Field Performance of Geogrid Bridges for Stress Reduction on Buried Utilities

Performance in-situ des pontages en géogrille pour réduire les contraintes dans les infrastructures souterraines

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ABSTRACT: The construction of highway embankments in urban areas often interferes with existing underground facilities such as sewer lines and other buried conduits. In many instances, the extra loads imposed by embankment construction on buried conduits would be unacceptably high. The severe consequences of overstressing an underground utility conduit include damage and interruption of services for both the utility and highway. This paper present results of a full scale instrumented test embankment constructed by Ontario Ministry of Transportation to study the effects of embankment construction on the underground utilities. The test embankment comprised four sections which facilitated the evaluation of four different configurations including the positive projection installation, induced trench installation and two at-grade geogrid reinforcing bridging with different spans. The numerical models of the test embankment are developed using two dimensional finite element analyses. This paper presents the results of stress measurements inside the trench protected using at-grade geogrid bridge arrangement as well as the results of numerical model that helped clarify mechanisms of stress reduction. The material presented is considered to be of interest to researches and engineers.

RÉSUMÉ : La construction de remblais dans les zones urbaines existantes interfère souvent avec des installations souterraines comme les égouts et autres canalisations enterrées. Dans de nombreux cas, les charges supplémentaires imposées par la construction du remblai sur les conduites enterrées sont trop élevées. Les conséquences graves d'une surcharge sur une conduite souterraine des services publics comprennent à la fois les dommages et l'interruption des services. Cet article présente les résultats d'un remblai d'essai instrumenté à grande échelle, construit par le Ministère des Transports de l'Ontario pour étudier les effets des étapes de la construction sur les réseaux souterrains. Le remblai d'essai comprend quatre sections, ce qui a facilité l'évaluation de quatre méthodes différentes d'installation de conduites enterrées : projection positive, tranchée induite et deux autres avec pontage par géogrille de renforcement avec des portées différentes. Par ailleurs, des modèles numériques par éléments finis en deux dimensions sont développés pour simuler le comportement du remblai d'essai. Les mesures expérimentales et numériques obtenues dans cette étude sont présentées et analysées. Les résultats obtenus sont d'un grand intérêt pour les ingénieurs praticiens.

KEYWORDS: Geogrid bridge, Test embankment, Underground utilities, Stress reduction, 2D FEM.

1 INTRODUCTION

The construction of highway embankments often intersects with the alignments of existing utility lines. Since additional embankment loads are not accounted for in the initial design of these buried structures, embankment construction may result in overstressing and damage of the existing utility lines.

Geogrids are flexible, synthetic meshes that are used for slope stabilization, highway pavement reinforcement, earth retention and sub-grade improvement. One of the main purposes of geogrids used in soil reinforcement is to provide confinement and reinforcement to the soil medium. Love (1984) and Hass et al. (1988) indicated that interlocking between geogrid and the aggregate leads to an increased confinement within the granular base course. Due to this enhancement in confinement, lateral spreading of the particles is minimized and the stiffness and strength moduli of the base course increase.

Geogrids improve the structural integrity of reinforced soil foundations by confining the soil and distributing applied forces which result in an increased load distribution angle.

Geogrids have been widely used to improve weak foundation soils for the construction of access roads and highways. More recently, they have been used in standard flexible pavement sections to reinforce the base course to support vehicular traffic during the life of the pavement structure (base-course reinforcement). When used as base-course reinforcement, geogrids provide significant structural benefits that are very attractive to transportation authorities, which include an improved pavement life and/or equivalent performance with a reduced structural section.

Geogrid bridging is an effective construction technique that can be used where bridging an area of very weak subgrade soils is necessary. In this technique a layer or more of geogrid works as a bridge that transfers overlaying stresses and distribute them to larger areas away from the zone of weakness.

Current versions of Canadian Highway Bridge Design Code (CHBDC 2006) and the AASHTO LRFD bridge design specifications (AASHTO 2007) do not include any clauses related to the performance of geogrids bridging. Only recommendations regarding the culvert installations are related to the ones installed using positive projection method (PPM). Design recommendations given in these codes for PPM include the assumptions of uniform earth pressure on top of the culvert crown and the uniform pressure on the bottom slab of culvert that is equal to the sum of the crown pressure and the pressure due to dead load of culvert. In order to gain a better understanding of the stress reduction that may be achieved by geogrid bridging, Ontario Ministry of Transportation (MTO) constructed an instrumented full scale test embankment over a bridged trench in order to study various stress reduction measures.

The main objectives of this study are to evaluate the stress reductions achieved by the use of geogrid bridging installation and to gain an understanding of the stress reduction mechanism. This paper presents the results of stress measurements inside the trench protected using at-grade geogrid bridging as well as the results of numerical model that helped clarify mechanisms of stress reduction. The material presented in this paper is considered to be of interest to researches and engineers.

2 METHODOLOGY

2.1 *Description of the field test*

An instrumented test embankment is constructed by Ontario Ministry of Transportation over a 3 m deep trench to study the effects of embankment construction on the underground utilities. The test embankment is constructed as part of Highway 407 contract and is located near the Highway 407 and Weston Road in Vaughan, Ontario. The test embankment comprised several sections which facilitated the evaluation of four different configurations including the positive projection installation, induced trench installation and at-grade geogrid reinforcing bridging. Since the objective of the testing program was to evaluate the stresses reductions achieved by each of the considered installation techniques, no actual culverts were installed. Instead, the earth pressure cells were placed in granular protective surrounds located at two different depths in the trench. The first section is constructed as a conventional control section (Section 1), which included an instrumented trench that was conventionally backfilled with granular material up to the grade level (Positive Projection Method). The second section represents an Induced Trench Method installation (ITM) (Section 2), which included an instrumented trench that was backfilled with granular material that is overlain by a layer of compressible Styrofoam chips up to the grade level. The third and fourth sections include geogrid void bridges installed on the instrumented trenches with different trench widths. The evaluation of the induced trench method installation section was performed as part of a separate study and is beyond the scope of the present work. This study involve the results of the PPM represented in Section 1 (the control case) and the 3 m wide at-grade geogrid bridging, GB (Section 4). The following is a general description of field tests. The layout of test embankment is depicted in Figure 1. The considered test configuration comprises a 3 m deep trench backfilled up to the grade level which underlies an embankment with the footprint dimension of 64m x 34m in length and width and with a height of 6 m. The slopes of the embankment were constructed with a 2H:1V inclination. Figure 2a and 2b show the details of instrumented trenches for Section 1 and 4, respectively.

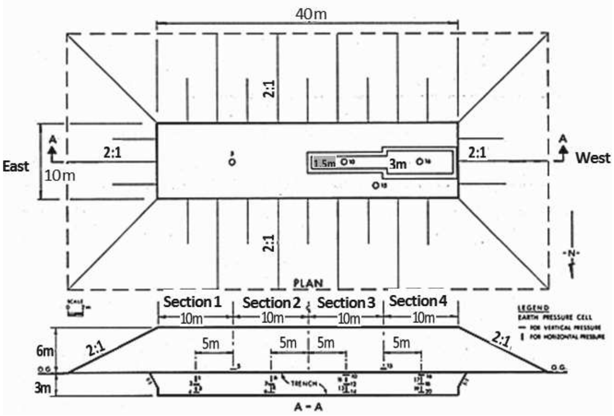

Figure 1. Plan and Profile of Test Embankment

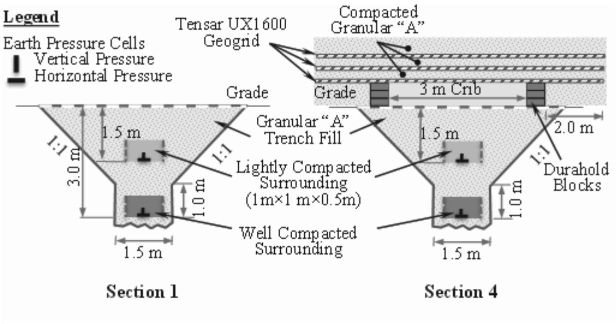

Figure 2. Details of the instrumented trenches

2.2 *Construction and Instrumentation*

The site was leveled and topsoil was removed prior to the construction of the test embankment. A 1.5 m wide trench was excavated to a depth of 3m. 1m above the bottom of trench, trench slopes were cut to 1H:1V to maintain the integrity of trench side slopes during the construction (see Figure 2). Two drainage sumps are installed at each end to maintain a dry condition in the trench. The base of the trench is filled with a 300mm thick compacted granular pad comprising Granular-A (OPSS 314). Then, the installation of earth pressure cells was initiated. Earth pressure cells with vibrating wire pressure transducers were used. The earth pressure transducers were placed at the bottom (3m from the grade) and the mid-height (1.5 m from grade) of the trench with horizontal and vertical orientations in order to allow measurement of vertical and horizontal pressures, respectively. The pressure transducers were placed in a 0.5m x 1m x 1m granular protective surrounds. The granular surrounds were constructed as lightly compacted at mid-height pressure cell and well compacted at the bottom pressure cell to see the effect of the surround compaction on the measured stresses. The protective surrounds, which were constructed to eliminate a possible damage to pressure cells were constructed using steel separators in four sides. The steel separators separate the lightly compacted and well compacted granular surrounds from compacted Granular-A, which constitutes the rest of the fill material in the trench. The steel separators were removed as the level of granular surrounds reached 0.5 m height. A total of 2 transducer couples (vertical and horizontal) were installed at the bottom and mid-height of each trench for each section and an additional transducer is installed at the interface between trench backfill and embankment. The trench backfill encapsulating granular surrounds, comprise Granular A compacted to 95% of Standard Proctor Dry Density. The pneumatic lines were extended through a PVC pipe, which followed the edges of trench and connected to a monitoring station.

The embankment was constructed using native cohesive soils in the area. The soil was placed in the lift of 300mm and compacted to 95% of Standard Proctor Dry Density. The portion of the embankment fill immediately above the trench was placed by hand and compacted by self-propelled compaction equipment (Bomag BW65S) in order to protect against heavy machine loading used for compaction of embankment fill. The embankment fill, including the zone above the trench, is compacted using regular compaction equipment after fill height reached 1.3 m.

3 NUMERICAL MODEL

In order to investigate the stress reduction mechanism of geogrid bridging, finite element analyses (FEA) were carried out using the software package Plaxis 2D (Plaxis bv. 2011). Two sets of numerical models were developed to analyze stresses generated in cases of PPM installation (Section 1) and the 3 m wide at-grade geogrid bridging, GB (Section 4) installation. An elastic-plastic soil model with Mohr-Coulomb failure criterion is used during the simulation of full scale field test due to the simplicity of the model and availability of model parameters.

The strength parameters were determined and reported in a geotechnical investigation and design report prior to the construction of test embankment. However, no direct determination for the stiffness properties of soil was performed. Thus, elastic moduli for various soils were determined based on experience and calibrated against the deformation of embankment. The stresses within the trench were affected by the stiffness of the granular fill. Thus, the hyperbolic hardening soil model from the Plaxis' library was used to model the stress dependent variation of stiffness of the fill materials within the trench. A fully fixed boundary condition was applied at the base of the models. The lateral boundaries of the models were placed such that a distance equal to five times of embankment width was maintained between the toe of the embankment and the external boundaries of the soil domain, which is assumed to be free in vertical direction and fixed in horizontal direction. The subsurface conditions were determined using three boreholes drilled within the footprint of embankment. The soil stratigraphy comprises clayey-silt-till/silty-clay-till from ground surface to a depth of 10 m (Layer 1). The Layer 1 is underlain by a layer of silt/silty-clay layer to a depth of 20 m (Layer 2). The embankment was constructed using native clayey silt-till/silty-clay-till material. Table 1 summarizes the mechanical properties of foundation soils, embankment fill and the materials that comprise the trench fill.

4 RESULTS AND DISCUSSION

The results of the field test are presented here along with the results of the FE analyses performed to investigate the trends of stress variations measured during the field tests using the estimated soil properties. The accuracy of the numerical analyses is subjected to realistic material property assumptions. The comparisons of the measured and calculated stresses for PPM and GB installations show that despite local differences in the magnitudes of stresses, the similar trends of stresses are captured using the material properties outlined in Table 1. The complex geometry of embankment-trench system and the use of material with different stiffness values around the load cells make the stress regime within the trench very complex both in field test and numerical analyses. Thus, the main objective was to show the influence of the geogrid bridging on the stresses occurring in a trench, rather than presenting an exact stress values that may occur in a conduit installation.

Figure 3 shows the variation of vertical stresses measured at the foundation-embankment interface (cell 5) with the embankment height. As it can be seen from Figure 3, the vertical stresses show a typical increase that is almost linearly proportional to depth of embankment.

4.1 *Stresses Measured at Mid-height of the Trench (1.5 m)*

Figure 4 shows the variation of the vertical and horizontal stresses that were measured at 1.5 m depth in the trench (cells 1and 2, vertical and horizontal pressure cells for PPM; and cells 17 and 18, vertical and horizontal pressure cells for GB) and its comparison to the stresses predicted by the FE analyses.

Table1. The parameters used in the FEA.

	Constitutive Modeling	Unit Weight (kN/m^3)	Modulus of Elasticity	E_{oed} (MPa)	Angle of internal friction
Layer 1	MC	22	130	-	40
Layer 2	MC	22	160	-	42
Embankment Fill	MC	20-22*	95	-	33
Trench Fill	HSM$^+$	20	130	130	36
Lightly compacted surrounding	HSM$^+$	18	20	20	33
Well compacted surrounding	HSM$^+$	22	120	120	35

* unit weight varies with depth
+ in HSM m=0.5 and $E_{ur} = 3E_{50}$

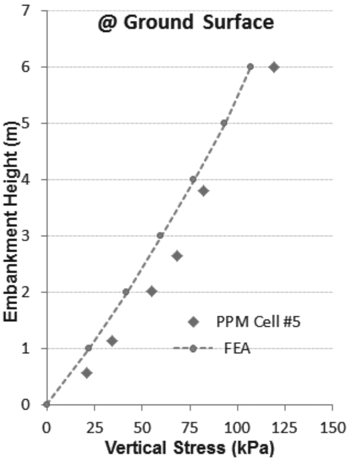

Figure 3. Compression of the FEA results with the vertical stress data obtained from the field test at the foundation-embankment interface.

The results indicate that there is a reasonable agreement between measured and calculated vertical and horizontal stresses. The results shown in Figure 4a indicate that the vertical stresses increased as the embankment height was increased. This increase ranges from 45 kPa to 76 kPa for PPM installation and from 45 kPa to 60 kPa for GB installation for embankment heights of 1m and 6 m, respectively. The results indicate that 22% reduction in vertical stresses was achieved by the use of GB installation when the full embankment height is reached.

The horizontal stress showed an inverse trend. The results shown in Figure 4b indicate that the increased embankment height increased the horizontal stresses from 11 kPa to 29 kPa for PPM installation and from 18 kPa to 33 kPa for GB installation. The results indicate that the use of GB installation increased the horizontal stresses relative to those generated by PPM installation. This increase was as high as 60% at the start of embankment construction. However, the difference weakened as the embankment height increased.

The results also show that both vertical and horizontal stresses measured/calculated at the 1.5 m depth are substantially lower than the values one would practically approximate using the depth of overburden and the unit weigh of material. Such reduction is caused by lightly compacted uniform granular surrounding, which has a substantially lower stiffness that reduced the magnitude of both vertical and horizontal stresses.

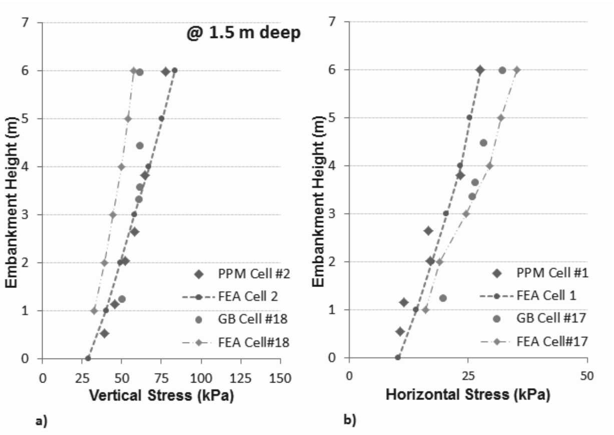

Figure 4. Compression of the FEA results with the vertical and horizontal stresses data obtained from the field test at 1.5 m depth.

4.2 *Stresses Measured at the Bottom of the Trench (3 m)*

Figure 5 shows the variation of the vertical stresses that were measured at 3.0 m depth in the trench (cells 4 for the PPM and 20 for the GB) and its comparison to the stresses predicted by the FE analyses. Pressure Cells #3 and 19 that measures the horizontal stresses were damaged during construction, thus Figure 5b shows only the results of the FEA for these cells. The stresses at this depth are also complex since their magnitude is influenced not only by the installation method (i.e. the existence of the 3 m wide crib in GB) but also the stress arching caused by the lightly compacted surrounding above this location. The measured and calculated stresses show a reasonable agreement at this depth as well.

The results shown in Figure 5a indicate that the measured vertical stresses increased as the embankment height was increased. This increase ranges from 78 kPa to 140 kPa for PPM installation and from 70 kPa to 125 kPa for the GB installation for embankment heights of 1m and 6 m, respectively. The results indicate that 11% reduction in vertical stresses is achieved by the use of GB installation when the full embankment height was achieved.

The results shown in Figure 5b indicate that the increased embankment height increased the horizontal stresses from 23 kPa to 47 kPa for PPM installation. It should also be noted that the difference between the horizontal stresses occurred at PPM and GB cases was not as pronounced at this depth, possibly due to the higher stiffness of well compacted granular surround and as the depth increase the arching effect due to the 3 m wide geogrid bridging softens.

5 SUMMARY AND CONCLUSION

A full scale instrumented test embankment was constructed by Ontario Ministry of Transportation to study the effects of embankment construction on the existing underground utilities. The test embankment comprised four sections which facilitated the evaluation of four configurations including the conventional backfill, induced trenching and two at-grade geogrid reinforcing bridging with different spans. Each configuration consisted of a 3 m deep trench underneath a 10 m wide, 10 m long and 6 m high embankment section. The earth pressure cells were installed to monitor stresses at the fill/ground interface and at the depths of 1.5 m and 3 m. A numerical model of the full scale instrumented test embankment was developed using the finite element program PLAXIS. Both measured and estimated material properties were utilized in the numerical analyses to reproduce the trends of changes in stresses as a result of installation methods. This paper presents the results of stress measurements in a utility trench overlain by an embankment. The measurements that were obtained both during and after construction of a full-scale test embankment and results of numerical modeling that helped clarify mechanisms of stress reduction were presented. The performed analyses showed that geogrid bridging has potential to reduce the stresses on buried infrastructures at shallow depths; however, the magnitude of reduction reduces with depth as the arching effect decreases.

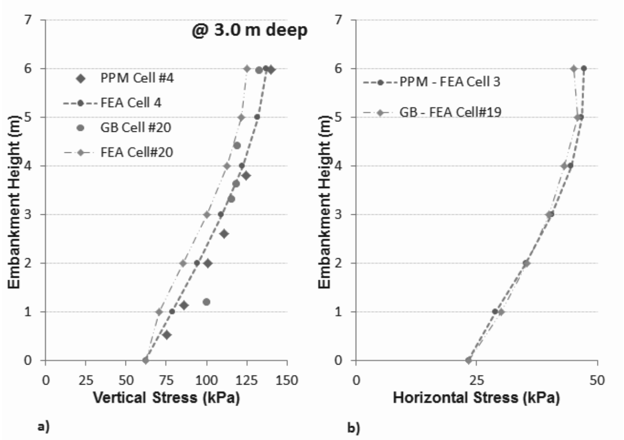

Figure 5. Compression of the FEA results with the vertical and horizontal stresses data obtained from the field test at 3.0 m depth.

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