did not sufficiently contribute to the positive arching process and, therefore, only minor pressure changes were calculated.

The above study suggests that placing light weight EPS material above a rigid subsurface structure can result in a significant reduction in vertical earth pressure resulting in economic design.

ACKNOWLEDGEMENTS

This research is supported by a research grant from the Natural Science and Engineering Research Council of Canada (NSERC). The support of Plasti-Fab Ltd. throughout this study is greatly appreciated.

REFERENCES

AASHTO. 2007. AASHTO LRFD Bridge Design Specifications. 4th ed. American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO. 2010. AASHTO LRFD Bridge Design Specifications. 5th ed. American Association of State Highway and Transportation Officials, Washington, D.C.

Ahmed, M.R., Meguid, M.A., and Whalen, J. 2013. Laboratory measurement of the load reduction on buried structures overlain by EPS geofoam, the 66th Canadian Geotechnical Conference in Montreal.

CSA, 2006. Canadian Highway Bridge Design Code (CHBDC), CSA standard CAN/CSA-S6-06, Canadian Standards Association, Mississauga, Canada.

Kang, J., Parker, F., Kang, Y.J., and Yoo, C.H. 2008. Effects of frictional forces acting on sidewalls of buried box culverts. International Journal for Numerical and Analytical Methods in Geomechanics, 32(3): 289–306.

Kim, K., and Yoo, C.H., 2002. Design Loading for Deeply Buried Box Culverts, Highway Research Center, Auburn University, Report No. IR-02-03.

McAffee, R.P., and Valsangkar, A.J. 2008. Field performance, centrifuge testing, and numerical modelling of an induced trench installation, Canadian Geotechnical Journal, 45(1): 85–101.

McGuigan, B.L., and Valsangkar, A.J. 2010. Centrifuge testing and numerical analysis of box culverts installed in induced trenches. Canadian Geotechnical Journal, 47(2): 147–163.

McGuigan, B.L., and Valsangkar, A.J. 2011. Earth pressures on twin positive projecting and induced trench box culverts under high embankments, Canadian Geotechnical Journal, 48(2): 173-185.

Oshati, O.S., Valsangkar, A.J., and Schriver, A.B. 2008. Earth pressures exerted on an induced trench cast-in-place double-cell rectangular box culvert, Canadian Geotechnical Journal, 49: 1267-1284.

Sladen J.A. and Oswell J.M., 1988. The induced trench method – a critical review and case history, Canadian Geotechnical Journal, Vol.25, 541-549.

Sun, C., Hopkins T.C., and Benkham, T.L., 2009. Reduction of stresses on buried rigid highway structures using the imperfect ditch method and expanded polysterene (geofoam). Kentucky Transportaion Center, Report No. KTC-07-14-SPR-228-01-1F.

Sun, L., Hopkins, T.C., and Beckham, T.L. (2011). "Long-term monitoring of culvert load reduction using an imperfect ditch backfilled with geofoam." Transportation Research Record, 2212, 56-64.

Sven, N., and Liedberg, D., 1997. Load reduction on a rigid pipe pilot study of a soft cushion, Transportation Research Record, No.1594, 217-223.

Vaslestad, J., Johansen, T.H., and Holm, W., 1993. Load reduction on rigid culverts beneath high fills: long term behavior, Transportation Research Record 1415, Norwegian Road Research Laboratory, Public Road Research Administration, 58-68.