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Structural Performance of a Medium-Span RC Three-Sided Culvert Based on Field Measurements and Numerical Analyses

Safwat H. Ramadan, MEng¹; M. Hesham El Naggar, Ph. D., P. Eng., FEIC, FASCE²

ABSTRACT

The structural performance and soil-structure interaction mechanism of a 7.3 m span three-sided culvert with flat top slab (TSC) are investigated through field monitoring and numerical analyses. The culvert was instrumented with 6 pressure cells and 14 strain gauges. Data were collected during and after construction. Two-dimensional finite element models were verified against the field measurements. The verified numerical models were then employed to evaluate the effects of the foundation soil condition (yielding or non-yielding) on the applied earth pressures along the culvert profile and the induced straining actions at the base of the culvert's sidewall. Moreover, the structural performance of TSCs at the ultimate loading condition has been evaluated from the numerical model. The results indicated that for TSC supported on yielding foundation soil, significant bending moment develops at the sidewall base and that the assumption of hinged connection at the base is only realistic for the case of TSCs supported on rock (non-yielding foundation). In addition, the precast unit width was found to have minimal influence on the failure mechanism and the ductility of the precast unit. Finally, failure may occur at a lower backfill height for the case of non-yielding foundation relative to the case of yielding foundation.

Keywords: three-sided culvert, field data, numerical analysis, soil-culvert interaction, strip footing.

¹ PhD Candidate, Dep. of Civil and Environmental Engineering, University of Western Ontario, London, Ontario, Canada N6A 5B9. E-mail: sramada6@uwo.ca

² Professor, Dep. of Civil and Environmental Engineering University of Western Ontario, London, Ontario, Canada N6A 5B9 (corresponding author). E-mail: naggar@uwo.ca

INTRODUCTION

Flat top three-sided culverts (TSCs) are bottomless reinforced concrete (RC) culverts typically produced in large spans, up to 16.0 m, and low-rise profiles to handle heavy flows at sites with limited vertical clearance. TSCs are gaining popularity as a sustainable and economical alternative for short span bridge replacement as opposed to multi-barrel box culverts in terms of materials cost, construction time, workforce, and future maintenance costs (Von Handorf 2004). TSCs are also more attractive from the environmental point of view compared to circular pipes, box, and multi-barrel box culverts. Meanwhile, with the absence of the bottom slab that restrains the lateral movement of the sidewalls, differential settlement may occur between the supporting strip footings and the large-span TSC deformation is expected to differ from that of the relatively rigid, short-span box culverts. These features would eventually influence the soil-structure interaction mechanism of TSCs. However, the current design practice treats TSCs same as box culverts in the Canadian Highway Bridge Design Code, CHBDC, (CSA 2014).

Field monitoring and numerical analyses of large-span arch culverts and TSCs with arched top slab, also known as Con/Spans, were discussed in previous studies. Oswald (1996) monitored a 12.3 m span arch culvert with 7.3 m of silty clay backfill cover. Oswald (1996) also conducted a two-dimensional (2D) finite element analysis (FEA) using CANDE software, in which the nonlinear behavior of both backfill soil and concrete were considered. The results demonstrated the importance of simulating concrete creep and shrinkage in the numerical analysis to reasonably simulate the culvert's long-term behavior. McGrath et al. (2002) instrumented a RC arch culvert with 9.1 m span, backfilled with up to 1.5 m soil over its crown. They performed stationary live load tests at various backfill heights to investigate the transferred loads to the buried structure. McGrath et al. (2002) also developed 2D and three-dimensional (3D) FEAs in which the soil behavior was simulated using the Mohr-Coulomb elastic-plastic model with a linear increase of the elastic modulus with depth. A linear-elastic behavior was assumed for the concrete material. Based on the obtained results, they proposed a non-uniform earth pressure distribution on the top slab of the arch

culvert. For Con/Spans, Beach (1988) performed a full-scale load test on a 5.8 m span arched top culvert, where the tested precast unit was covered with 0.3 m (1 ft) of backfill and then loaded to failure using a hand-operated hydraulic jack. It was reported that the Con/Span exhibited a capacity that exceeded five times the design service live loads. Beach (1988) assumed a hinged connection at the base of the sidewall considering the lateral restraint provided by the surrounding soil. McGrath et al. (1996) instrumented an 11.0 m span Con/Span, on which live loads were conducted at different backfill heights, up to 0.9 m. They reported excellent performance of the tested culvert under the applied loads in terms of the measured deflections and crack formations. Zoghi and Farhey (2006) load tested an 11.0 m span Con/Span, covered with 0.9 m of soil when loaded, but could not reach its ultimate capacity. They concluded that the passive resistance provided by the backfill soil surrounding the culvert enhanced the culvert's ultimate capacity. They also reported negligible lateral displacement at the base of the sidewall, which they argued it justified the assumption of hinged connection.

To date, field monitoring of TSC with flat top slab is limited to the study presented by Ramadan and El Naggar (2021) on a 10.4 m span TSC covered with 3.1 m of backfill. Based on field measurements and numerical analysis results, they proposed a vertical arching factor (VAF) of 1.05 for TSCs instead of the 1.2 value stipulated in the CHBDC (CSA 2014) for B1 standard installation. The CHBDC arching factor is based on previous practice and limited soil-structure interaction FEAs. Ramadan and El Naggar (2021) also reported the development of bending moment at the base of the sidewall. They proposed quantifying that bending moment using the estimated footing rotation from the numerical analysis and the computed rotational stiffness from the formula provided by Muskhelishvili (1963) for smooth strip footing. More studies are still required to enhance the understanding of the soil-structure interaction mechanism of this type of buried structures and evaluate the need for developing specific guidelines for their design. Consequently, this paper contributes to the literature of TSC through the following: 1) collection and analysis of field data obtained from an instrumented 7.3 m span TSC under 1.7 m of backfill; 2) 2D finite

element analysis to evaluate the nonlinear behavior of the concrete material at service limit state (SLS); 3) the analysis of the structural behavior and soil-structure interaction mechanism considering the influence of foundation soil condition (yielding or non-yielding) at SLS; 4) a verified 3D FEA model to predict the ultimate capacity of the precast unit of the subject TSC under a specific experimental loading configuration reported in the literature; 5) evaluating the effect of the precast unit width on its load capacity and failure mechanism; and 6) predicting the maximum backfill height at failure of the test TSC for yielding and non-yielding foundations. In this study, the term yielding foundation refers to the case of TSC constructed on the native soil, while non-yielding foundation refers to the case of TSC constructed on rock.

FULL-SCALE TEST

A precast RC TSC was constructed to provide a watercourse crossing through a road embankment in the Greater Toronto Area, Ontario. The culvert was 26.9 m in length, assembled using 11 precast RC units with span of 7.3 m, and total rise of 2.4 m. The geometry and reinforcement details of the used precast units are presented in **Fig.1**.

The native soil at the site consisted of silty clay to clayey silt till with very stiff to hard consistency below the foundation level (FL), which was approximately 6.3 m from the ground surface. The average SPT-N value of the silty clay layer extending from the ground surface and up to 3.9 m below the FL was 25, while a stiffer layer was encountered below that layer with an SPT-N value of 55 as shown in **Fig. 2**. The TSC was supported on a 3.6 m wide keyed cast-in-place strip footing with a total thickness of 1.3 m, including a 0.7 m thick pedestal as shown in **Fig. 3**. The FL was 1.5 m below the streambed elevation due to the presence of the pedestal, which was below the 1.2 m frost depth at the construction site ([OPSD 2010](#)).

The subject TSC was instrumented along one section, where the full backfill height, 3.2 m, would be covering the culvert top slab and the highest live loads would be experienced during the future road

operation. The contact earth pressure imposed on the buried structure was measured using Geokon Model 4810 vibrating wire pressure cells (PCs) and the structural behavior was monitored using Geokon Model 4000 vibrating wire strain gauges (SGs). Model 4810 PC provides an accuracy of 0.1 % of its full-scale range when using the polynomial expression in the data reduction of its measurements (i.e., 0.35 kPa for the used PC with 350 kPa full-scale range). The utilized Model 4000 SG accuracy is 0.5% of its full-scale range (3000 $\mu\epsilon$). The monitoring system was planned based on a preliminary numerical analysis, and the details of the instruments selection criteria and field installation procedure have been presented elsewhere (Ramadan et al. 2021). The culvert was instrumented with 6 PCs and 14 SGs distributed along its span as illustrated in **Fig. 3**.

Culvert Construction Sequence

Construction of the precast RC TSC started with the watercourse diversion to dry the construction work area. Excavation works then commenced to prepare the culvert trench and cast the keyed cast-in-place strip footings, which was underlain by a 0.1 m thick concrete levelling slab. The footing concrete was allowed sufficient time for curing to achieve at least 70% of its design strength (OPSS 2010) before being backfilled. Levelling shims (plastic shims) were placed along each strip footing keyway to guarantee the correct seating of the culvert units, as depicted in **Fig. 4**. Once the precast unit was in place, leg shims (plastic shims) were placed between the culvert sidewall and the keyway sides to secure the precast unit and avoid any lateral spreading. After installing all the precast units, non-shrink grout was poured to fill in the voids between the keyway and the culvert sidewall.

The CHBDC (CSA 2014) specifies different VAFs for vertical earth pressure calculations on RC box culverts based on the soil type and compaction level of the backfill material in the sidefill and bedding zones. The code specifies two standard installations, named B1 and B2, with higher compaction requirements for B1 installation compared to B2 installation for the same type of backfill material. Based

on previous practice and limited soil-structure interaction FEAs, the code stipulates a VAF of 1.20 for B1 installation and 1.35 for B2 installation. Backfill against the subject TSC was completed in lifts of well-graded sand (SW) compacted to 90-100% Standard Proctor, which complies with the construction requirements for B1 installation (CSA 2014). Each backfill lift was less than 300 mm, and the difference in backfill elevation on both sides of the culvert did not exceed 0.6 m at any stage of backfilling to prevent exposing the culvert to excessive lateral deflection. A hand-operated vibrating plate compactor was utilized adjacent to the culvert body until reaching 0.6 m of backfill over the top slab to avoid distressing the culvert body by heavy compaction equipment. The four wing walls of the subject TSC were cast-in-place.

Field Data

The full height of backfill up to the finished road grade level is 3.2 m and is not expected to be reached during this construction season. Accordingly, field measurements were recorded during the backfilling process and up to the final reached cover depth of 1.7 m. All the installed sensors survived the construction activities and the collected measurements from PCs and SGs with construction time to date are presented in **Figs. 5** and **6**, respectively.

Construction of the cast-in-place wing walls started upon reaching a 0.6 m backfill cover on top of the culvert. Accordingly, the culvert was exposed to transient loads due to the wing walls construction activity between days 18 and 105 of construction as shown in **Fig. 5**. These construction activities resulted in damaging SG 6-2 on day 19, which was replaced on day 41. Replacing SG 6-2 required exposing the culvert and removing the soil cover over the SG and near the culvert edge to check the sensors cables, which resulted in sudden drop of the lateral earth pressure measured by the upper pressure cell (PC 2-1) on day 41 of construction as displayed in **Fig. 5a**. The culvert edge was not backfilled until day 96, when the lower pressure cell (PC 1-1) exhibited a sudden drop and PC 2-1 measured an increase in the lateral earth

pressure. The sudden drop in PC 1-1 readings is attributed to the relaxation of the compaction-induced locked-in-stresses in the sidefill material. **Fig. 5a** demonstrates that PC 2-1 exhibited higher magnitudes than PC 1-1 after reaching a backfill height of 1.7 m. The imposed lateral pressure depends on the sidewall deflection relative to the surrounding soil. Ramadan and El Naggar (2021) reported parabolic distribution of the lateral earth pressure along the sidewall of a 10.3 m span TSC based on field measurements and numerical computations. In the current study, the exact pressure distribution along the sidewall can not be determined with only two PCs; however, the collected data indicates that the linear increase of lateral earth pressure with depth stipulated in the CHBDC (CSA 2014) does not represent the actual loading case on the TSC, which agrees with Ramadan and El Naggar (2021) findings as well.

Figs. 5b-c show the vertical earth pressure measurements at four locations on the top slab. During the backfilling from 0.6 m to 1.7 m high, the earth pressure measurements at the top slab exhibited frequent spikes. These spikes resulted from transient loads applied to the culvert while constructing the wing walls as previously discussed. Both PCs located near the far edges of the slab (PC 3-1 and PC 6-1) recorded higher values than the other two PCs, reflecting non-uniform earth pressure distribution on the top slab. Similar studies on TSCs (Ramadan and El Naggar 2021) and arch culverts (McGrath et al. 2002) have reported similar findings.

The measured strains at the culvert's inner and outer surfaces at the instrumented sections are presented in **Fig. 6**. Tensile strains are denoted positive and compressive strains are denoted negative. Visual inspection of the test culvert did not reveal any cracks development on the inner surface of the top slab, which is supported by the measured tensile strains that fall below the theoretical cracking strain ($\epsilon_{cr} = 132 \mu\epsilon$) calculated according to ACI 318-14 (2014) for an average 28-day concrete compressive strength (f'_c) of 57 MPa. Only the tensile strain measured on the top corner of the sidewall by SG 2-2 slightly

exceeded ε_{cr} ; however, cracks development at this zone can not be visually confirmed due to the surrounding sidefill.

NUMERICAL MODEL - SERVICEABILITY LIMIT STATE (SLS)

According to the site topography, the finished road elevation is approximately at the ground level. Thus, the subject TSC was installed in a trench with sloping sides (1:1). Backfilling against the structure was completed in lifts no greater than 300 mm and compacted to 90-100% Standard Proctor in compliance with the construction requirements stipulated by the CHBDC (CSA 2014) for B1 standard installation. The culvert was backfilled with well-graded sand (SW), which consisted of 25% gravel, 73% sand, and 2% fines. The estimated uniformity coefficient ($C_u = D_{60}/D_{10}$) and coefficient of curvature ($C_c = D_{30}^2/(D_{60} \times D_{10})$) were 10.67 and 1.04, respectively. Backfill adjacent to the culvert was compacted using hand-operated vibrating plate compactor and the conducted nuclear density tests indicated an average wet unit weight of 23.0 kN/m³.

2D plane-strain FE models have been created for the monitored culvert using the computer program DIANA 10.1 (2016). The 2D models comprised the native soil, backfill soil, culvert body, and supporting strip footing as illustrated in **Fig. 7**. The native soil, backfill soil, and strip footings were discretized using eight-node quadrilateral plane-strain elements, whilst the culvert body was discretized using plane-stress elements to account for the absence of structural connectivity between the adjacent precast units forming the buried culvert (Pimentel et al. 2009). The model was composed of 1870 plane-stress elements and 23559 plane-strain elements, with the mesh being refined within the vicinity of the culvert body to ensure high accuracy within the studied area.

Fig. 7 shows the geometry and boundary conditions of the 2D model. Based on a conducted sensitivity analysis, the model width was 11 times the TSC span to eliminate the influence of the boundaries on the zone of interest. The model was extended vertically 8.0 m below the strip footing to

represent the native soil, which was divided into two equal layers to account for the increase in strength and stiffness of the till layer reflected in the measured SPT-N values. The model was fixed at its bottom boundary and the side boundaries were restrained from lateral movement. The sidefill and backfill soils were divided into 8 and 7 layers, respectively, to mimic the incremental phases of installation. The overburden stresses of each layer were introduced in a separate phase. Each loading increment was solved iteratively using the Quasi-Newton (Secant) iteration method. Convergence was achieved once the computed out-of-balance force norm and displacement norm ratios were below 0.01 for each loading increment. To date, the backfill height has reached 1.7 m on top of the culvert; however, the full backfill height (3.2 m) was considered in the 2D models.

The effect of simulating the nonlinearity of RC was evaluated by developing two different sets of 2D models: a) linear elastic model (LEA) in which the RC culvert body was simulated using a linear-elastic material model, and b) nonlinear model (NLA) in which a nonlinear material model with crack formation predictions was applied. It is worth mentioning that backfill soil's nonlinear behavior was considered in both sets of models.

The backfill material adjacent to the culvert body was compacted using a hand-operated vibrating plate compactor up to a backfill height of 0.6 m over the top slab. Elshimi and Moore (2013) evaluated the influence of compaction effort on the behavior of two concrete pipes. They concluded that the compaction-induced stresses can be neglected when vibratory plate compactors are used. Accordingly, the effect of compaction during the backfill placement was not considered in the developed models.

Native and Backfill Soils

The behavior of the native clayey silt till was approximated by the elastic perfectly plastic Mohr-Coulomb material model. The empirical correlation proposed by Balachandran et al. (2015) to estimate the elastic modulus (E_s) of clayey silt till using SPT-N values, i.e., $E_s = 1.67 \times N$, was employed for both the very stiff

and hard till layers, resulting in E_s values of 43 MPa and 90 MPa, respectively. The Mohr-Coulomb strength parameters (angle of internal friction, ϕ , and cohesion, c) were adopted from the ranges provided by Cao et al. (2015) for silty clay and clayey silt tills; ϕ ranges between 31° and 41° , and c ranges between 0 and 50 kPa. In accordance, ϕ of 35° and c of 10 kPa were assigned for the very stiff till layer, while ϕ of 37° and c of 30 kPa were assumed for the hard till layer as depicted in Fig. 7.

It is important to account for the nonlinear stress-dependent stiffness of the backfill soil to ensure realistic prediction of applied earth pressures on buried structures. Thus, the hyperbolic model developed by Duncan and Chang (1970) was implemented to simulate the backfill soil behavior. Boscardin et al. (1990) performed an extensive laboratory testing on different backfill materials, e.g., well-graded sand. The particle size distributions of the SW used in backfilling the subject culvert and that tested by Boscardin et al. (1990) were very similar. Accordingly, the Duncan-Chang parameters reported by Boscardin et al. (1990) for SW compacted to 90% of the Standard Proctor density (SW-90) were assigned for the backfill material. The assigned parameters are as follow: modulus number (K) = 640, modulus exponent (n) = 0.43, failure ratio (R_f) = 0.75, Poisson's ratio (ν) = 0.3, angle of internal friction (ϕ) = 42° , cohesion (c) = 0.1 kPa, and density (γ) = 23 kN/m³.

Reinforced Concrete

The 2D FE models comprised two different RC parts, i.e., the strip footing and the culvert body. The strip footing is designed to remain in the elastic range, and nonlinearity of RC is not expected. Accordingly, the strip footing was assigned linear-elastic behavior in all developed models with an average 28-day compressive strength (f'_c) of 40 MPa, elastic modulus (E_c) of 29.7 GPa (fib Model Code 2010), and ν of 0.15.

The precast RC culvert had an average f'_c of 57 MPa. Linear-elastic behavior was assumed for the RC culvert in LEA model. An equivalent E_c of 35.5 GPa and ν of 0.15 were assigned. On the other hand,

the total strain-based smeared crack model with fixed crack criteria was adopted in NLA model to simulate the culvert's nonlinear behavior and predict the formation of cracks. The compressive strain-softening behavior was simulated using the Thorenfeldt model (1987), and the Hordijk nonlinear model (1991) was used to represent the strain-softening behavior under tension loading. The smeared crack model parameters were estimated based on the formulae stipulated in the fib Model Code 2010 (2013) for normal weight concrete and are summarized in **Table 1**.

Embedded bar elements, which are fully bonded to the host elements, were utilized in NLA model to represent the steel reinforcement detailed in **Fig. 1**. The mechanical behavior of the steel reinforcement was simulated using the von Mises plasticity model with yielding stress (f_y) of 450 MPa and Young's modulus (E_{st}) of 200 GPa. The steel hardening behavior was ignored in the analysis.

Interfaces

Line interface elements were introduced at the soil-footing, soil-culvert, and culvert-footing interfaces. The soil-footing interface comprised contact surfaces between the foundation soil and footing base (footing base interface) and the footing sides and sidefill material (footing sides interface). At the culvert-footing interface, the sidewall was secured in the strip footing keyway, then non-shrink grout was poured to fill in the voids between the keyway and sidewall. The friction behavior of the interface elements followed the Mohr-Coulomb yielding criterion and the displacement across the interface was governed by the defined normal and tangential stiffnesses, D_{nn} and D_{tt} , respectively. The interface properties were estimated according to the following formulae (TNO DIANA BV 2016):

$$D_{tt} = \frac{A^2 E_s}{t 2(1 + \nu_s)} \quad ; \quad D_{nn} = f \times D_{tt} \quad (1)$$

$$c_{int} = A \times c \quad ; \quad \phi_{int} = A \times \phi$$

where A is a reduction factor (0.67), f is a multiplication factor (50), t is the virtual thickness of the interface (10 mm), E_s is the soil Young's modulus, ν_s is the soil Poisson's ratio, c_{int} is the interface cohesion, and ϕ_{int} is the interface friction angle. The backfill soil properties were employed to estimate the interface properties at the soil-culvert and footing sides interfaces. The interface properties at the footing base interface were estimated using the foundation soil properties, either very stiff till in the case of yielding foundation or rock in the case of non-yielding foundation. The grout properties reported by Samaiklang and Fuenkajorn (2013) for a cement grout with a compressive strength of 27.64 MPa, which is comparable with the 30 MPa of the grout used during construction, were used to determine the culvert-footing interface properties. The reported grout properties were: $E = 3.56$ GPa, $\phi = 40^\circ$, and $c = 5.42$ MPa (Samaiklang and Fuenkajorn 2013).

Model Verification

The field data recorded by the SGs and PCs were utilized to verify the developed FE models (LEA and NLA). Representative values of contact earth pressures and strain values at 1.7 m of backfill were selected after the readings stabilized. The field measurements stabilized 2 days after reaching the 1.7 m of backfill, day 107, as presented in **Figs. 5** and **6**. Accordingly, the average readings of pressure cells and strain gauges after day 107 were used in the comparison with the numerical results. **Fig. 8** compares the measured and calculated earth pressures acting on the culvert surfaces as well as the values estimated based on the CHBDC (CSA 2014) provisions for the two standard installations (B1 and B2 installations). The predicted vertical earth pressures on the top slab from LEA and NLA FE models almost coincided. The numerical models slightly underestimated the earth pressures on the top slab compared to the field measurements with a maximum difference of 12% at the far-right edge of the culvert; however, they demonstrated the same trend of the measured values. The earth pressure on the top slab was non-uniformly distributed with higher magnitudes at the stiffer edges compared to the relatively flexible mid-span.

Similar findings were reported in previous studies on box culverts (Chen and Sun 2014), arched culverts (McGrath et al. 2002), and TSCs (Ramadan and El Naggar 2021). The installation of the subject TSC followed the specifications listed in the CHBDC (CSA 2014) for B1 installation, even though both the field measurements and analytical results in **Fig. 8a** indicated that almost 80% of the top slab experienced earth pressures lower than the code values. Similarly, Ramadan and El Naggar (2021) reported that the specifications stipulated in the CHBDC (CSA 2014) overestimated the applied earth pressures on the top slab of a 10.4 m span TSC covered with 3.1 m of backfill.

The calculated lateral earth pressures on the culvert sidewall agree well with the field measurements as presented in **Fig. 8b**. There are some differences between the measured and calculated values due to the simplifications and assumptions in the numerical analysis compared to the exact field conditions. The numerical results exhibited a parabolic distribution of the lateral earth pressure, which has been reported in previous studies on box culverts (Chen and Sun 2014; Oshati et al. 2012) and a similar study on a TSC (Ramadan and El Naggar 2021). The differences between the estimated pressures from LEA and NLA models are trivial. Thus, it can be claimed that the consideration of RC nonlinear behavior for estimating the applied earth pressures on the buried structure at SLS is not essential and simplifying the analysis by defining a linear elastic behavior is sufficient at such level of loading. It can be recognized from **Fig. 8b** that the measured and computed values of the lateral earth pressure almost lie within the bounds stipulated in the CHBDC (CSA 2014) with minimum and maximum horizontal arching factors of 0.3 and 0.5 for B1 installation, respectively. Therefore, the load combinations considered in the design of these structures, which include the minimum and maximum values at separate combinations, are expected to cover the actual applied lateral earth loads experienced by the buried structure.

The measured tensile and compressive strains at each instrumented cross-section were utilized to further verify the FE models. Visual inspection of the culvert's inner surfaces did not show any signs of cracks, which was corroborated by the predictions of NLA model and the SG measurements that did not

exceed the theoretical cracking strain ($\varepsilon_{cr} = 132 \mu\epsilon$). Consequently, the analysis proceeded with the assumption of uncracked sections and the bending moment (BM) at each cross-section was calculated using Eq. 2 as follows:

$$BM = \frac{E_c I_g (\varepsilon_i - \varepsilon_e)}{d} \quad (2)$$

where I_g is the gross moment of inertia; ε_i and ε_e are the strains measured on the culvert's inner and outer surfaces, respectively; and d is the section depth. As per Eq. 2, the BM is considered positive when the tensile stresses are located on the culvert's inner surface.

BM is not a direct output of either plane-stress or plane-strain elements. Consequently, composed line elements were implemented in LEA and NLA models to estimate the BM by integrating the induced internal stresses in the plane-stress elements of the culvert body in post-processing of analysis results (TNO DIANA BV 2016).

Fig. 9 displays the computed BM s from the SGs measurements and those numerically computed from LEA and NLA models. Additionally, a FE plane frame analysis was conducted using SAP 2000 (2020) in which the measured earth pressures presented in **Fig. 8** were utilized. The base connection of the sidewall was assumed to be hinged in the plane frame analysis, as typically assumed in the current design practice. The results indicated excellent match between the BM s estimated from the SGs and PCs measurements, which confirm the integrity of the monitoring system. **Fig. 9a** shows that the computed BM s across the top slab from NLA model were in better agreement with that estimated from the field measurements compared to LEA model, which slightly overpredicted the BM at the mid-span. On the other hand, LEA model results better matched the estimated BM from the SGs data along the sidewall as depicted in **Fig. 9b**. Consequently, it can be interpreted that the BM magnitudes from LEA would lead to a conservative design.

The BM distribution along the sidewall obtained from the plane frame analysis exhibited zero BM at the base of the sidewall as depicted in **Fig. 9b**. This output is attributed to the definition of a hinged condition at the base of the sidewall. Conversely, the SGs measurements and numerical results indicated non-zero BM at the sidewall base. Similar observations were reported by Ramadan and El Naggar (2021). They attributed this BM at the base to the rotation of the strip footing along with the connection condition at the base, which results in additional stresses in the enclosed depth of the sidewall in the strip footing keyway, thus, the development of BM at the base. This explanation will be further inspected in the following subsection.

Effect of Foundation Soil Condition

The LEA numerical model, which was verified for the analysis at service loading conditions, was utilized to conduct a parametric study to elaborate on the influence of the foundation soil condition, either yielding or non-yielding, on the applied earth pressures on TSCs, stress distribution underneath the strip footing, and the induced straining actions in the culvert body. The FE model for yielding foundation was denoted LEA-Y and that for non-yielding foundation was denoted LEA-NY. The existing native till layer represented the case of yielding foundation, while rock was assumed for the non-yielding foundation case. The behavior of the competent rock was simulated using the Mohr-Coulomb material model and the assigned properties were adopted from Yoo et al. (2005); listed as follows: $E_s = 34.3$ GPa, $\nu = 0.25$, $\phi = 0^\circ$, and $c = 6.9$ MPa.

Fig. 10 shows the applied earth pressures on the culvert surface and the lateral deflections of the sidewall for both conditions of yielding and non-yielding foundation soils at 3.2 m of cover depth. The computed earth pressure distributions on the top slab displayed in **Fig. 10a** show insignificant influence of the subsurface condition, where both LEA-Y and LEA-NY models almost exhibited similar earth pressures. This contradicts with the findings of Kim and Yoo (2005) in their study on box culverts, which

stated that the applied earth pressures on the top slab were much higher for the case of non-yielding foundation compared to the case of yielding foundation. Similar to the results obtained at 1.7 m of backfill, the majority of the top slab is expected to experience earth pressures lower than the code value for B1 installation as shown in **Fig. 10a**. The effect of the foundation soil properties on the lateral earth pressure applied on the sidewall is shown to be significant in **Fig. 10b**. Although the sidewall lateral deflection towards the sidefill material was much higher in the case of yielding foundation soil compared to the case of non-yielding foundation as elaborated in **Fig. 10c**, higher lateral earth pressures were computed for the case of non-yielding foundation with values at the base of the sidewall being the only exception. This is attributed to the higher confinement of the compacted sidefill material when surrounded by rock as opposed to the yielding native soil. It can also be noticed from **Fig 10b** that the computed lateral earth pressures in LEA-Y and LEA-NY models almost fall within the bounding values of the CHBDC ([CSA 2014](#)). The stress distributions underneath the strip footing for the cases of yielding and non-yielding foundations are presented in **Fig. 11a**. The results show that the stresses are almost uniform in the case of non-yielding foundation, where it varies between 170 kPa at the backfill side and 203 kPa at the waterside. On the other hand, the stresses are almost uniform over 70% of the strip footing in the case of yielding foundation, with an average value of 181 kPa, then significantly increases to a maximum value of 273 kPa over the remaining 30% on the waterside. Generally, higher stresses are expected at the waterside for both cases of yielding and non-yielding foundations.

Fig. 11b shows the *BM* distribution along the sidewall. The results of the developed *BM* at the sidewall base indicate that *BM* develops due to the rotation of the strip footing; the *BM* developed at the sidewall base for the case of yielding foundation but did not develop for the case of non-yielding foundation, in which the rotation of the footing is restrained. Therefore, the assumption of pure hinged condition, usually considered in the current design practice of TSCs, is realistic for the case non-yielding

foundation. However, for yielding foundation, BM develops due to the rotation of the strip footing and should be accounted for in the design.

NUMERICAL MODEL – ULTIMATE LIMIT STATE (ULS)

The ultimate capacity of an 11.0 m span RC Con/Span has been experimentally measured through load testing one precast unit to failure by Marshall et al. (2014). The results of the load test were utilized to verify a three-dimensional (3D) FE model, which was then used to predict the ultimate capacity of the precast units used in constructing the TSC analyzed in this study. Moreover, the 2D numerical models discussed above were modified to evaluate the structural performance and soil-structure interaction mechanism at the ultimate limit state (ULS) for yielding and non-yielding foundations.

Capacity of Precast Units

Marshall et al. (2014) load tested an 11.0 m span precast unit with a mid-span height of 2.7 m and 1.2 m in width. The precast unit rested on rigid floor and was restrained from lateral movement at the base of its sidewalls. Loading was applied incrementally on the top slab of the Con/Span through three W8×35 beams spanning the width of the precast unit as shown in **Fig. 12a**. Each load increment was 17.8 kN/beam until the maximum load was reached. The concrete compressive strength, f'_c , was 48.5 MPa and the elastic modulus, E_c , was 28.3 GPa. The yielding strength of steel rebars, f_y , was 507 and 642 MPa depending on the size of bars, and their elastic modulus, E_{st} , was 182.7 GPa (Marshall et al. 2014).

A 3D FE model was developed for the Con/Span with the same reported geometry, concrete and steel properties, steel reinforcement, and loading configuration. The precast unit was discretized using 20-node soil brick elements and the steel rebars were simulated using embedded elements. Similar to NLA model previously described, smeared crack material model was used for the concrete and von Mises plasticity model was used for the reinforcing steel. The precast unit was fixed from vertical movement at its base and horizontal movement at the outer edge of the sidewall base as shown in **Fig. 12a**. Vertical

stresses were applied at the loading beams locations along the precast unit width in increments of 71.7 kPa (i.e., corresponding to 17.8 kN load increment applied to the contact surface of loading beams, $1.2 \times 0.204 \text{ m}^2$). The same analysis procedure was repeated on the precast unit of the subject TSC, as shown in **Fig 12b**, with the same geometry and reinforcement details as in **Fig. 1**. Moreover, the material properties of concrete and steel reinforcement were the same as in the 2D NLA model.

The 3D model of the Con/Span precast unit was verified by comparing its predictions with the measured load-deflection curves at the mid-span and the top corner of the sidewall as shown in **Fig. 13a**. Downward vertical deflection at the mid-span and outward lateral deflection at the corner are considered positive. As can be observed from **Fig. 13a**, the measured and calculated ultimate load capacity of the precast unit agree well, where a value of 675.7 kN was measured and 673.0 kN was predicted by the FE model. Nevertheless, the computed vertical deflection at the mid-span that corresponds to the ultimate load (90.5 mm) was much lower than the measured 150 mm. Assuming full bond between the concrete and steel bars as well as the stress-locking behavior that occurs in displacement-based FE analyses with smeared softening approach ([Rots and Blaauwendraad 1989](#)) might explain the calculated stiffer behavior. The computed outward lateral deflection at the top corner of the sidewall matched well with the measured values throughout the whole loading stages.

The ductility factor (μ) was evaluated from the load-deflection curves at the mid-span as the ratio between the ultimate displacement (Δ_{ult}), corresponding to the ultimate load, and the yield displacement (Δ_y), corresponding to the development of 75% of the ultimate capacity. The μ values obtained from the experiment data and FE model are 1.71 and 1.89, respectively.

Fig. 13b shows the deformed shape and cracks pattern predicted by the 3D model. The numerical model predicted yielding of the steel reinforcement at three different locations leading to the formation of plastic hinges as presented in **Fig. 13b**. This explains the increase of vertical deflection at the mid-span

without remarkable increase in the load prior to failure. Interestingly, Marshall et al. (2014) reported yielding of the steel reinforcement at the failure surface, almost located at plastic hinge-A shown in **Fig. 13b**, before the concrete crushed. Additionally, Beach (1988) reported the formation of three-plastic hinges in his testing program of a Con/Span culvert. The three plastic hinges occurred at the same locations predicted by the 3D numerical model presented in **Fig. 13b**.

The load-deflection curve, deformed shape, and crack pattern of the test TSC are displayed in **Fig. 14**. A brittle behavior can be noted from the load-deflection curve at the mid-span; **Fig. 14a** shows that crushing of concrete occurred at failure while steel reinforcement did not reach its yield strength. This results in a low vertical deflection at the ultimate load; 20.9 mm. Moreover, the estimated μ is 1.38, which is 37% lower than the estimated value for the Con/Span indicating the lower ductility of the subject TSC compared to arched top culvert. The top corner of the sidewall experienced negligible lateral movement up to failure as shown in **Fig. 14a**. Failure of the subject TSC is predicted just below the culvert haunch at the top corner of the sidewall, where the highest concrete crack strain is anticipated as depicted in **Fig. 14b**. Such findings agree with the predicted failure surface of a box culvert under high embankment by Pimentel et al. (2009) and that predicted for a 10.4 m span TSC under the same loading condition as the subject TSC (Ramadan and El Naggar 2021).

Effect of Precast Unit Width

Precast RC units are produced in widths varying between 1.0 and 2.4 m. The width (W) of the precast unit is governed by many factors, including: 1) total length of the culvert, 2) size and delivery vehicle payloads restrictions on major roads, and 3) handling loads and lifting equipment limits. This section discusses the influence of the precast unit width on its ultimate capacity, ductility, and failure mechanism. A parametric study was conducted employing 3D nonlinear FE models considering four different precast unit widths: 1.0, 1.5, 2.0, and 2.4 m. The four FE models had the same boundary conditions and loading configuration

as that presented in **Fig. 12**. Additionally, the steel reinforcement per meter width was kept the same in the longitudinal and transversal directions.

Fig 15a displays the calculated vertical load-deflection relations at the mid-span. The results demonstrate the expected increase of ultimate capacity as the unit width increases. However, the peak load seems to occur at higher deflection levels for smaller unit widths as summarized in **Table 2**. To benchmark the results against each other, the total load was normalized by the corresponding unit width. The normalized load-deflection curves are presented in **Fig. 15b**. It is noted from **Fig. 15b** that the initial linear behavior is identical for the four cases up to a load of 180 kN/m, beyond which stiffness degrades due to the formation of cracks. **Fig 15b** demonstrates that the ultimate load capacity per unit width and the corresponding deflection increase as the precast unit width decreases. In terms of ductility, **Table 2** provides a summary of the results for all four cases. The results suggest minimal influence of the precast unit width on the ductility with a maximum variation of 5% between the considered widths. Eventually, the precast unit width has no influence on either the failure mechanism or the predicted failure surface.

Backfill Height at Failure

The investigated TSC was designed for a total backfill height of 3.2 m. The maximum backfill height at which the TSC would fail has been predicted employing the verified numerical models and extending the backfill height to 10.0 m. The influence of the concrete nonlinear behavior and the subsurface condition (yielding and non-yielding) on the maximum height at failure were investigated.

Fig. 16 displays the vertical load-deflection curve at the mid-span and the lateral load-deflection curves at the corner and mid-height of the sidewall (denoted as Side-M). As expected, the linear-elastic model, LEA model, exhibited linear load-deflection curves up to 10.0 m of backfill. Conversely, NLA model demonstrated stiffness degradation and higher deflections at the mid-span beyond 3.6 m of backfill height due to the formation of cracks. For the case of yielding foundation, presented in **Fig. 16a**, failure is

expected to occur at a backfill height of 9.2 m, which is 2.8 times the design backfill height. Ductile failure is expected for the TSC at the 9.2 m of backfill, where yielding of the steel reinforcement occurred prior to failure resulting in noticeable increase in the culvert deflection with minimal increase in the backfill height. This failure mechanism is an inverse of the predicted brittle mode of failure under the laboratory loading configuration previously discussed. Such improvement in the structural performance of the RC culvert can be attributed to the confinement provided by the surrounding soil, which agrees with previous findings on TSCs ([Ramadan and El Naggar 2021](#)) and box culverts ([Moradi et al. 2016](#); [Marshall et al. 2014](#); [Zoghi and Farhey 2006](#)).

Similar observations can be interpreted for the case of non-yielding foundation from **Fig. 16b**. However, failure is expected to occur at a lower backfill height, 8.6 m, and lower vertical and lateral deflections are estimated compared to the case of yielding foundation. For instance, the maximum mid-span vertical deflections at failure are 27.6 and 21.6 mm for yielding and non-yielding foundations, respectively.

CONCLUSIONS

Field data obtained from an instrumented 7.3 m span TSC covered with 1.7 m of well-graded sand were presented and discussed. The recorded pressure cells' and strain gauges' measurements were employed to verify 2D FE models, which were then utilized to evaluate the effect of nonlinear behavior of RC on the culvert's structural performance and soil-structure interaction mechanism. The verified models were used to investigate the effect of foundation soil condition (yielding or non-yielding) on the culver behaviour. Moreover, experimental data of a Con/Span precast unit loaded to failure was utilized to verify a 3D finite element model and investigate the structural performance of the subject TSC at ultimate limit state. The 3D model was replicated for the test TSC and the influence of the precast unit width was investigated. Eventually, the 2D FE analysis was extended to predict the maximum backfill height at which failure of

the TSC is expected for the cases of yielding and non-yielding foundations. The following conclusions can be drawn from the conducted analyses:

1. For the studied TSC, the vertical earth pressure is non-uniformly distributed along the top slab with 80% of the top slab experiencing earth pressures lower than the value specified in the CHBDC (CSA 2014) for B1 standard installation. On the other hand, the measured and estimated lateral earth pressures along the sidewall fell within the minimum and maximum bounds of the code for the same standard installation method.
2. Bending moment was developed at the base of the sidewall due to the footing rotation for yielding foundation, while negligible bending moment was estimated at the base for the case of non-yielding foundation. Thus, the assumption of hinged support at the base is suitable for the case of non-yielding foundation. However, bending moment develops at the sidewall base for yielding foundation condition and should be considered in design.
3. The concrete behavior can be simulated by the linear-elastic material model to reasonably estimate the applied earth pressures on TSCs at service loading conditions.
4. The foundation soil condition has negligible effect on the estimated vertical earth pressure on the top slab. However, the estimated lateral earth pressure on the sidewall was much higher for the case of non-yielding foundation compared to the case of yielding foundation.
5. The precast unit width has no influence on the ductility, failure mechanism, or predicted failure surface. However, the ultimate load capacity per unit width and the corresponding mid-span vertical deflection increase as the unit width decreases.
6. The subject TSC is expected to exhibit ductile failure as the backfill height increases to 9.2 m in case of yielding foundation and backfill height of 8.6 m in case of non-yielding foundation. Moreover, the estimated maximum mid-span deflection at failure is higher in case of yielding foundation as opposed to the non-yielding foundation.

REFERENCES

- ACI Committee 318. (2014). "Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)." American Concrete Institute, Farmington Hills.
- Balachandran, K., Liu, J., Cao, L. and Peaker, S. (2015). "Statistical correlations between pressuremeter modulus and SPT N-value for glacial tills." Proc. 68th Canadian Geotechnical Conference and 7th Canadian Permafrost Conference, Quebec.
- Beach, T. J. (1988). "Load test report and evaluation of a precast concrete arch culvert." Transportation Research Record 1191, Transportation Research Board, Washington, DC, 12–21.
- Boscardin, M.D., Selig, E.T., Lin, R.S. and Yang, G.R. (1990). "Hyperbolic parameters for compacted soils." J. Geotech. Eng. Am. Soc. Civ. Eng., 116(1), 88-104.
- Canadian Standards Association (CSA). (2014). "Canadian Highway Bridge Design Code." CAN/CSA-S6-14, Mississauga, ON, Canada.
- Cao, L., Liu, J., Peaker, S. and Ahmad, S. (2015). "Engineering characteristics of glacial tills in GTA." Proc. 68th Canadian Geotechnical Conference and 7th Canadian Permafrost Conference, Quebec.
- Chen, B., and Sun, L. (2014). "Performance of a reinforced concrete box culvert installed in trapezoidal trenches." J. Bridge Eng., Am. Soc. Civ. Eng., 19(1), 120–130.
- CSI, 2020. "SAP2000 Integrated Software for Structural Analysis and Design," Computers and Structures Inc., Berkeley, California.
- Dasgupta, A., and Sengupta, B. (1991). "Large scale model test on square box culvert backfilled with sand." J. Geotech. Eng. Viv., Am. Soc. Civ. Eng., 117(1), 156–161. [10.1061/\(ASCE\)0733-9410 \(1991\) 117:1\(156\)](#)
- Duncan, J. M., Chang C. Y. (1970). "Nonlinear analysis of stress and strain on soils." J. Soil Mech. Found., 96(5), 1629-1653.

- Hordijk, D.A. (1991). "Local approach to fatigue of concrete." Doctor of Philosophy, Ph.D. dissertation, Delft University of Technology, Delft, The Netherlands.
- International Federation for Structural Concrete. (2013). "Fib model code for concrete structures 2010." Berlin: Verlag Ernst & Sohn.
- Kim, K., and Yoo, C.H. (2005). "Design Loading for Deeply Buried Box Culverts." J. Geotech. Geoenviron. Eng., ASCE, 131(1), 20-27. [10.1061/\(ASCE\)1090-0241\(2005\)131:1\(20\)](https://doi.org/10.1061/(ASCE)1090-0241(2005)131:1(20)).
- Marshall, J. D., Anderson, J. B., Meadows, R. L., and Jensen, T. J. (2014). "Full-scale testing of three-sided precast concrete arch sections." J. Bridge Eng. 19(12):04014051. [10.1061/\(ASCE\)BE.1943-5592.0000630](https://doi.org/10.1061/(ASCE)BE.1943-5592.0000630)
- McGrath, T. J., Moore, I. D., Selig, E. T., Webb, M. C., and Taleb, B. (2002). "Recommended specifications for large-span culverts." Rep. No. 473, Transportation Research Board, Washington, D.C.
- McGrath, T. J., Selig, E. T., and Beach, T. J. (1996). "Structural behavior of three-sided arch span bridge." Transportation Research Record 1541, Transportation Research Record, Washington, DC, 112–119.
- Moradi, M., Valipour, H., and Foster, S. (2016). "Reserve of strength in inverted u-shaped RC culverts: effect of backfill on ultimate load capacity and fatigue life." J. Bridge Eng., Am. Soc. Civ. Eng., 21(2): 04015051. [https://doi.org/10.1061/\(ASCE\)BE.1943-5592.0000800](https://doi.org/10.1061/(ASCE)BE.1943-5592.0000800)
- Muskhelishvili, N.I., (1963). "Some Basic Problems of the Mathematical Theory of Elasticity." P. Noordhoff, Groningen, The Netherlands.
- Ontario Ministry of Transportation (2010). "Frost Penetration Depths for Southern Ontario" OPSD 3090.101 rev.1.
- Ontario Ministry of Transportation (2010). "Construction Specification for Excavating and Backfilling-Structures" Ontario Provincial Standard Specification OPSS 902.
- Oshati, O.S., Valsangkar, A.J., and Schriver, A.B. (2012). "Performance of two cast-in-place box culverts under high embankments." Can. Geotech. J., 49(12), 1331–1346. [10.1139/t2012-094](https://doi.org/10.1139/t2012-094)

- Oswald, C. J., (1996). "Analysis of reinforced concrete culvert considering concrete creep and shrinkage." Transportation Research Record 1541, TRB. National Research Council. Washington, D. C., 120–126.
- Pimentel, M., Costa, P., Félix, C., and Figueiras, J. (2009). "Behavior of reinforced concrete box culverts under high embankments." J. Struct. Eng. [10.1061/\(ASCE\)0733-9445\(2009\)135:4\(366\)](https://doi.org/10.1061/(ASCE)0733-9445(2009)135:4(366)), 366–375
- Ramadan, S. H., and El Naggar, M. H. (2021). "Field Monitoring and Numerical Analysis of a Large Span Three-Sided Reinforced Concrete Culvert." J. Geotech. Geoenviron. Eng., Am. Soc. Civ. Eng., 147(4): 04021008. [https://doi.org/10.1061/\(ASCE\)GT.1943-5606.0002489](https://doi.org/10.1061/(ASCE)GT.1943-5606.0002489)
- Ramadan, S. H., El Naggar, M. H., Drbe, O., and Embaby, K. (2021). "Preliminary analysis and instrumentation of large-span three-sided reinforced concrete culverts." Submitted for possible publication in Can. Geotech. J.
- Rots, J. G., and Blaauwendraad, J. (1989). "Crack models for concrete: Discrete or smeared? Fixed, multi-directional or rotating?" Heron, 34(1), 5–55.
- Samaiklang, W., and Fuenkajorn, K. (2013). "Mechanical and hydraulic performance of cement grouts from 5 suppliers in Thailand." Proc. of the Second Thailand Symposium on Rock Mechanics (ThaiRock 2013), Im Poo Hill Resort, Nakhonratchasima, 333–342.
- TNO Diana BV (2016): DIANA—User's Manual. Material library, Delft, The Netherlands.
- Thorenfeldt, E., Thomaszewic, A., and Jensen, J. J. (1987). "Mechanical properties of high-strength concrete and applications in design." Proc., Symp. on Utilization of High-Strength Concrete, Tapir Publishers, Stavanger, Norway, 149–159.
- Von Handorf, J. J. (2004). "Clear-span culverts: Economical designs." Better Roads, Feb. 2004.
- Yoo, C. H., Parker, F., and Kang, J. (2005). "Bedding and Fill Heights for Concrete Roadway Pipe and Box Culverts." Final Report, ALDOT Project No. 930-592, Highway Research Center, Auburn University, AL.
- Zoghi, M., and Farhey, D. N. (2006). "Performance assessment of a precast concrete, buried, small arch

bridge.” J. Perform. Constr. Facil. [10.1061/\(ASCE\)0887-3828\(2006\)20:3\(244\)](https://doi.org/10.1061/(ASCE)0887-3828(2006)20:3(244)), 244–252

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TABLE 1 Input parameters for the concrete smeared crack model in NLA model.

Compressive strength	Young's Modulus	Tensile strength	Fracture energy	Poisson's ratio
f'_c	E_c	f_{ct}	G_f	ν
(MPa)	(GPa)	(MPa)	(Nmm/mm ²)	-
57.0	40.1	5.9	0.202	0.15

TABLE 2 Results summary of the 3D nonlinear models.

Precast unit	Ultimate load (kN)	Ultimate load/unit width (kN/m)	Δ_{ult} (mm)	Δ_y (mm)	Ductility, μ
Arched TSC (Exp)	675.7	563.1	150.0	87.8	1.71
Arched TSC (FEA)	672.0	560.0	91.1	48.1	1.89
TSC (W = 2.4 m)	1273.0	530.4	20.9	15.2	1.38
TSC (W = 2.0 m)	1104.5	552.3	21.6	16.1	1.34
TSC (W = 1.5 m)	858.0	572.0	23.1	17.3	1.34
TSC (W = 1.0 m)	581.0	581.0	22.9	17.5	1.31

LIST OF FIGURE CAPTIONS

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Fig. 12. Overview of the 3D FE models: a) TSC with arched top slab; and b) TSC of this study.

Fig. 13. Arched TSC results: a) Load-deflection curves; and b) deformed shape and crack pattern.

Fig. 14. TSC results: a) Load-deflection curves; and b) deformed shape and crack pattern.

Fig. 15. Effect of precast unit width: a) total load; and b) total load/unit width.

Fig. 16. Deflection of the TSC at different backfill heights: a) yielding soil; and b) non-yielding soil.

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Fig. 1. Geometry and reinforcement details of the TSC. (All dimensions are in mm)

181x73mm (300 x 300 DPI)

Fig. 2. Variation of SPT-N value with depth.

85x88mm (300 x 300 DPI)

Fig. 3. The TSC instrumentation layout: a) pressure cells; b) strain gauges; and c) instrumented segment. (All dimensions are in m)

181x106mm (300 x 300 DPI)

Fig. 4. Installation of the culvert precast units.

85x71mm (300 x 300 DPI)

Fig. 5. Sample pressure cell measurements: a) pressures on the sidewall; b) pressures on the top slab (PC 3-1 & PC 4-1); and c) pressures on the top slab (PC 5-1 & PC 6-1).

181x156mm (300 x 300 DPI)

Fig. 6. Sample strain gauge measurements.

181x200mm (300 x 300 DPI)

Fig. 7. Geometry, boundary conditions, and meshing details of the 2D FE model.

181x47mm (300 x 300 DPI)

Fig. 8. Comparison between measured and estimated pressures on the culvert surfaces at 1.7 m of backfill: a) vertical earth pressure on the top slab; and b) lateral earth pressure on the sidewall.

181x84mm (300 x 300 DPI)

Fig. 9. Comparison between measured and estimated bending moments at 1.7 m of backfill: a) top slab; and b) sidewall.

181x85mm (300 x 300 DPI)

Fig. 10. Effect of foundation soil condition on the applied earth pressure and sidewall deflection at 3.2 m of backfill: a) vertical pressure on the top slab; b) lateral pressure on the sidewall; and c) lateral deflection of the sidewall.

142x167mm (300 x 300 DPI)

Fig. 11. Effect of foundation soil condition at 3.2 m of backfill: a) stresses underneath the footing; and b) bending moment along the sidewall.

181x88mm (300 x 300 DPI)

Fig. 12. Overview of the 3D FE models: a) TSC with arched top slab; and b) TSC of this study.

85x101mm (300 x 300 DPI)

Fig. 13. Arched TSC results: a) Load-deflection curves; and b) deformed shape and crack pattern.

85x89mm (300 x 300 DPI)

Fig. 14. TSC results: a) Load-deflection curves; and b) deformed shape and crack pattern.

85x93mm (300 x 300 DPI)

Fig. 15. Effect of precast unit width: a) total load; and b) total load/unit width.

181x67mm (300 x 300 DPI)

Fig. 16. Deflection of the TSC at different backfill heights: a) yielding soil; and b) non-yielding soil.

181x70mm (300 x 300 DPI)