INNOVATIVE LOW COVER BRIDGES UTILIZING DEEP-CORRUGATED STEEL PLATE WITH ENCASED CONCRETE COMPOSITE RIBS

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Abstract

Deep-corrugated steel plate structures have been in use throughout the world since 1988. While they provide economical alternatives for short to medium span bridges, highway and rail grade separations and hydraulic structures, the most abundant application is for box culverts. Until the use of deep corrugated plate, typical maximum spans for metal box culverts were around 8 m (25 ft.). Deep corrugated plate quickly extended this span range up to 12 m. The innovative application of encased-concrete, composite ribs, has further extended box culvert spans up to the 14 m range.

This paper describes the features and construction of a 14.1 m span, deep corrugated steel box culvert with encased concrete ribs which was installed over Price Creek on Highway 11 in Northern Ontario. This structure represents one of the longest box culvert structures in North America.

The manufactured, deep-corrugated plate and the components of the composite rib system are presented. The paper also describes some of the construction aspects of the bridge project including; the plate assembly, staged construction, pouring of the encased rib concrete and use of steel sheet piling headwalls on the ends of the structure. Detailed instrumentation and live load testing that was later undertaken on the in-service structure as part of an independent study by the owner is also described. The measured live load response of the structure is compared with the theoretical results from 2-D frame analyses and a finite element analysis. Comparisons in design capacity are also presented for current Ontario design vehicle live loading.

The paper highlights several advantages of this innovative bridge system for low cover, long span, stream crossings, including versatility, the strength benefits of deep-corrugated plate with encased concrete ribs, cost effectiveness, rapid installation time, environmental friendliness, aesthetics and durability. It is shown that these structures can be conservatively designed using a frame analysis or using finite element analysis, modeling the beneficial effects of soil-steel interaction as demonstrated in the full scale testing.

Introduction

Many bridge structures on today's road and highway systems have become functionally or structurally deficient and need to be replaced. Road realignment and new roadway construction coupled with this deterioration of existing structures means a growing list of new bridge projects for Provincial and State road and bridge authorities. To optimize tax dollars with limited operating budgets, authorities are looking for innovative, safe and durable bridge solutions that still meet the design and safety requirements of traditional bridges yet with lower initial capital costs and lower long term maintenance costs.

One innovative bridge solution for low rise stream crossings is the use of corrugated metal box culverts. With the advent of deep-corrugated steel structures in the late 1980's, typical maximum spans for this type of structure increased from around 8 m to over 12 m. The added innovation of utilizing encased concrete stiffening ribs has further increased the maximum attainable spans under typical highway loading up to approximately 14 m.

This paper describes the construction aspects of a 14.1 m span deep-corrugated box culvert which is typical of this innovative type of structure. Comparisons are made between simple and rigorous analysis of the structure and the actual as-built behaviour of the box culvert under high-load field testing.

Description - Price Creek Box Culvert

The box culvert was constructed in the fall of 2002 on Highway 11 over Price Creek, District 61, in Northern Ontario, about 50 km West of the junction of Highways 11 and 11B. The structure was fabricated by Atlantic Industries Limited (AIL), Dorchester, New Brunswick and sold to the Ministry of Transportation of Ontario (MTO), Thunder Bay District. It was built as a fast-track replacement for a two-cell timber box culvert that washed out during the tendering of the project. A 36.57 m span truss bridge was used as a temporary bridge during construction. A plan and cross-section of the box culvert are shown in Figures 1 and 2 respectively.

Corrugated metal box structures are characterized by a relatively flat crown, tightly curved haunches, or corners, and flat sides, yielding a long span and low rise "box" shape. The Price Creek box structure has an inside transverse span of 14.078 m, a rise of 3.07 m and a length of 26.632 m. It is oriented on a 15° skew to the highway and was constructed in two stages to facilitate traffic during construction. The completed roadway is two lanes, approximately 10.8 m wide over the structure. The cover over the crown of the box culvert varies from between 1300 mm at the centre to 850 mm at the ends.

The box culvert was fabricated from deep-corrugated plates referred to by AIL as Super-Cor. This corrugation profile has a pitch of 381 mm and a depth of 140 mm. The individual plates have a net width of 762 mm and all barrel and rib plates on this structure had a design thickness of 7.01 mm (1 gauge plate). The structure consisted of 35 main barrel rings with 18 encased concrete ribs spaced at 1143 mm centre to centre. The barrel and rib plates were assembled ring-by-ring in a shingling pattern, with five barrel plates in each transverse ring and 4 rib plates in each rib ring. The transverse "length" of each plate is denoted by the number of "S", where S = 406 mm. All plates were bolted together using 19 mm diameter galvanized bolts, 50 to 75 mm in length. A three-dimensional view of the assembled barrel plates showing the skewed cut plates for one end of the structure is given in Figure 3 and a photo of the structure under construction is shown in Figure 4.

The encased concrete ribs are comprised of 762 mm wide deep-corrugated rib plates bolted to the main barrel with the opposing crests in contact. Concrete shear connectors were welded in an alternating pattern to the barrel and rib plates during fabrication. After assembly of the plates in the field, 35 MPa flowable concrete was pumped into the cavity formed between the plates. The rib concrete was pumped through grout fittings shop welded to each rib near the crest of the box and was carefully monitored to ensure that the voids were filled completely. The total volume of concrete in the ribs was approximately 32 m^3. See Figure 5 for details of the encased concrete ribs.

The base of the box culvert was bolted to an unbalanced channel section which in turn was anchored to the top of a concrete pile cap. The concrete pile cap was cast over a pile foundation, designed to support the thrusts from the box structure. A 640 mm high by 1000 mm wide concrete collar was cast at each end of the assembled structure to aid in support of the skewed-end, cut plates. The concrete collar detail is shown in Figure 6. The end fills of the structure were supported by 4.18 mm thick galvanized steel sheet piling headwalls. The sheet piling walls are supported to the back of the collar around the box culvert and are anchored along the top using galvanized cable tiebacks. Beyond the structure, the sheet piling walls are embedded at the base and use cable tiebacks attached to dead-man anchors for lateral support. A temporary steel sheet piling headwall was utilized on the interior end of the first stage of the structure to retain the side fills of the road while the existing temporary structure was removed and the second stage of the box was assembled.

Each stage of the assembled box structure was backfilled using granular backfill material, compacted to 95% standard proctor density. The engineered backfill was extended a minimum of 1 m beyond the sides the structure and placed in 200 mm transverse lifts simultaneously on each side of the structure so as not to produce an unbalanced load condition. Standard base course and asphalt was used to produce the final road surface. The completed structure is shown in Figure 7.

Material Properties

Corrugated Steel

The steel plate material used in the box structure conformed to ASTM A1018/A1018M Grade 40. The average yield strength for the flat plate material, as taken from the mill certificates, was 310 MPa. For design and analysis purposes, a minimum yield strength of 300 MPa was used for the corrugated and curved material. The corrugated barrel and rib plates are 7.01 mm in thickness, hot dip galvanized to achieve a Z915 coating (> 64 micron zinc thickness per side) in accordance with CAN/CSA-G164. The galvanized bolts and nuts conform to ASTM A449 and A563 respectively. The shear connectors are manufactured in accordance with ASTM A108. All anchor bolts are hot dip galvanized and meet ASTM A307 Grade C specifications.

Composite Encased Concrete Ribs

Full scale testing in 1996 (1) concluded that the structure with encased concrete ribs acted in a predominantly composite manner. Further laboratory bending tests in 2004 on deep-corrugated, rib stiffened beams, confirmed fully effective composite properties for encased concrete sections. These tests also confirmed approximately a 20% enhancement in the composite steel bending capacity if the concrete contribution is considered. The composite section properties are given in Table 1 with and without the concrete included in the bending properties. For this structure type, where axial forces are small, the contribution of the concrete to axial stiffness is not important and was neglected.

Haunch Sections

The tight curvature of the haunch sections was accomplished during fabrication by utilizing cross-corrugations on the inside crests. These are visible in Figures 8 and

9. The reduction in bending capacity and stiffness of the cross-corrugated haunches is achieved by applying a haunch capacity reduction factor based on testing (2). This factor is listed in the footnote of Table 1 and the reduced values are given in parentheses in the table.

Backfill
The backfill was a well-graded, clean, Granular "A" material. The backfill soil models used are described under the analyses section. For gravity load a backfill unit weight of 21.21 kN/m^3 was assumed.

Box Culvert Live Load Test by MTO

In September 2003, the Ministry of Transportation of Ontario (MTO) instrumented the structure and conducted live load tests as part of their own research and development activities. MTO then undertook a plane-frame analysis, calibrated against the response from the live load tests. A brief summary of the MTO testing and results is given below. A full description of the tests and test results can be found in the MTO Load Test Report (3).

Loading
The full-scale testing was done using two MTO test trucks, each consisting of a three-axle tractor and two-axle trailer combination. The trucks were loaded incrementally with concrete blocks to maximum gross weights of 881.6 kN and 868.2 kN. One or both trucks were positioned at seven step locations along five different load lines on the roadway.

Instrumentation and Testing
The structure was instrumented with 42 strain gauges and 9 displacement transducers along 5 selected rings under the traffic loads. Deflection and strain measurements were taken with the test trucks in each step position for each load line.

Test Results
Some of the observations of the MTO testing were:

(i) the structure rings behaved in a linearly elastic fashion up to the maximum test loads, as verified by linear load-deflection and load-strain relationships,

(ii) strain distributions were linear in nature and therefore the structure is behaving in a composite manner and linear extrapolation could be used to estimate strains in the outside crests,

(iii) the maximum measured deflections were small, 7.7 mm vertical at the crown and 2.5 mm and 1.3 mm horizontal at the haunch and base respectively

(iv) maximum deflections typically occurred with two trucks near the centerline of the road (load line 5) and the two tandems centred over the crown (step 4) or both on one side of the crown (step 7),

(iii) the structure moved outward into the soil at the side for all load cases

(iv) the maximum tensile strain at the crown occurred under load line 5 at step 7 and was approximately 100 με,

(v) the maximum compressive strain at the crown, as extrapolated from the measured strains, was -97 με,

(vi) the maximum compressive strain occurred at the west haunch under load line 5 at step 4 and was -231 με ,

(vii) the maximum tensile strain at the haunch, as extrapolated from the measured strains, was 256 με,

(viii) with two truck load lines, deflections and strains were spread fairly uniformly across the road width,

Theoretical Analyses

Original Design and Analysis

The steel box structure was originally analyzed and designed in 2002 using the plane-frame program SODA (Structural Optimization Design and Analysis). The lateral loads due to the backfill were based on passive resistance Kp = 3, multiplied by an adjustment factor on each side, calibrated to measured deflections from previous field tests (1) and experience. Washed composite steel properties were used at the time of the original design.

The original design live load, distribution, load factors, dynamic load allowance and resistance factors were based on the provisions of the Ontario Highway Bridge Design Code (OHBDC) (4) in effect at the time. The design considered a cover range of 850 mm to 1300 mm with a tandem axle of 320 kN centred over the crown and a front axle of 120 kN applied 3.6 m from the tandem. For purposes of comparison in this investigation, the SODA analysis was redone using the MTO test vehicle arrangement, cover height and the current Canadian Highway Bridge

Design Code (CHBDC) (5) provisions, as used in the other analysis methods. Note that the load distributions are identical in both codes and only the loads and load factors differ. The analysis was also completed for the current CHBDC (CL-625-ONT) design live load.

Frame Analysis by MTO
The theoretical response of the structure under the test vehicle loading was calculated by MTO using the computer program S-FRAME. Details of this analysis can be found in the MTO Load Test Report (3). The unique feature of their work was the use of horizontal ground springs, calibrated using actual load test deflections, to model the lateral restraint provided by the soil.

2-D Finite Element Analysis
2-D modeling of the steel box was performed for the worst case loading conditions found from the test using the computer program CandeCAD Professional. CandeCAD is based on the public domain program CANDE-89 (6) which is a finite element program widely used to model buried structures incorporating nonlinear soil models and incremental construction. The typical CandeCAD mesh for the Price Creek box structure is shown in Figure 10. The box was modeled using beam-column elements (Table 1), pin connected to the footing elements. The backfill was modeled using the SW 95% Selig Modified soil model in the CandeCAD analysis. The representative native backfill on the sides was modeled as ML 95% and the foundation under the backfill as clay. The backfill, foundation and in-situ material properties used in the analysis are given in Table 2.

The analysis was done for the applied live loads based on the worst case test vehicle locations, load line 5, steps 4 and 7. Axle loads were distributed in the out-of plane, longitudinal direction of the box, using a distribution of 1 horizontal to 2 vertical from the outside of the axles down to the top of the box, in accordance with CHBDC. The distributed loads were then applied as 250 mm long footprint pressures to the appropriate top elements of the mesh for each load case. The analysis was done with the concrete rib properties ignored, then repeated with the concrete properties included. Sample outputs from CandeCAD showing the soil shear stresses and bending moments in the structure due to combined dead and live load are shown in Figures 11 and 12.

The finite element analysis was also done using the CHBDC design live load, located to produce the largest crown and haunch bending moments.

Results of Tests and Analyses

A comparison of actual and predicted maximum live load deflections at the crown and haunches is given in Table 3. The S-FRAME predictions by MTO use a spring stiffness K=30 $kN/m^2/m$, which was determined to give relatively close agreement with measured deflections. Values given in parentheses include the concrete properties in the analysis. Maximum measured lateral deflections near the base of the structure were small, in the order of 1 to 2 mm.

The actual and predicted live load bending moments at the haunches and crown are given in Table 4. These are unfactored values based on the fully loaded test vehicles in the positions that produced the largest strains. The CandeCAD results in parentheses include the concrete properties. The SODA and S-FRAME moments consider composite steel properties only.

Axial thrusts were not easily extrapolated from the strain readings in the test. Based on the symmetrical nature of the strains, it was assumed that axial effects were small and the measured strain values were resolved to bending moments using the elastic properties of the composite section. The theoretical live load axial compression forces from CandeCAD and SODA are listed in Table 5 for the locations of corresponding maximum moments.

Table 6 shows a comparison of the factored moments from SODA and CandeCAD for a CL-625-ONT design vehicle with a live load factor of 1.7, dynamic load allowance of 0.15 and resistance factors in accordance with CHBDC. Table 7 shows a comparison of the theoretical live load capacity factors for the crown and haunch sections of the box for the various analysis methods. The MTO factor is taken directly from their report and is based on observed versus theoretical strains, correlated to the CHBDC truck. The SODA and CandeCAD values are similarly calculated but using the calculated moments as given in Table 6, correlated to the measured versus theoretical moments from the test trucks. The available moment capacity is also adjusted by accounting for the theoretical thrust component using the plastic interaction equation per CHBDC Clause 7.6.2.3.

General Discussion

The box structure was manufactured, shipped to the site and assembled in a compressed schedule as an emergency washout bridge replacement. Assembly was done using lightweight equipment and a small assembly crew. The structure assembly and design was able to accommodate two-stage construction using a temporary sheet piling retaining wall system. The encased concrete ribs were easily poured in one day for each of the stages. Since the ribs are formed by the corrugated plates themselves, no additional formwork was required. Insulation of the ribs and temporary heating during curing was also easily accommodated. The span of the structure allowed the construction activities to remain well outside of the stream. Due to the stiffness of the deep-corrugated plate, no additional formwork or bracing of the box culvert was required. The concrete collars and sheet piling headwalls provide an economical and aesthetically pleasing end treatment. The structure demonstrated that by using cast-in-place concrete collars, skewed ends can be accommodated by this corrugated plate system.

Deep-corrugated structural plate has approximately nine times the stiffness of shallow corrugated (152 x 51 mm corrugation profile) plate, or approximately three times the bending capacity, for a given thickness of steel. The composite steel properties provided by intermittent deep-corrugated ribs, with encased concrete, provide about five times the stiffness and 2.7 times the plastic moment capacity of the unreinforced, deep-corrugated section. Including the concrete properties adds an additional 20 percent to the stiffness and moment capacity of the system.

Excellent performance of the box culvert was verified in the field by the independent live load tests. With a total live load of about 1750 kN on the structure, it displayed a vertical deflection of less than 8 mm at the crown and horizontal deflections of less than 3 mm at the sides. From the extrapolated strains, the maximum compressive and tensile steel stresses in the haunches due to the full test load are approximately 50 MPa. The maximum live load stress in the steel at the crown is around 20 MPa.

The live load deflections are overestimated by all of the analyses methods. A 3-D analysis would likely yield a more accurate prediction of the longitudinal load spreading and yield closer defections to those measured. Alternatively, additional longitudinal load spreading, or an appropriate reduction in the live load, could be used for deflection determination. It has been suggested in other recent research of buried structures analyzed using finite element techniques, that a reduction of up to $1/10^{th}$ of the load could be employed for estimating deflections (7).

The theoretical responses from the frame analyses were very conservative compared to the measured values. The 2-D finite element analysis provided overall closer correlation to measured values, although crown moments were still overestimated. Sensitivity to several parameters in the 2-D model was also investigated. These included the backfill soil model, in-situ and foundation soil models, interface element tensile breaking force and interface friction coefficient. None of these had a significant effect on the results. The results were most sensitive to the longitudinal distribution of the live load and the cover height. Study of the strain and deflection readings at the various instrumented rings in the test displayed substantial spreading of the load in the longitudinal structure direction. Further study is needed to quantify this for use in live load distribution for a 2-D analysis. The skew effect of the structure may have also influenced the test results and additional 3-D analysis may be useful to study this more closely.

The outward movement of the sides of the box under load confirmed the importance of soil resistance on the sides, even for such a low rise structure. While an adjusted passive resistance approach or horizontal ground springs can be used in a frame analyses, the results are sensitive to the magnitude of the side support. Further, these can not be accurately predicted without actual field measured deflections under load. The non-linear soil models in the finite element analysis provide a more rational design approach.

The analyses confirm that the live load thrusts in the structure, of around 100 to 200 kN/m, are relatively small compared to the axial capacity of the section of almost 5000 kN/m. In terms of axial and bending interaction, the bending component is the dominant effect and ignoring thrust in the test strains is a reasonable assumption. Strain gauges on the extreme fibres and neutral axis of the composite section are required to accurately determine the axial effects from load tests.

Including the concrete properties in the CandeCAD analysis had little effect on the deflections and maximum moments. Including the concrete in the bending resistance increases the theoretical live load capacity factors substantially. The incorporation of the axial capacity of the encased concrete ribs would be more advantageous in an arch type structure where thrust is the dominant force effect.

Conclusions

1. This type of structure can be constructed economically, quickly, in remote locations, using light equipment and in stages if required. These structures are environmentally friendly as the longer spans minimize disturbance to the stream, and they require little or no maintenance over the design life.

2. Deep-corrugated steel plate box culverts with encased concrete composite ribs can safely carry highway loading at spans up to 14 m.

3. This structure type can be conservatively designed using the CHBDC live loading, load factors and distribution provisions. Design bending moments and axial forces can be determined using either a calibrated steel frame computer program or a more rigorous finite element model. Ignoring the concrete contribution in the section properties is conservative.

4. A 2-D finite element analysis provides a more rational and realistic model of the true soil-structure behaviour compared to a frame analysis.

5. Deep-corrugated box structures can be conveniently instrumented and load tested in the field. Instrumentation of a new box culvert from the beginning, including gauges on the soil side of the structure, would provide more useful information on the backfilling strains and live loading response.

6. Further study of longitudinal distribution is required to more accurately predict the live load response using 2-D analysis methods and to provide more economical designs.

7. A parametric study of long-span deep-corrugated box culverts with varying geometries, covers, live load intensity and live load positions could likely be used to develop simplified equations for design.

Acknowledgements

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Disclaimer

Results from the testing and analysis by MTO have been included in this paper to compare with the analytical results by AIL. Photos of the instrumentation were taken from MTO’s report, with permission. All views and conclusions in this report are those of the author and may not necessarily be shared by MTO.

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TABLES

	I	A	Z	P_y	M_y	M_p
	(mm^4/mm)	(mm^2/mm)	(mm^3/mm)	(kN/m)	(kN-m/m)	(kN-m/m)
Composite Properties Steel Only	122638 (103592)	16.36	1187.7 (1003.2)	4908	222.6 (188.0)	356.3 (301.0)
Composite Properties w Concrete	147445 (124546)	16.36	1419.3 (1198.9)	4908	222.6 (188.0)	425.8 (359.7)

Description of Symbols:

I moment of inertia
A cross-sectional area
Z plastic section modulus
P_y axial force in steel only (not including concrete effects) to cause yield in the absence of moment, $A \times F_y$
M_y moment to cause yield in the absence of axial load, $S \times F_y$
M_p fully plastic moment, $Z \times F_y$

Note additional material properties:

E_s Young's modulus of steel = 200,000 MPa
F_y yield strength of steel = 300 MPa
S Elastic section modulus of composite steel only section = 742.2 mm^3/mm
f'_c 28-day compressive strength of rib reinforcement concrete = 35 MPa
A_c Concrete area per 762 mm rib = 104625 mm^2
E_c Young's modulus of concrete = 26,622 MPa
HRF Reduction factor for haunch capacity and stiffness = 0.8447
Values in parentheses are reduced values for the cross-corrugated haunch sections.

Table 1: Composite Steel Member Properties

	Backfill SW-95% Selig Mod	Side Fill ML-95% Selig Mod	Backfill Foundation	Pile Foundation	EPS Foam in Fill
Unit Weight (kN/m^3)	21.2	21.2	0	0	0
Cohesion (kPa)	0	27.5	--	--	--
Initial Friction Angle (deg)	48	34	--	--	--
Reduction in Friction Angle	8	0	--	--	--
Modulus Number	950	440	--	--	--
Modulus Exponent	0.60	0.40	--	--	--
Failure Ratio	0.70	0.95	--	--	--
Bulk Modulus	*187.0	*120.8	--	--	--
Ultimate Volumetric Strain	0.014	0.043	--	--	--
Poisson's Ratio	0	0	0.17	0.17	0.30
Young's Modulus (MPa)	--	--	21000	21000	29.43

* Normalized initial tangent bulk modulus

Table 2: Soil Properties used for CandeCAD Analyses

Location	Load Case	*Actual Measured (mm)	*S-FRAME K=30kN/m^2/m (mm)	SODA (mm)	CANDE (mm)
West Haunch (horizontal into soil)	Line 5 Step 4	2.0	3.6 (3.5)	7.2	12.5 (12.3)
Crown Vicinity (vertical downwards)	Line 5 Step 7	7.7	17.1 (15.3)	28.3	38.5 (38.1)
East Haunch (horizontal into soil)	Line 5 Step 4	2.5	3.3 (3.2)	7.0	12.6 (12.8)

* From MTO Report (3)

Table 3: Comparison of Maximum Live Load (Test) Deflections (mm)

Location	Load Case	*Actual Measured kN-m/m	*S-FRAME K=30 kN/m^2/m kN-m/m	SODA kN-m/m	CANDE kN-m/m
West Haunch	Line 5 Step 4	-37.4	-81.3	-114.0	-85.0 (-86.8)
Crown	Line 5 Step 7	19.1	98.1	85.6	71.0 (71.0)
East Haunch	Line 5 Step 4	-27.2	-107.4	-113.6	-83.4 (-85.0)

* Converted from strain readings in MTO Report (3)

Table 4: Comparison of Maximum Live Load (Test) Moments (kN-m/m)

Location	Load Case	SODA kN/m	CANDE kN/m
West Haunch	Line 5 Step 4	133.6	217.9
Crown	Line 5 Step 7	62.1	95.9
East Haunch	Line 5 Step 4	134.1	218.4

Table 5: Comparison of Axial Loads (Test Live Loads) (kN/m)

Location	SODA DL Moment	SODA LL Moment	CANDE DL Moment	CANDE LL Moment
West Haunch	188.0	85.4	202.7	99.5
Crown	84.3	85.4	104.1	102.7
East Haunch	188.4	79.7	203.3	97.2

Table 6: Comparison of Factored Live and Dead Load Moments for CL-625-ONT Design Truck

Location	*S-FRAME	SODA	CANDE
West Haunch	1.06	2.3	1.1 **(2.6)
Crown	1.47	12.2	7.6 **(9.9)
East Haunch	2.20	2.4	1.4 **(3.5)

* From MTO Report (3)
** Values in parentheses include the concrete in the moment capacities

Table 7: Minimum CL-625-ONT Live Load Capacity Factors from Analyses

FIGURES

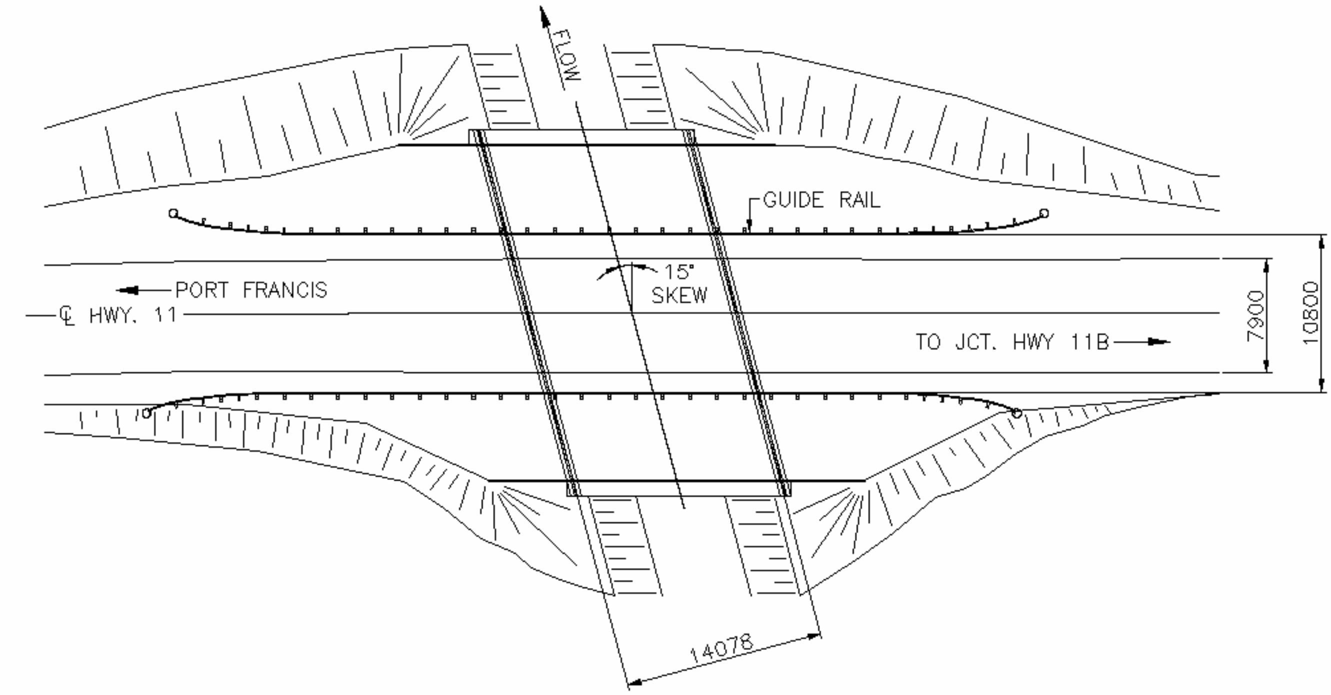

Figure 1: Price Creek Box Culvert – Plan

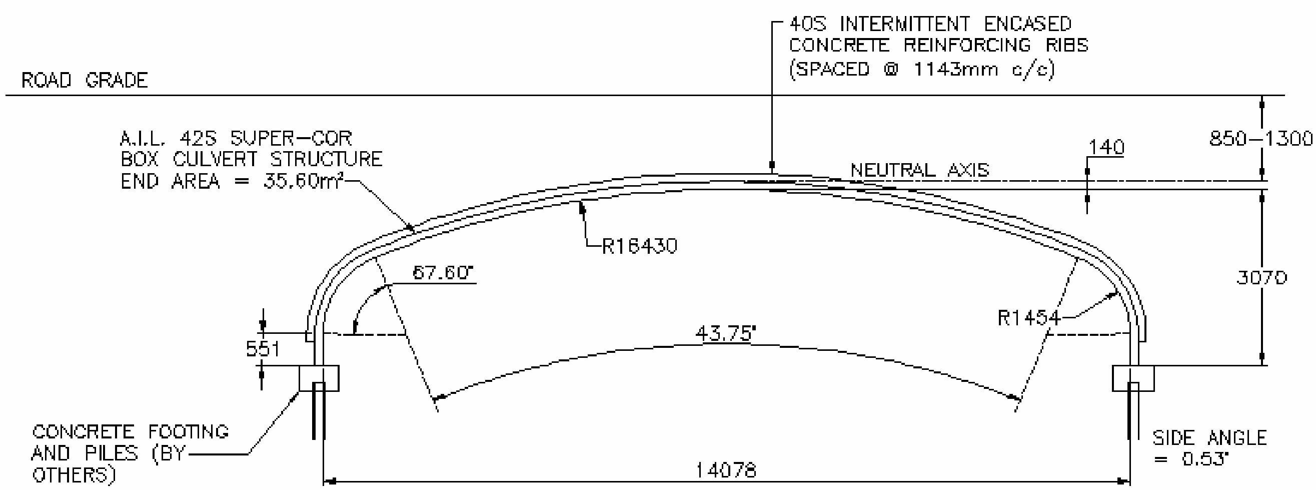

Figure 2: Price Creek Box Culvert – Cross-Section

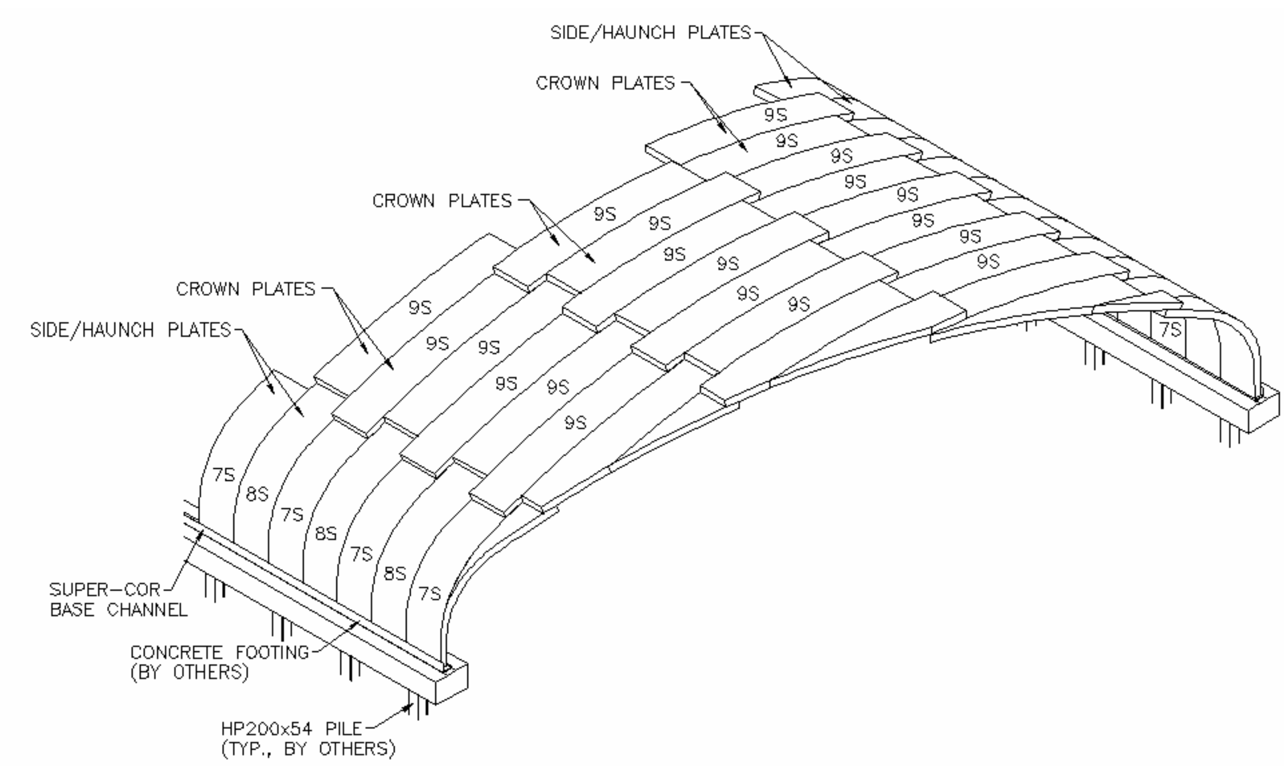

Figure 3: Assembled Barrel Plates and Skewed End Plates

Figure 4: Structure During Assembly

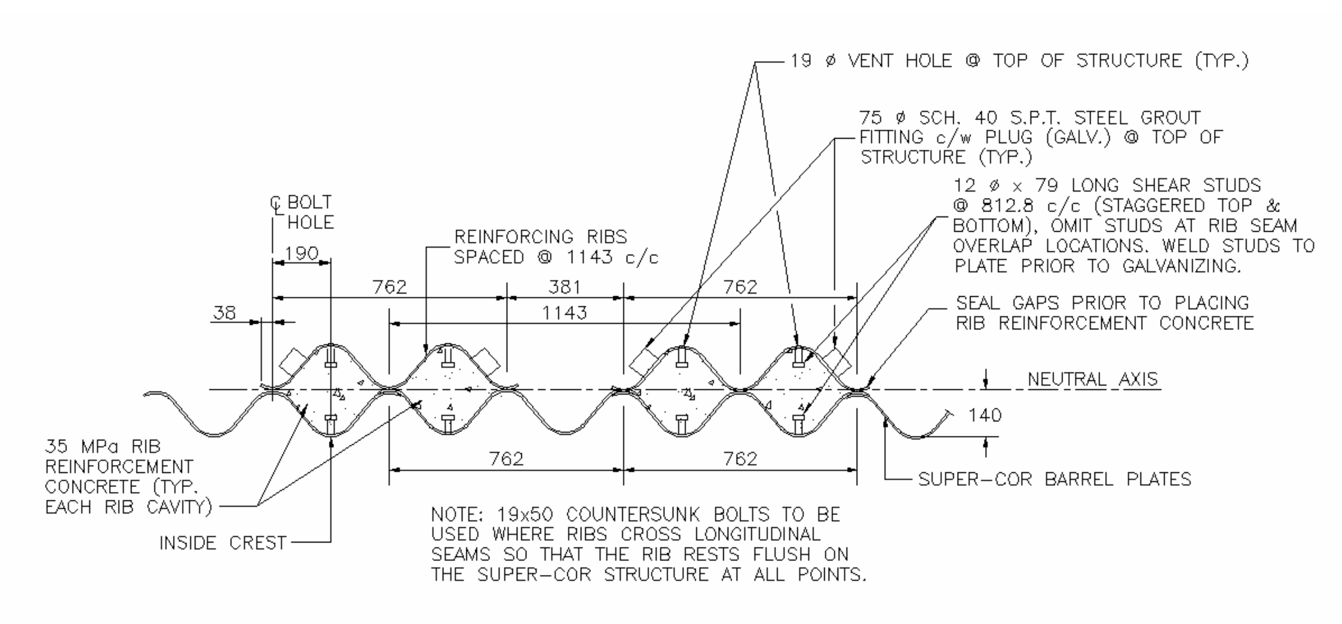

Figure 5: Detail of Encased Concrete Composite Ribs

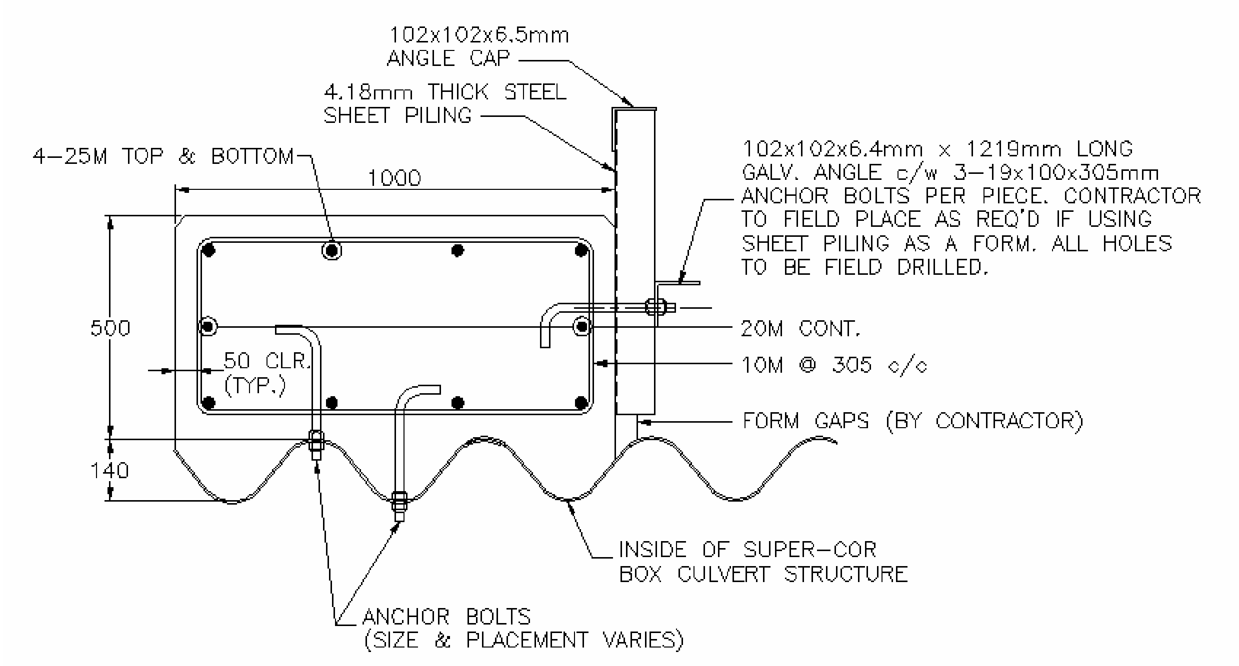

Figure 6: Detail of End Collar

Figure 7: Completed Structure

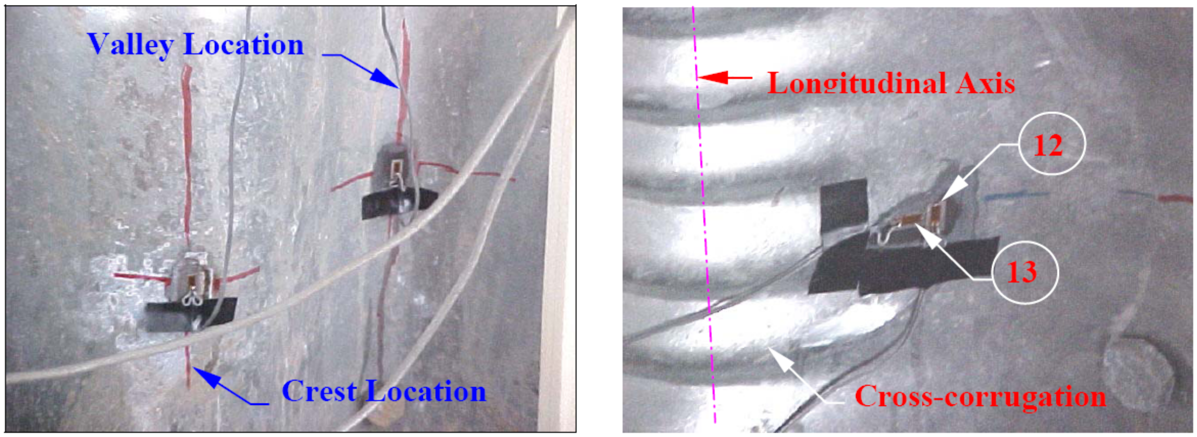

Figure 8: Gauges at Crest and Valley (shown on left) and in Cross-Corrugated Haunch Locations (shown on right) (from MTO Report (3), Figs. 19 and 20)

Figure 9: Horizontal Displacement Transducers at the Haunch and Base (from MTO Report (3), Figs. 21(a) and 21(b)

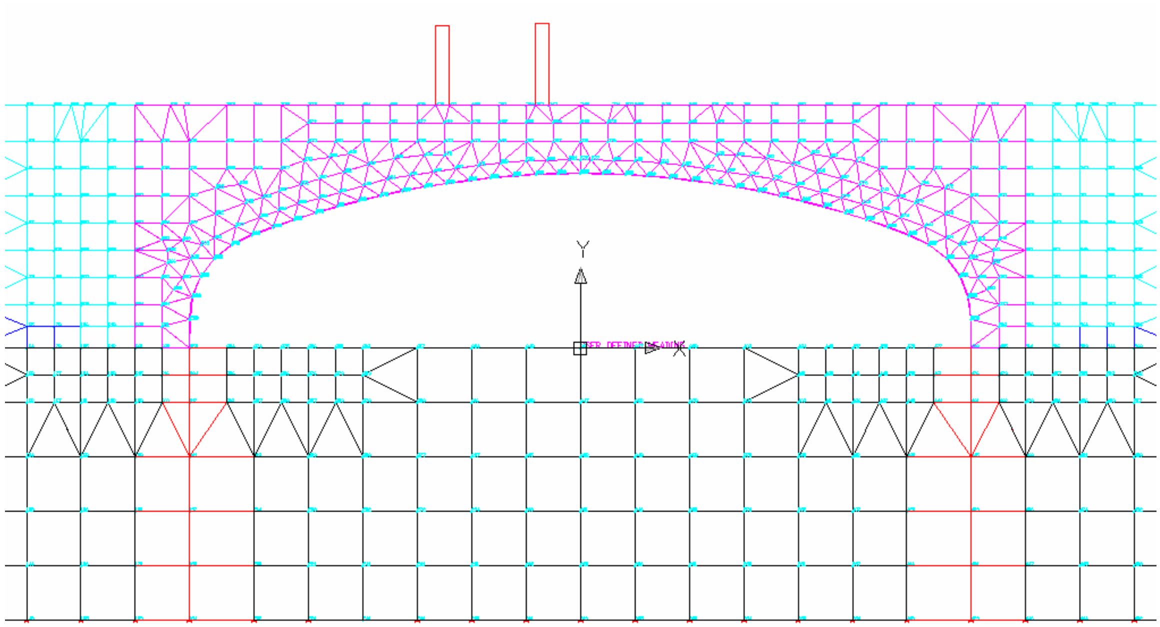

Figure 10: CandeCAD 2-D Finite Element Mesh used for the Analysis (see Table 2 for soil and foundation properties, tandem axle position shown for step 7)

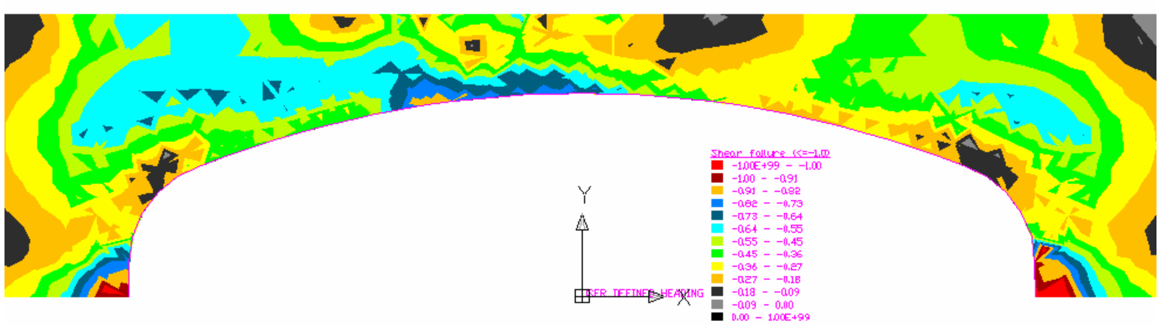

Figure 11: Soil Shear Stresses (Dead + Live Loads) From CandeCAD for Load Line 5, Step 7

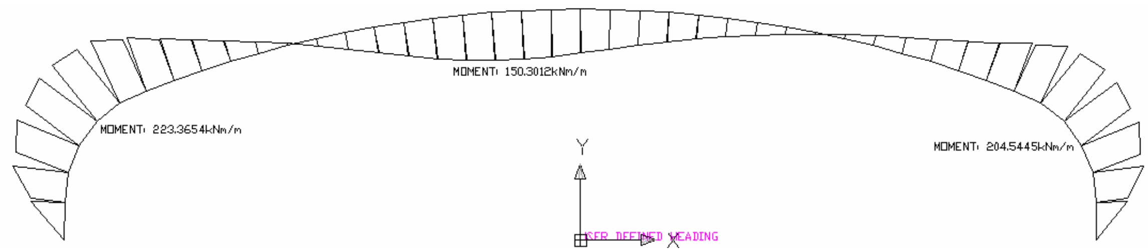

Figure 12: Bending Moments (Dead + Live Loads) From CandeCAD for Load Line 5, Step 7