ULTRACOR – 1-ST REALIZATION IN EUROPE, DESIGN, ERECTION, TESTING[1]

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This paper presents design, realization and testing process of 1-st UltraCor structure in Europe. The scope of the design and intalation is 25.50 m Span, 9.00 m Rise structural plate structure of 500x237mm corrugation with only 9.5 mm plate thickness. Structure will cary the heavy traffic of S7 express road. It was designed for the highest class of loads acc. to Polish Standard. Under the structure there is a local raod and animal crossing.

Key words: Buried steel structures, underpass, large span, Full scale test

1. INTRODUCTION

With the continuous increase in infrastructure needs and the existing constrained public budgets there is an increased emphasis on innovative solutions to build road and railway infrastructure. Flexible buried bridges have been an integral part of the Canadian infrastructure for decades. It has been started to be popular in Poland since 1996. Flexible buried bridges, also known as buried structures, comprise of a corrugated metal structure surrounded by engineered backfill. Buried bridges are used for various applications including, transportation, mining, tunneling and forestry.

Over the last three decades, buried bridges have been increasingly used for large span applications, providing at times a more cost-effective alternative to conventional bridges. Their maximum spans have increased from 8 m to more

than 20. The increased demand on these large span structures demonstrates the need to better understand their performance under various loading conditions, and to develop appropriate design methodologies to be used in their design.

Depending on the corrugation profile, corrugated metal structures are classified as shallow, deep corrugated, or deeper corrugated. The deepest corrugation profile was developed in 2011. The profile has a pitch of 500 mm and rise of 237 mm (Williams et al. 2012). The first structure built with this profile was in 2011 for a highway underpass in Eastern Canada with a 13.3 m span and a 5.3 m rise (Vallee et al., 2014 and Vallee, 2015).

Vallee (2015) instrumented the 500x237mm corrugation structure with strain gauges and deflection prisms. Vallee (2015) found that the code simplified method can be very conservative for long span single radius arches and conservative for short span arches with relatively stiff plates.

In 2016, the largest flexible buried structure in the world has been built for a transportation application in Ostróda Town in Poland (Europe). Utilizing the 500x237mm corrugation, the structure have a span of 25.5 m and a rise of 9.0 m.

A full-scale field test of part of this structure was conducted at Atlantic Industries Limited (AIL) facility in Dorchester to study its performance during construction. The structure was instrumented with strain gauges and deflection prisms and readings were taken during backfilling. Mentioned above structure is during construction now.

2. TEST STRUCTURE DETAILS

2.1. Structure geometry

The structure is a custom dual radius arch with inside span of 25.504 m and rise of 8.997 m. The structure was manufactured in Dorchester, NB (Canada). The geometry was custom designed to fit the project requirements.
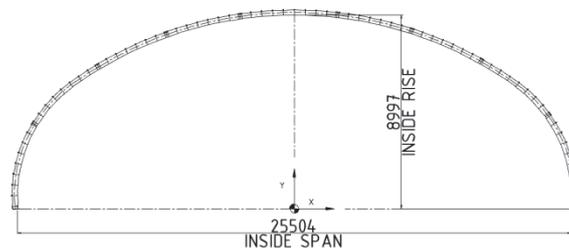

Figure 1. Structure geometry (Cross section)

In the longitudinal profile structure was beveled on inlet and outlet 1:1.5. The BCL lenght is 92.08 m
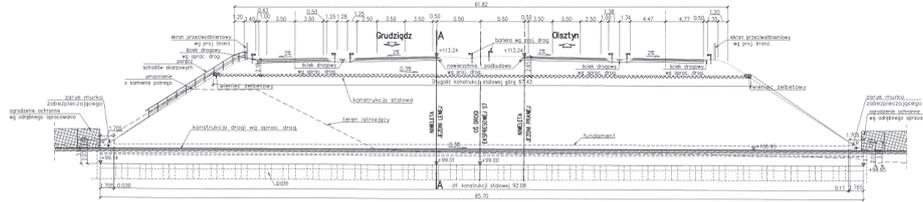
Figure 2. Structure geometry (longitudinal profile)

2.2. Structural plate corrugated steel

As mentioned above, the corrugation profile of this structure is known as 'Ultra-Cor' which is designated under CAN/CSA-G401-14 as 'Type III corrugation' and under CAN/CSA-S6-14 as 'deeper corrugation.' The corrugation profile is 500 mm x 237 mm, as can be seen in Figure 2. The straight portion is called tangent while the curved portions are called valley and crest. Thickness of the steel varies from 7 mm to 9.65 mm. The deeper corrugation offers more than three times the flexural rigidity compared to deep corrugation plates (380 mm x 140 mm) of the same thickness.

The test structure cross sectional properties as per ASTM A796 are shown in Table 1. The mechanical properties per the mill certification are shown in Table 2.

Table 1. Cross-section properties for 500 x 237 mm profile (ASTM A796/A796M-15A)

Specified thickness, [mm]	Area, [mm²/mm]	Moment of inertia [mm⁴/mm]
9.65	14.509	97 031.45

Table 2. Strength properties of test structure per mill certs.

Yield Stress (Fy), [MPa]	Ultimate Stress (Fu), [MPa]	Elongation
478	583	29%

2.3. Structure assembly

The structure had 92 rings with each ring consisting of nine plates. In the structure transverse direction, the plates are bolted together through bolted overlap longitudinal seems. In the longitudinal direction, the rings are bolted together through bolted overlap joints located every 500 mm along the periphery.

The structure was assembled as per industry standards. First odd rings were pre-assembled then lifted and secured into place. Remaining rings were installed plate by plate by bolting the plates to the fully erected adjacent rings as can be seen in Figure 3. Beveled ends were assembled at the very end.

Figure 3. Structure during assembly

2.4. Backfill

The backfill material conform contract requirements i.e. well graded gravel with sand (GW). The minimum uniformity coefficient, $C_u = 5$ and coefficient of gradation, $1<Cc<3$. The maximum modified proctor dry unit weight per ASTM D1557 was 2160 kg/m^3 at optimum moisture content of 7.8%. Backfill was placed and compacted with a maximum loose lift thickness of 300 mm. The backfill progressed evenly on both sides of the structure in an effort to balance lateral loads. The achieved compaction on-site with a vibratory plate ranged between 98% and 100% standard proctor density.

Figure 4. Structure during backfilling

3. FIELD TESTING

3.1. Instrumentation

The behaviour of the structure during backfilling process was to be closely monitored. The three-dimensional deflections of the steel structure were measured using a geodetic laser device. Fourty five measuring points were arranged in nine points in five sections of the arch. Location of the measurement points is shown in Figure 5. For each of the measuring points deflections were registered as changes between a fixed reference points on the ground and the optical prisms installed on the steel plates.

Strain gauges were placed at 17 locations inside the arch in cross section no. 2 (acc. Figure 5). Two gauges were fitted at each location: one at the crest of the corrugation and one in its valley. Additionally in 7 points i.e.: 3L, 3R, 4L, 4R, 8R, 8L, 9 an extra strain gauges were instaled in longitudinal direction (in total 48 starin gauges). The locations of all the strain gauges are shown in Figure 6. This configuration allowed to the axial and the bending strains be measured in longitudinal and latitudinal directions in specific points. Each strain gauge is protected against weather conditions as well as the signal cables are hidden in the protection pipes and fixed to the steel plates. Four dummy gauges were installed to provide temperature compensation. Four independent dummy gauges prevent the occurrence of errors caused by the uneven heating of the structure during backfilling process in sunny days.
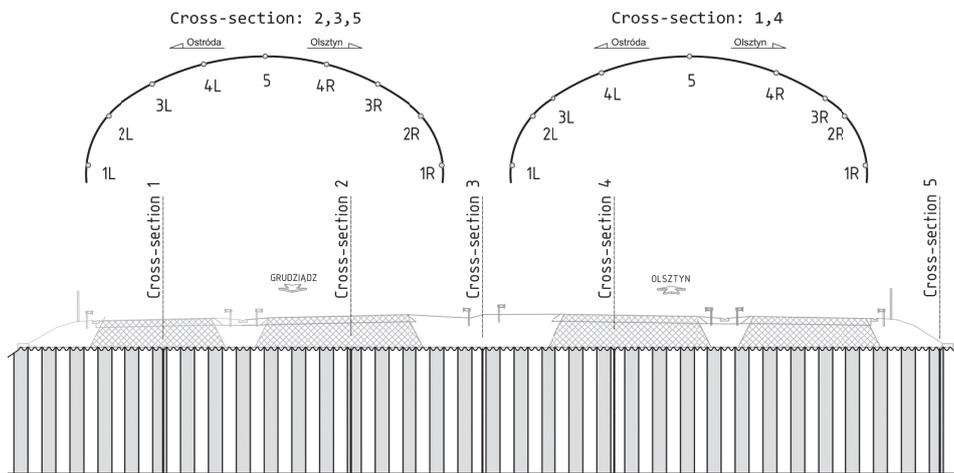

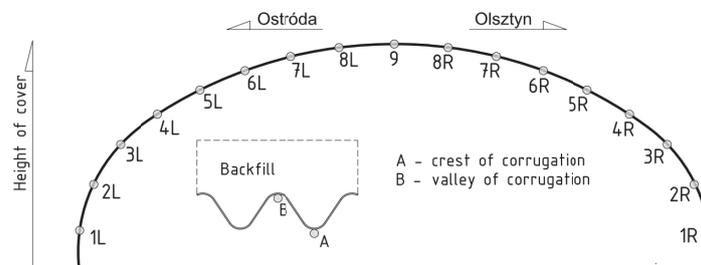Figure 5. Locations of the optical prism

Figure 6. Locations of the strain gauges

4. RESULTS

While submitting this paper backfilling process of the structure is in progress and therefore authors show only the preliminary results of the measurements for the level of backfill equal to 6.3 m over the top of the foundation.

4.1. Deflections

Deformation monitoring during construction is a regular practice in buried soil-metal structures. It serves an indication of structure performance. CAN/CSA-S6-14-S6-14 states that, upward deflection should not exceed 2% of the rise. According to the relevant code commentary (CAN/CSA-S6-14-S6-14-Commentary), this limit is based on empirical consideration rather than analysis. CAN/CSA-S6-14-S6-14 does not impose limits on lateral movements of these structures.

The vertical and horizontal deflections during backfilling proces were measured. Upwards and to the right side of the structure is positive movement. The crown area is moving upwards with a maximum of 206 mm, and the haunch area is moving downwards with a maximum of 65 mm.

The maximum upward movement is 2.3% of the rise for the cross-section no. 5 located close to the inlet of structure and 1,1 % of the rise for the remaining cross-sections no. 1, 2, 3, 4.

The crown area experienced minimal horizontal deflections of less than 20 mm while the haunch area had a 95 mm maximum horizontal deflection. The observed trend of inward movement at the haunch and upward movement at the crown is typical during backfilling of soil-metal buried structures.

The above observations indicate that the structure limitations on maximum upward deflection may need to be revised, especially for Type III corrugation structures. Numerical analysis may be required to permit a higher deflection limit and should be on a project to project basis.

Table 3. The vertical displacement of the crown for cross-sections from 1 to 5

Height of backfill [m]	Cross-section				
	1 [m]	2 [m]	3 [m]	4 [m]	5 [m]
0.0	0.000	0.000	0.000	0.000	0.000
1.3	0.002	0.002	0.004	0.004	0.005
2.1	0.007	0.008	0.008	0.009	0.012
2.1	0.004	0.004	0.006	0.006	0.010
3.3	0.016	0.018	0.021	0.020	0.030
4.1	0.023	0.027	0.030	0.029	0.048
5.0	0.046	0.052	0.056	0.056	0.097
5.7	0.068	0.076	0.081	0.080	0.150
6.3	0.085	0.094	0.100	0.097	0.206
(Canada test)					

4.2. Internal forces

The stresses in steel structure were calculated based on strains taking into cosideration that the elastic modulus of steel is E = 205 GPa and Poisson's ratio $\nu = 0.3$. The strains at measurement points allowed axial stresses and bending stresses to be calculated according to the following equations:

$$\sigma_a = \frac{\sigma_A + \sigma_B}{2}, \ \sigma_b = \frac{\sigma_A - \sigma_B}{2},$$

where:

σ_a – axial stresses,

σ_b – bending stresses,

σ_A – stresses at the crest of corrugation,

σ_B – stresses at the valley of corrugation.

The extreme stresses σ_{ext} (in the edge fibres of the shell cross section) are an algebraic sum of the stress σ_a produced by axial force and the stress σ_b produced by bending moment. The maximum stresses were recorded in point 3R for the height of backfill equal to 6,3 m over the top of the foundation. The stresses in point 3R amount to respectively: σ_{ext} = -225,1 MPa; σ_a = -116,0 MPa; σ_b = ±109,2 MPa. The stresses produced by the applied dead loads for height of backfill equal to 6,3 m are about 2.1 times lower than the yield point (in the measured cross-section of the arch). Distribution of bending and axial stresses is presented in Figure 8 for the height of backfill equal to 6,3 m over the top of the foundation.
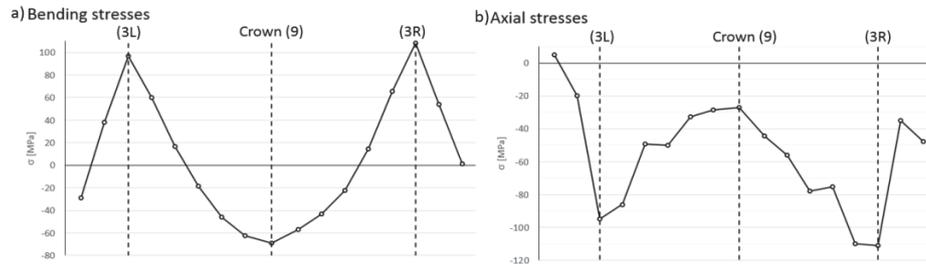
Figure 8. Distribution of a) bending stresses and b) axial stresses for the height
of backfill equal to 6,3 m over the top of the foundation.

5. CONCLUSIONS

The largest flexible buried bridge is still during construction now. A portion of the structure was built and backfilled up to the crown in Dorchester, Canada. The structure was instrumented with strain gauges and deflection prisms. Strain gauge data is analyzed and converted to bending moment and thrust forces. Results are compared with CAN/CSA-S6-14 simplified equations.

The results show that the maximum bending moments due to peaking were at the crown and the haunch, with the crown displaying slightly higher values. The bending maximum bending moment was 62% of that predicted by CAN/CSA-S6-14.

The thrust close to the arch tip was found to be in close match with CAN/CSA-S6-14 predictions. Gauges placed at the tangent are recommended to be away from the cross-section neutral axis.

The maximum vertical deflection during the test was 2.3% of the rise which exceed the limit of 2% defined by the code. It is recommended that the deflection limit in CAN/CSA-S6-14 be updated for such structures to 2.5% of the rise, provided field measurements and/or finite element analysis are used.

It is recommended that CAN/CSA-S6-14 equations for bending moment prediction to be further investigated for use in long span and dual radius structures. This initial project indicates that modifications are required. In the meantime, alternative methods such as finite element analysis may be viable.

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