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I, Nevil Gomes, hereby submit this original work as part of the requirements for the degree of Master of Science in Civil Engineering.

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Experimental and Analytical Evaluation of an Innovative Strengthening System for Long-Span Deep-Corrugated Buried Bridges

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Experimental and Analytical Evaluation of an Innovative Strengthening System for Long-Span
Deep-Corrugated Buried Bridges

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ABSTRACT

Experimental and analytical study of an innovative strengthening system for deep corrugated structural plates (15-in. pitch by 5.5-in.) is described. A typical 5/16-in.-thick deep corrugated structural plate can typically span up to 65-ft for typical bridge design loads. In order to achieve longer spans, it is necessary to increase the flexural stiffness of the plate. The new strengthening system consists of a wide flange beam connected to the corrugated structural plate using a rigid bracket. Several tests were conducted at the University of Cincinnati Large Scale Testing Facility to evaluate the behavior and stiffness of the beam-plate system. Tests were carried out using a number of reinforcing beams and different bracket spacing. Equations for determining the cross-sectional properties and capacity of the stiffened system were developed. Design case studies were conducted using a special finite element program called CANDE (**C**ulvert **A**nalysis and **D**esign). Different plate gauges in combination with a number of wide flange beams were used in the finite element analyses. The efficiency of the strengthened system was evaluated. In this paper, recommendations are made for design and analysis of strengthened long-span buried bridges. This study has found that it is possible to design lighter, longer, and cost effective structures through the use of stiffened plates.

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TABLE OF CONTENTS

ABSTRACT.....	i
ACKNOWLEDGEMENTS.....	iii
TABLE OF CONTENTS.....	iv
LIST OF TABLES.....	v
LIST OF FIGURES	vi
INTRODUCTION	1
LABORATORY TESTING.....	3
RECOMMENDATIONS FOR DESIGN CALCULATIONS	7
ANALYTICAL PARAMETRIC STUDY	8
SUMMARY AND CONCLUSIONS	15
REFERENCES	29
APPENDIX A Test Setup and Instrumentation.....	30
APPENDIX B Laboratory Test Results.....	35
APPENDIX C CANDE Modelling.....	41

LIST OF TABLES

TABLE 1 Test Matrix 16

TABLE 2 Test Results: Moment of Inertia..... 17

TABLE 3 Test Results: Bending Moment 18

LIST OF FIGURES

FIGURE 1 Previous Strengthening Methods (3) – Dimensions in mm (a) Continuous Corrugated Stiffener (b) Concrete Encased Corrugated Stiffener	19
FIGURE 2 Strengthening System	20
FIGURE 3 Schematic Diagram of a Strengthened Buried Bridge (backfill not shown for clarity).....	21
FIGURE 4 Details of Test Specimens	22
FIGURE 5 Test Setup	23
FIGURE 6 Benchmark Structure (courtesy of Contech Engineering Solutions).....	24
FIGURE 7 (a) CANDE Finite Element Mesh (b) Backfill Mesh with Construction Steps	25
FIGURE 8 Cross Section of Structural Steel Pipe	26
FIGURE 9 Comparison of Span and Weight.....	27
FIGURE 10 Effects of Stiffener spacing on (a) 3/8-gage plate (b) 5/16-gage plate (c) 1-gage plate (d) 7-gage plate	28

INTRODUCTION

Corrugated plates have sufficient capacity to carry the axial thrust generated in culverts. The flexural resistance of the plates is low because of thin plate elements and residual stresses caused by large deformations that are needed to form corrugations. In long-span structures, the moments and deflections of the crown during the construction, and backfilling operations are important design criteria that must be checked as part of design. It is a common practice to provide additional stiffness to the crown of the structure by attaching reinforcing elements. Stiffeners were originally used to only control upward deflection of the culvert and maintain the design geometry of the culvert during construction. Later, the structural advantages of stiffeners were recognized and several methods were developed to increase crown stiffness and improve the overall structural performance (1, 2).

One of the most common reinforcing methods involves bolting a corrugated plate, having the same pitch as the main plate, to the crown (3), as shown in Figure 1a. The plates do not perform as a composite section because of slippage at the bolted connections. The performance can be improved by using concrete encased corrugated stiffener plate (4). For this purpose, concrete is pumped into the cavity formed by placing one corrugated plate on top of the other. Shear studs are welded inside the cavity before installation to provide increased shear transfer with the concrete. The resulting system (see Figure 1b) is efficient and acts as a composite section; however, the construction requires a significant amount of fabrication and field pumping of concrete; the associated costs might make this system uneconomical. Previous studies (3) have shown that it is more practical to use corrugated stiffeners that act cumulatively than utilizing encased composite stiffeners.

Rolled steel sections can also be used to stiffen the corrugated structural plates (2). Wide flange, tee beams, or angle sections have typically been used for this purpose. Steel sections have a larger shape factor and prove to be a better alternative compared to the corrugated plate stiffeners. Rolled beams are curved to match the radius of the structural plate and are bolted to the plate at regular intervals. The combined stiffness of the structure can be controlled by varying the spacing and size of the structural sections. This method works well for smaller corrugation sizes of 6-in. pitch by 2-in. rise.

Deep corrugated structural plates have corrugations greater than 15-in. pitch by 5.5-in. depth. Since the pitch of the corrugation is greater than 15-in., it is not possible to attach the strengthening beam to the plate unless it is a beam heavier than W36×231, for which the flange width is greater than 15-in. Using such a heavy section is not practical as it is not cost effective; moreover, a heavy beam would result in a highly conservative design. As shown in Figure 2, this problem was solved by using a bracket bolted onto two adjacent crests of the corrugated plate. In other words, the bracket “bridges” the gap between the crests so lighter strengthening beams with flange widths smaller than 15-in. can be used. The bracket is sufficiently stiff so that it provides adequate shear transfer between the beam and the plate. A curved reinforcing member such as a wide flange structural beam is bolted to each of the brackets (see Figure 3). Installation of this strengthening system does not require any special formwork and it can be assembled directly on site. This system uses fewer bolts as compared to corrugated plate stiffener while providing sufficient stiffness and stability to the main structure. The behavior and potential benefits of this strengthening system were evaluated experimentally and analytically.

LABORATORY TESTING

To evaluate the performance of the proposed strengthening system, seven deep corrugated plates reinforced with four different wide flange beams were fabricated and tested (see Table 1).

Additionally, two plates alone were tested to determine the basic properties of the plate.

Test Setup and Loading

Each plate was 3/8-in.-thick, 12-ft long and, 3-ft 11-in. wide. The plates were strengthened using four different wide flange structural shapes to identify the most economical size. The beams were connected to the plate using four brackets spaced evenly at 48-in. or 32-in. on center.

Varying the bracket spacing provided data to investigate the effect of bracket spacing on the stiffness of the plate. Lateral “tension straps” (Figure 4) were added because of concerns that the plate would tend to expand laterally under loading. It should be noted that in field applications lateral expansion is not expected due to adjoining corrugated plates.

The tests were conducted in two phases. In phase I, seven specimens were tested to their ultimate capacity. The bottom flange of the strengthening beam was bolted to the bracket (Figure 4a). The failure modes were classified as (1) local compression flange buckling, (2) lateral torsional buckling, and (3) flange lateral bending. To provide lateral bracing to the top flange of the section, the two bolts connecting the bracket to the beam were extended to the top flange of the beam. Two specimens with this detail were tested in phase II (Figure 4b). In Table 1, the specimens are listed in the order that they were tested.

As shown in Figure 5, the test specimens were tested in a four-point bending arrangement, with a 4-ft spacing between the applied loads. The specimens had a 12-ft simple span between two rocker-supports, and the shear span was 4-ft in all the specimens. The

specimens were tested in position control, i.e., each specimen was pushed through a series of displacement increments until the beam experienced excessive deformation, or the beam had excessive local or global buckling.

Stiffness

The measured load-deflection relationship was plotted and the linear portion of the curve was used to calculate the flexural rigidity of the strengthened system. The effective moment of inertia of the beam was calculated using Eq. 1.

$$I = \frac{Pa(3La-4a^2)}{6E\Delta} \quad \text{Eq. 1}$$

In Eq. 1, P/Δ is the slope of the linear portion of the plot, L = the span (144 in.), a = the shear span (48 in.), and E = the modulus of elasticity of steel taken as 29,000 ksi.

The results suggest no composite action between the beam and the plate. Had the system performed compositely, the calculated moments of inertia would have been nearly two to three times larger than the measured values (see Table 2). The lack of composite action can be attributed to the slippage between the brackets and the plate, and the brackets and the beam at the bolted connection. In other words, the system behaved in a cumulative manner and not compositely. This observation is consistent with the previous tests conducted on reinforced buried bridges (3, 5). As evident from Table 2, a very good correlation of the measured moment of inertia was possible by adding the moments of inertia of the plate and beam.

Flexural Capacity

Considering the lack of composite action between the beam and the corrugated plate, the applied load is distributed to the beam and plate proportional to their stiffness, as shown by Eq. 2 and Eq. 3.

$$M_{beam} = \left(\frac{I_{beam}}{I_{beam} + I_{plate}} \right) M_{applied} \quad \text{Eq. 2}$$

$$M_{plate} = \left(\frac{I_{plate}}{I_{beam} + I_{plate}} \right) M_{applied} \quad \text{Eq. 3}$$

Where $M_{applied}$ = applied moment, M_{beam} = portion of applied moment resisted by stiffening beam, M_{plate} = portion of applied moment resisted by plate.

Based on the observations made above and basic mechanics principles, the expression shown in Eq. 4 was derived to compute the nominal moment carrying capacity of the strengthened system.

$$M_{n,system} = \left(\frac{I_{beam} + I_{plate}}{I_{beam}} \right) M_{n,beam,AISC} \quad \text{Eq. 4}$$

$M_{n,beam,AISC}$ is the nominal capacity of the beam calculated based on *AISC Specifications* and is available in *AISC Steel Construction Manual (6)*. Using Eq. 4, the nominal capacity of the strengthened specimens was computed and compared against the maximum measured moment (Table 3). On average the measured moment was 1.15 times the calculated capacity with a coefficient of variation of 0.125. Therefore, Eq. 4 can be used to obtain a reasonable, yet conservative, estimate of flexural strength of corrugated structural plates strengthened by a structural steel beam. For design purposes, the nominal moment capacity determined from Eq. 4 is to be multiplied by strength reduction factor $\phi = 0.90$.

Beam Spacing

In order to achieve an efficient design, the beams must be laterally spaced at a distance that gives the required stiffness to the plate. As discussed above and evident from Table 2, the beam and plate do not act compositely but rather they act cumulatively. Therefore, if s is the lateral spacing of the beams and I_{req} is the required stiffness of the structure per unit width, Eq. 5 can be derived.

$$I_{req} = \frac{I_{beam}}{s} + I_{plate} \quad \text{Eq. 5}$$

Where $\frac{I_{beam}}{s}$ is the moment of inertial of the beam per unit width and I_{plate} is the moment of inertia of the plate per unit width. By rearranging the terms, the required spacing of the beams can be determined from Eq. 6.

$$s = \frac{I_{beam}}{I_{req} - I_{plate}} \quad \text{Eq. 6}$$

For a corrugated plate with a 15-in. pitch, the possible minimum spacing of the beams is 30-in.; hence, the spacing is 30 in., 45 in., 60 in., etc.

RECOMMENDATIONS FOR DESIGN CALCULATIONS

As part of design, a number of key parameters need to be specified. Based on the experimental data and aforementioned discussions, the following equations are proposed for determining the cross-sectional properties of the reinforced plate.

$$A_{total} = \frac{A_{beam}}{s} + A_{plate} \quad \text{Eq. 7}$$

$$I_{total} = \frac{I_{beam}}{s} + I_{plate} \quad \text{Eq. 8}$$

$$S_{total} = \frac{S_{beam}}{s} \left(\frac{F_{y, beam}}{F_{y, plate}} \right) + S_{plate} \quad \text{Eq. 9}$$

$$Z_{total} = \frac{Z_{beam}}{s} \left(\frac{F_{y, beam}}{F_{y, plate}} \right) + Z_{plate} \quad \text{Eq. 10}$$

A_{total} is in in.²/in., I_{total} is in in.⁴/in., S_{total} is in in.³/in., and Z_{total} is in in.³/in. Eq. 9 and Eq. 10 accounts for different yield strengths of the beam and plate ($F_{y, beam}$ and $F_{y, plate}$).

A trial and error method may be used to determine the required size of the stiffening beam. Instead, the following equations may be used to size the stiffening beam needed to resist the moment and axial load (M_u and P_u) computed from finite element analysis.

$$M_{beam} = (sM_u) \left(\frac{I_{beam}}{I_{beam} + sI_{plate}} \right) \quad \text{Eq. 11}$$

$$P_{beam} = (sP_u) \left(\frac{A_{beam}}{A_{beam} + sA_{plate}} \right) \quad \text{Eq. 12}$$

where M_{beam} and P_{beam} are the moment and axial load used to design the stiffening beam.

ANALYTICAL PARAMETRIC STUDY

The impacts of strengthening system were evaluated through a series of parametric studies. A special-purpose finite element program called CANDE (Culvert **A**Nalysis and **D**Esign (7)) was used for this purpose. “CANDE’s finite-element methodology is based on a two-dimensional slice of the culvert installation so that both the structure and soil mass are modeled as a combined soil-structure system subjected to an incremental loading schedule. Buried bridges of any shape, size, and material, including corrugated metal, reinforced concrete and thermoplastic may be analyzed and designed to withstand dead weight, incremental soil-layer loading, temporary construction loads, and surface loads due to vehicular traffic.” (7–9)

CANDE solution level 3 was used for the reported analyses. This option allows a complete control over the modeling parameters, structural shape, soil system, and loading conditions. However, it is necessary to define the finite element mesh, assign material properties to different elements, and specify the loading sequence. A spreadsheet was developed to facilitate generation of culvert profiles and the models.

Two series of analyses were performed. In the first series, an existing bridge was redesigned using the proposed strengthening system. The use of thinner plates and increasing the span length were the primary objectives of redesign. The focus of the second series was on evaluating the influence of plate thickness and beam spacing.

Series 1

A 3-radius arch with a span of 67-ft 8-in. and a rise of 27-ft 5-in. was selected as the benchmark structure (Figure 6). In this bridge, the culvert is made of 5/16-gage (thickness = 0.318-in.) deep

corrugated plate and has 6-ft cover over the crown. Interstate traffic goes through the bridge with wildlife crossing on the top.

Geometry

One aspect of redesign was to increase the span through the use of the proposed strengthening system. Culverts with different combinations of span and rise were obtained by varying the radii of the crown, haunch, and the walls. Culvert shapes composed of different crown, haunch radii, and lengths of plates can have the same span and rise. Two culverts with identical structural steel weight can also have different spans. To allow comparison of different culverts, the shape of the culvert profile was not changed; the values of span and rise were proportionally increased to obtain different culverts while keeping a rise to span ratio 0.4.

Figure 6 shows the geometry of the 3-radius arch that was analyzed and redesigned using the new strengthening system. Crown segment of the arch has a radius R_1 of 43-ft 9-in. and central angle 74° . Haunch segment has a radius R_2 of 17-ft 9-in. and central angle 51° on each side of the crown. The sidewalls are made of a circular segment with a radius of R_3 24-ft 9-in. and central angle 10° . For the analyses that required a longer bridge span, the culvert geometry was obtained by proportionally increasing the radii of each circular segment while keeping the constant central angles. In all the culvert geometries used in the analyses, the following ratios were used: $R_1: R_2: R_3 = 1: 0.4 : 0.56$. These ratios resulted in a uniform and proportional bridge geometry which allowed comparison of two bridges.

Finite element mesh

A finite element mesh consisting of 1095 nodes and 1059 elements was used to model the structure along with backfill soil and in-situ soil (Figure 7). The mesh was designed such that any geometry (span and crown elevation) and material properties could be modeled by changing the nodal coordinates without changing the element formation (7). This procedure greatly reduced the time required to manually generate the CANDE level 3 input files.

The corrugated metal plate was modeled using 38 beam-column elements. The reinforced sections (Figure 8) of the plate were simulated by using the modified cross sectional properties obtained from Eqs. 7-10. The backfill, foundation, and in-situ soils were modeled using 984 quadrilateral and triangular elements. The contact between the corrugated metal and backfill soil was represented by using 37 friction-contact interface elements to simulate frictional slippage.

Material Properties

Soil properties were based on AASHTO M-145 soil classification system (10). Long-span structures are typically backfilled with materials that are classified as A-1-a, A-1-b, A-3, A-2-4, and A-2-5 (11). In general, longer spans have more stringent backfill requirements. In the reported research, class A-1-b was selected. Backfill soil was modeled using the nonlinear Duncan/Selig soil model (8). Using National Corrugated Steel Pipe Association (NCSPA) design guide (11), the soil properties were obtained. In-situ soil was modeled as a linear elastic isotropic material.

Structural beams have a nominal yield strength of 50 ksi, and the nominal yield strength of corrugated plates is typically 44 ksi. Eq. 9 permits different yield strengths; however, a single

yield strength of 44 ksi was used for both the plates and the beams to obtain a conservative design.

Loading

Considering that the primary use of this bridge is to allow wildlife crossing over an interstate highway, the primary loading is dead load (taken as 140 pcf). The bridge was designed for HL-93 truck load as the live load to account for the possibility that a maintenance truck might have to cross over the structure. The anticipated construction sequence and construction loads need to be simulated in the analysis of a soil-structure system. Typically, the backfill is placed symmetrically in 6-in. to 12-in. layers and compacted to a minimum of 90% of standard proctor density (11). In the model, backfill load was applied in 20 layers (load steps) that are 18-in. deep. This loading sequence was selected to expedite generation of the models. Past experience indicates that this slight deviation from the actual thickness of layers has negligible effects on the results (12).

HL-93 truck load was applied as live load at the midspan. Equivalent two-dimensional out-of-plane distribution of the truck load was calculated using the procedure described in CANDE (9). The equivalent load was applied in four load steps (steps 21 to 24) at two nodes in the center of the span.

Strengthening Method

The original structure was strengthened by structural shapes attached to the crown of the culvert. The beams were spaced at 30-in. on center, and three structural shapes were considered: W10×12, W8×15, and W10×19.

Results and Observations

As mentioned previously, the bridge was constructed with 5/16-gage structural plates. The weight of steel used in this structure is 1676 lbs/ft. After evaluating 7-gage and 5-gage plates (thickness = 0.188-in. and 0.218-in., respectively) in conjunction with the three selected structural shapes (W10×12, W8×15, and W10×19), it was determined that the most efficient design would be to use 5-gage structural plate with W10×12 beams. The total weight of steel in the re-designed structure is 1482 lbs/ft, out of which 350 lbs/ft (23.6%) is the weight of the strengthening system. Therefore, 11.64% of structural steel can be saved by strengthening thinner structural plates, i.e., it is possible to build lighter and cost effective structures using the new strengthening system.

Attempts were made to maximize the span by using 3/8-gage (thickness = 0.380-in.) and strengthening. The longest span without stiffening was determined to be 77-ft 1-in. (rise = 30-ft 10-in.). As shown in Figure 9, the span could be increased by 11.2% to 85-ft 9-in. (rise = 34-ft 2-in.), 14.3% to 88-ft 2-in. (rise = 35-ft 7-in.), and 19.8% to 92-ft 3-in. (rise 36-ft 10-in.) by using W10×12, W8×15, and W10×19, respectively. The longer spans required additional steel. For the selected bridge, the use of W10×19 is more efficient than W10×12 and W8×15 in terms of increasing the span in comparison to the weight of the additional steel, i.e., the span could be increased nearly 19% by adding about 20% more steel. It should be noted that the selected structural shapes (W10×12, W8×15, and W10×19) were intended to illustrate the concept. Other shapes could also have been used.

Series 2

The proposed strengthening technique was further evaluated by obtaining the maximum span possible for various combinations of lateral spacing between the beams and thickness of deep corrugated plates. The stiffener beams were spaced at 15-in., 30-in., 45-in. and 60-in. Four different plate thicknesses were considered: 7-gage, 1-gage, 5/16-gage, and 3/8-gage (0.188-in., 0.280-in., 0.318-in., and 0.380-in. thick, respectively).

The analyses were similar to those conducted in Series 1. The maximum spans for different combinations of beam spacing, beam size, and plate thickness are provided in Figure 10. For all structural plate thicknesses, there is a significant increase in stiffness when the beam spacing is reduced, as reflected in the maximum possible span. The contribution of strengthening beam towards the cross-sectional properties (Eqs. 7-10) is doubled when the beam spacing is halved. Accordingly, there is a notable increase in the span length when the spacing of the beams is reduced from 60-in to 30-in., but it is not as pronounced when the spacing is decreased further to 15-in. In unstiffened metal culverts, the weakest point is in the crown region as it resists the maximum bending moment. By stiffening the crown, the moment capacity of the crown is increased and the design is governed by the capacity of the haunch, which is not stiffened (*1*). Hence, the beams spaced at less than 30-in. will have no significant effect on the performance of the culvert.

For culverts in which the crown is stiffened, the design is still governed by the axial and flexural capacity of the plate. The weakest section of the culvert is at the locations where the beam is terminated, i.e., outside the crown region. Therefore, the benefit of the strengthening system is more pronounced when it is used in conjunction with a thicker structural plate, as

evident by comparing the maximum possible spans for 7-gage plate shown in Figure 10 against what is possible for thicker plates.

Figure 10 serves as an example for designing long-span deep corrugated metal buried bridges using W10×12, W8×15, and W10×19. For any required span between 50-ft to 95-ft, the chart recommends various combinations of structural plates and beams that will provide a safe design for AASHTO HL-93 truck load. It must be noted that the performance of a buried bridge and its load carrying capacity is affected by the rise/span ratio as well as the culvert geometry, among other factors. Therefore, the information provided in Figure 10 should not be generalized as the cases illustrated in this figure are for a specific rise/span ratio (0.4) and a specific culvert geometry (3-radius arch with crown radius: haunch radius: side wall radius: 1.0: 0.40: 0.56). Other structural shapes like deeper sections such as W12×14 or W12×16, or shallow sections like W6×12 could also have been used as stiffening beams for the strengthening system.

SUMMARY AND CONCLUSIONS

The focus of the presented research was on an innovative strengthening system for buried bridges made with deep corrugated structural plates. This system consists of brackets that straddle between the crests, which is typically 15-in. for deep corrugated metal culverts, and stiffening beams that are bolted to the bracket. The effectiveness of this system was evaluated experimentally through full-scale testing of specimens constructed with flat plates and beams. Parametric design case studies were conducted to better understand the feasibility of utilizing the strengthening system to increase the span and/or use thinner plates.

The test results indicate that stiffening beams and plates do not act compositely. Using the measured data, equations were developed to determine the cross-sectional properties and flexural strength of stiffened plates.

Design case studies suggest that by using the developed stiffening system it is possible to (1) increase the span length for a given plate thickness, and (2) use a thinner plate while keeping the span the same. However, the strengthening system is more advantageous if used in conjunction with thicker plates. Considering that the addition of stiffening beams will increase the total weight of structural steel, it is recommended to analyze various combinations of corrugated plate gauges, structural beams, and beam spacing to obtain the most efficient design.

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TABLE 1 Test Matrix

Testing Phase	Specimen I.D.	Reinforcing beam	Bracket spacing
I	1	N/A	N/A
	2	N/A	N/A
	3	W8×15	48 in.
	4	W8×15	32 in.
	5	W10×12	48 in.
	6	W8×15	48 in.
	7	W6×25	48 in.
II	8	W8×15	48 in.
	9	W10×19	48 in.

TABLE 2 Test Results: Moment of Inertia

Specimen I.D.	Beam	Bracket spacing (in.)	Experimental		$I_{\text{Composite}}$ (in. ⁴)	Nominal I_{beam} (in. ⁴) ^b	I_{total} ^c (in. ⁴)	$I_{\text{exp.}} / I_{\text{total}}$
			I_{plate} (in. ⁴) ^a	$I_{\text{effective}}$ (in. ⁴)				
3	W8×15	48	25.6	73.4	157	48.0	73.6	1.00
4	W8×15	32		71.1	157	48.0	73.6	0.97
5	W10×12	48		80.2	244	53.8	79.4	1.01
6	W8×15	48		75.0	157	48.0	73.6	1.02
7	W6×25	48		76.8	231	53.4	79.0	0.97
8	W8×15	48		80.5	157	48.0	73.6	1.09
9	W10×19	48		116	355	96.3	122	0.95

^a From Specimens 1 and 2

Avg. 1.00

^b From AISC Design Manual

C.O.V. 0.048

^c $I_{\text{total}} = I_{\text{beam}} + I_{\text{plate}}$

TABLE 3 Test Results: Bending Moment

Specimen I.D.	Beam	$M_{exp.}$	$M_{n,beam}^a$	$M_{n,calc.}^b$	$M_{exp.}/M_{n,calc.}$
		(k-ft)	(k-ft)	(k-ft)	
3	W8×15	101	53.8	82.4	1.23
4	W8×15	105	53.8	82.4	1.27
5	W10×12	61	47.3	69.8	0.88
6	W8×15	102	53.8	82.4	1.24
7	W6×25	141	78.8	116.4	1.21
8	W8×15	98	53.8	82.4	1.18
9	W10×19	111	85.2	108	1.03

Avg. 1.15

C.O.V. 0.125

^a From AISC Design Manual

^b $M_{n,calc.} = M_{n,beam} (I_{total}/I_{beam})$

FIGURE 1 Previous Strengthening Methods (3) – Dimensions in mm (a) Continuous Corrugated Stiffener (b) Concrete Encased Corrugated Stiffener

Adapted from McCavour, T.P., et al. (1998) and Morrison, T. (2000)

FIGURE 2 Strengthening System

FIGURE 3 Schematic Diagram of a Strengthened Buried Bridge (backfill not shown for clarity)

FIGURE 4 Details of Test Specimens

(a) Phase I

(b) Phase II

FIGURE 5 Test Setup

FIGURE 6 Benchmark Structure (courtesy of Contech Engineering Solutions)

FIGURE 7 (a) CANDE Finite Element Mesh (b) Backfill Mesh with Construction Steps

(a)

(b)

FIGURE 8 Cross Section of Structural Steel Pipe

FIGURE 9 Comparison of Span and Weight

FIGURE 10 Effects of Stiffener spacing on (a) 3/8-gage plate (b) 5/16-gage plate (c) 1-gage plate (d) 7-gage plate

(a) 3/8-gage plate

(b) 5/16-gage plate

(c) 1-gage plate

(d) 7-gage plate

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APPENDIX A Test Setup and Instrumentation

Test Setup

All the structural plate test specimens in Table 2 were 3/8-gauge deep corrugated plates with the nominal dimensions of 15-inch pitch and 5.5-inch corrugation depth. Each plate was 12-ft long and 3.97-ft wide. The plate was strengthened using four different wide flange structural beam sections to select the most economical section. The beams were connected to the plate using 4 brackets spaced evenly at 4-ft. The effect of bracket spacing on the stiffness of the plate was also considered by connecting one specimen with bracket spaced 32-inch apart. Lateral straps were used because of concern that the plate would tend to expand laterally under load, while in field applications lateral expansion would not be possible due to adjoining corrugated plates.

The tests were conducted in two phases. In Phase I, eight specimens were tested to their ultimate capacity. The failure mode for the most of the beam sections was lateral torsional buckling. To provide lateral bracing to the top flange of the section, the two bolts connecting the bracket to the beam were extended to the top flange of the beam. Two specimens with this assembly were tested in Phase II. Schematic diagrams for both the testing phases are shown in Figures A-1 and A-2. The test matrix is shown in Table A-1. The specimens are listed in the order that they were tested. Specimens 1-7 were part of phase I, and Specimens 8 and 9 were part of phase II.

FIGURE A-1 Schematics of strengthening system for phase I

FIGURE A-2 Schematics of strengthening system for phase II

TABLE A-1 Test Matrix

Testing Phase	Specimen I.D.	Reinforcing beam	Bracket spacing
I	1	N/A	N/A
	2	N/A	N/A
	3	W8×15	48 in.
	4	W8×15	32 in.
	5	W10×12	48 in.
	6	W8×15	48 in.
	7	W6×25	48 in.
II	8	W8×15	48 in.
	9	W10×19	48 in.

Loading

The test specimens were tested in the four-point bending arrangement shown in the Figure A-3.

The specimens had 12-ft simple span between the rocker supports, the shear span was 4-ft in all the specimens, and the distance between the load points was 4-ft in all the specimens.

FIGURE A-3 Loading arrangement

The force from a 220-kip servo-valve controlled hydraulic actuator was divided through a W14×145 spreader beam (see Figure A-4) that was supported by a pair of roller supports resting

on top flange of the stiffening beam (Figure A-5). For the test specimens without a stiffening beam, the load was transferred through brackets.

FIGURE A-4 Test specimens without stiffening beam (Specimens 1 and 2)

FIGURE A-5 Test specimens with stiffening beam (Phase I)

The specimens were tested in position control, i.e., each specimen was pushed through a series of displacement increments until the beam experienced excessive deformation or the beam

had excessive local or global buckling. The following displacement increments were used for all the specimens.

TABLE A-2 Displacement Increments

Deflection (in.)	Increment (in.)
0 to $\frac{1}{2}$	0.005
$\frac{1}{2}$ to 1	0.010
1 to 2	0.020
After 2	0.050

Instrumentation

The load cell of the hydraulic actuator measured the applied load. The test specimens were instrumented to monitor the deflection of the plate at four locations. The average value of the deflection was used to analyze the test data. Figure A-6 illustrates the instrumentation.

FIGURE A-6 Measurement of vertical deflections

APPENDIX B Laboratory Test Results

The measured load deflection relationship was plotted and the linear portion of each plot was used to calculate the flexural rigidity of the system. Using the modulus of elasticity of steel as 29,000 ksi, the effective moment of inertia of the beam was calculated. Sample calculations for Specimens 1 and 3 are shown below.

Calculation for Specimen 1

FIGURE B-1 Linear portion of load deflection plot for Specimen 1

Specimen 1 had an 3/8-gauge plate. Slope of the linear part of the load deflection curve is 8.07 kips/in.

$$Slope = \frac{P}{\Delta} = 8.07 \text{ kips/in}$$

Deflection of the plate in this loading arrangement is given by

$$\Delta = \frac{Pa}{6EI} (3La - 4a^2)$$

where $a = 48$ -inch and $L = 144$ -inch.

On rearranging the equation, we can calculate the value of EI as,

$$EI = \frac{Pa}{6\Delta} (3La - 4a^2)$$

$$EI = \frac{8.07 \times 48 \times (3 \times 144 \times 48 - 4 \times 48^2)}{6} = 743714.7 \text{ kip} - \text{in}^2$$

$$I = \frac{743714.7}{29000} = 25.6 \text{ in}^4$$

To get an accurate value for the stiffness of the plate without stiffeners, two plates were tested to their maximum loading capacity. The moment of inertia of the second plate (Specimen 2) was also calculated to be 25.6 in^4 .

Calculation for Specimen 3

FIGURE B-2 Linear portion of Load Deflection curve for Specimen 3

Specimen 3 had an 3/8-gauge plate reinforced with W8×15 beam. The slope of this line is equal to 23.09 kips/in.

$$Slope = \frac{P}{\Delta} = 23.09 \text{ kips/in}$$

$$EI = \frac{23.09 \times 48 \times (3 \times 144 \times 48 - 4 \times 48^2)}{6} = 2128715 \text{ kip} - \text{in}^2$$

$$I = \frac{2128715}{29000} = 73.4 \text{ in}^4$$

The calculations for all the specimens are summarized in Table B-1.

TABLE B-1 Summary of Calculations

Specimen I.D.	Beam	Bracket spacing (in.)	Experimental		Nominal	$I_{\text{total}}^{\text{d}}$	$I_{\text{exp.}}/I_{\text{total}}$
			$I_{\text{plate}} (\text{in.}^4)^{\text{a}}$	$I_{\text{effective}} (\text{in.}^4)$	$I_{\text{beam}} (\text{in.}^4)^{\text{b}}$	(in. ⁴)	
3	W8×15	48	25.6	73.4	48.0	73.6	1.00
4	W8×15	32		71.1	48.0	73.6	0.97
5	W10×12	48		80.2	53.8	79.4	1.01
6	W8×15	48		75.0	48.0	73.6	1.02
7	W6×25	48		76.8	53.4	79.0	0.97
8	W8×15	48		80.5	48.0	73.6	1.09
9	W10×19	48		116	96.3	122	0.95

^a From Specimens 1 and 2

Avg. 1.00

^b $I_{\text{total}} = I_{\text{beam}} + I_{\text{plate}}$

C.O.V. 0.048

Discussion on Unbraced Length

Based on the experimental observations and basic mechanics principles, the ultimate moment carrying capacity of the system can be calculated by using Eq. B-1.

$$M_{system} = \left(\frac{I_{beam} + I_{plate}}{I_{beam}} \right) M_{beam, AISC} \quad \text{Eq. B-1}$$

$M_{beam, AISC}$ is the nominal capacity of the beam, calculated from the AISC flexure provisions. Per AISC 360-10 flexure provisions, nominal capacity of the beam depends on the unbraced length of the beam (L_b). Unbraced length is defined as the length between points that are either braced against lateral displacement of the compression flange or braced against twist of the cross section (6).

In the laboratory testing, the beams were connected to the plate using 4 brackets spaced at 48-in. and loaded using the spreader beam in a four-point bending arrangement as seen in Figures A-4 and A-5. In the first testing phase (Specimens 3-7) the bottom flange of the beam was connected to the bracket and the plate (Figure A-1). The failure mode for this test setup was lateral torsional buckling. To provide lateral bracing to the top flange of the section, the two bolts connecting the bracket to the beam were extended to the top flange of the beam (Figure A-2). Two specimens (Specimens 8 and 9) with top flange braced at the location of the brackets were tested in Phase II.

To determine the unbraced length of the beam in this system, moments for unbraced length 48-in. and 144-in. were compared. The moments are given in Tables B-2 and B-3.

TABLE B-2 Moment comparison for $L_b=48$ -in

Specimen I.D.	$P_{\max}$ kips	$M_{\max}$ kip-ft	$Mn, L_b=48''$	$M_{pl+beam}$	$M_{\max}/M_{pl+beam}$
3	50.00	100.00	56.67	86.89	1.15
4	52.49	104.98	56.67	86.89	1.20
5	30.59	61.18	52.11	76.91	0.79
6	50.91	101.82	56.67	86.89	1.17
7	70.55	141.10	78.75	116.50	1.21
8 (Phase II)	48.81	97.62	56.67	86.89	1.12
9 (Phase II)	55.41	110.81	90.00	113.93	0.97

TABLE B-3 Moment comparison for $L_b=144$ -in

Specimen I.D.	$P_{\max}$ kips	$M_{\max}$ kip-ft	$Mn, L_b=144''$	$M_{pl+beam}$	$M_{\max}/M_{pl+beam}$
3	50.00	100.00	30.79	47.21	2.12
4	52.49	104.98	30.79	47.21	2.22
5	30.59	61.18	17.55	25.91	2.38
6	50.91	101.82	30.79	47.21	2.17
7	70.55	141.10	78.75	116.50	1.20
8 (Phase II)	48.81	97.62	30.79	47.21	2.08
9 (Phase II)	55.41	110.81	46.38	58.71	1.88

For adequate safety, the actual moment capacity of the system would have to be greater than the analytical capacity. The ratio $M_{\max}/M_{pl+beam}$ is the ratio of experimental value of the moment to the analytical value. Hence, in these calculations, the ratio $M_{\max}/M_{pl+beam}$ is desired to be greater than 1. For unbraced length of 48-in., it is observed that Eq. B-1 gives values closer in the acceptable range of the applied moment except specimen 5. Specimen 5 was the only beam section with a non-compact flange, and it failed at a lower load than the other beams. The results also indicate that there is no significant difference between the Phase I and Phase II strengths. Extending the bolts to the top flange did not apparently provide any appreciable lateral support to the top flange. The unbraced length of 48-in. is more appropriate for all the specimens. This observation is attributed to the fact that the external loading assembly provided some lateral support to the top flange, which prevented lateral buckling. In an actual buried bridge, stability of

the top flange is not anticipated to be an issue because the soil around the steel plate, compacted to 90% or more, will provide sufficient bracing to the beam.

The average of all the $M_{\max}/M_{pl+beam}$ ratios is 1.15 for $L_b = 4$ ft. and 2.01 for $L_b = 12$ ft. Hence, Eq. B-1 gives a reasonable estimate of the moment capacity of the system and it agrees with the experimental values. A better match is possible by using $L_b = 4$ ft. because of the apparent restraints provided by the loading apparatus at the load points. This comparison suggests that Eq. B-1 can be used to obtain a reasonable, yet conservative, estimate of the flexural strength of corrugated structural plates strengthened by a structural steel beam.

APPENDIX C CANDE Modelling

Finite element analyses were conducted using the Level 3 solution method in CANDE (8). The model input is given in a text file format as it makes it easier to modify and edit the input file.

The various input properties for the model are discussed below.

Mesh Geometry

Mesh geometry included x and y coordinates of each node and nodal connectivity to form the elements and boundary conditions.

Nodal Co-ordinate Input

Nodal coordinates for each culvert shape were generated using an MS Excel specifically written for this project. The spreadsheet requires the user to input the values for the radii and the angles for the culvert shape. By varying the radii and the angles, it is possible to obtain geometry for any given span and rise. The coordinates obtained from the spreadsheet were directly used in the CANDE input file.

Element Input

CANDE elements are defined using seven numerical values. The first four numbers determine the nodes connecting the elements in counterclockwise order. Triangular elements are defined with three nodes and the fourth number is set to 0. The fifth number assigns a material property to the element and the sixth number assigns the loading step in which the element is created.

The last number determines if the element is a regular element or an interface element. Element input used for all the analysis is given below. By varying the nodal input, it was possible to use the same element input for any culvert geometry. Several trial analyses were conducted to

ensure that the mesh is free of errors and to make sure that convergence issues do not arise due to slender elements.

1	38	39	0	0	1	1	0
2	37	38	0	0	1	1	0
3	36	37	0	0	1	1	0
4	35	36	0	0	1	1	0
5	34	35	0	0	1	1	0
6	33	34	0	0	1	1	0
7	32	33	0	0	1	1	0
8	31	32	0	0	1	1	0
9	30	31	0	0	1	1	0
10	29	30	0	0	1	1	0
11	28	29	0	0	1	1	0
12	27	28	0	0	1	1	0
13	26	27	0	0	1	1	0
14	25	26	0	0	1	1	0
15	24	25	0	0	1	1	0
16	23	24	0	0	1	1	0
17	22	23	0	0	1	1	0
18	21	22	0	0	1	1	0
19	20	21	0	0	1	1	0
20	19	20	0	0	1	1	0
21	18	19	0	0	1	1	0
22	17	18	0	0	1	1	0
23	16	17	0	0	1	1	0
24	15	16	0	0	1	1	0
25	14	15	0	0	1	1	0
26	13	14	0	0	1	1	0
27	12	13	0	0	1	1	0
28	11	12	0	0	1	1	0
29	10	11	0	0	1	1	0
30	9	10	0	0	1	1	0
31	8	9	0	0	1	1	0
32	7	8	0	0	1	1	0
33	6	7	0	0	1	1	0
34	5	6	0	0	1	1	0
35	4	5	0	0	1	1	0
36	3	4	0	0	1	1	0
37	2	3	0	0	1	1	0
38	1	2	0	0	1	1	0
39	358	357	369	370	3	2	0
40	357	356	368	369	3	2	0
41	356	355	367	368	3	2	0
42	355	354	366	367	3	2	0
43	354	353	365	366	3	2	0
44	353	1	315	365	3	2	0
45	39	359	371	332	3	2	0
46	359	360	372	371	3	2	0
47	360	361	373	372	3	2	0
48	361	362	374	373	3	2	0
49	362	363	375	374	3	2	0
50	363	364	376	375	3	2	0
51	364	343	344	376	3	2	0
52	334	370	382	383	3	3	0
53	370	369	381	382	3	3	0
54	369	368	380	381	3	3	0
55	368	367	379	380	3	3	0
56	367	366	378	379	3	3	0
57	366	365	377	378	3	3	0
58	365	315	316	377	3	3	0

59	332	371	384	331	3	3	0
60	371	372	385	384	3	3	0
61	372	373	386	385	3	3	0
62	373	374	387	386	3	3	0
63	374	375	388	387	3	3	0
64	375	376	389	388	3	3	0
65	376	344	390	389	3	3	0
66	344	345	390	0	3	3	0
67	383	382	396	397	3	4	0
68	382	381	395	396	3	4	0
69	381	380	394	395	3	4	0
70	380	379	393	394	3	4	0
71	379	378	392	393	3	4	0
72	378	377	391	392	3	4	0
73	377	316	317	391	3	4	0
74	331	384	398	330	3	4	0
75	384	385	399	398	3	4	0
76	385	386	400	399	3	4	0
77	386	387	401	400	3	4	0
78	387	388	402	401	3	4	0
79	388	389	403	402	3	4	0
80	389	390	404	403	3	4	0
81	390	345	346	404	3	4	0
82	397	396	410	411	3	5	0
83	396	395	409	410	3	5	0
84	395	394	408	409	3	5	0
85	394	393	407	408	3	5	0
86	393	392	406	407	3	5	0
87	392	391	405	406	3	5	0
88	391	317	318	405	3	5	0
89	330	398	412	329	3	5	0
90	398	399	413	412	3	5	0
91	399	400	414	413	3	5	0
92	400	401	415	414	3	5	0
93	401	402	416	415	3	5	0
94	402	403	417	416	3	5	0
95	403	404	418	417	3	5	0
96	404	346	347	418	3	5	0
97	333	358	370	334	3	2	0
98	334	383	335	0	3	3	0
99	335	383	397	336	3	4	0
100	336	397	411	337	3	5	0
101	337	411	425	338	3	6	0
102	411	410	424	425	3	6	0
103	410	409	423	424	3	6	0
104	409	408	422	423	3	6	0
105	408	407	421	422	3	6	0
106	407	406	420	421	3	6	0
107	406	405	419	420	3	6	0
108	405	318	319	419	3	6	0
109	329	412	426	328	3	6	0
110	412	413	427	426	3	6	0
111	413	414	428	427	3	6	0
112	414	415	429	428	3	6	0
113	415	416	430	429	3	6	0
114	416	417	431	430	3	6	0
115	417	418	432	431	3	6	0
116	418	347	348	432	3	6	0
117	338	425	439	339	3	7	0
118	425	424	438	439	3	7	0
119	424	423	437	438	3	7	0
120	423	422	436	437	3	7	0
121	422	421	435	436	3	7	0
122	421	420	434	435	3	7	0
123	420	419	433	434	3	7	0
124	419	319	320	433	3	7	0

125	328	426	440	327	3	7	0
126	426	427	441	440	3	7	0
127	427	428	442	441	3	7	0
128	428	429	443	442	3	7	0
129	429	430	444	443	3	7	0
130	430	431	445	444	3	7	0
131	431	432	446	445	3	7	0
132	432	348	349	446	3	7	0
133	339	454	340	0	3	8	0
134	339	439	453	454	3	8	0
135	439	438	452	453	3	8	0
136	438	437	451	452	3	8	0
137	437	436	450	451	3	8	0
138	436	435	449	450	3	8	0
139	435	434	448	449	3	8	0
140	434	433	447	448	3	8	0
141	433	320	321	447	3	8	0
142	327	440	455	326	3	8	0
143	440	441	456	455	3	8	0
144	441	442	457	456	3	8	0
145	442	443	458	457	3	8	0
146	443	444	459	458	3	8	0
147	444	445	460	459	3	8	0
148	445	446	461	460	3	8	0
149	446	349	462	461	3	8	0
150	349	350	462	0	3	8	0
151	340	454	470	341	3	9	0
152	454	453	469	470	3	9	0
153	453	452	468	469	3	9	0
154	452	451	467	468	3	9	0
155	451	450	466	467	3	9	0
156	450	449	465	466	3	9	0
157	449	448	464	465	3	9	0
158	448	447	463	464	3	9	0
159	447	321	322	463	3	9	0
160	326	455	471	325	3	9	0
161	455	456	472	471	3	9	0
162	456	457	473	472	3	9	0
163	457	458	474	473	3	9	0
164	458	459	475	474	3	9	0
165	459	460	476	475	3	9	0
166	460	461	477	476	3	9	0
167	461	462	478	477	3	9	0
168	462	350	351	478	3	9	0
169	341	487	342	0	3	10	0
170	341	470	486	487	3	10	0
171	470	469	485	486	3	10	0
172	469	468	484	485	3	10	0
173	468	467	483	484	3	10	0
174	467	466	482	483	3	10	0
175	466	465	481	482	3	10	0
176	465	464	480	481	3	10	0
177	464	463	479	480	3	10	0
178	463	322	323	479	3	9	0
179	325	471	488	324	3	9	0
180	471	472	489	488	3	10	0
181	472	473	490	489	3	10	0
182	473	474	491	490	3	10	0
183	474	475	492	491	3	10	0
184	475	476	493	492	3	10	0
185	476	477	494	493	3	10	0
186	477	478	495	494	3	10	0
187	478	351	496	495	3	10	0
188	351	352	496	0	3	10	0
189	342	487	225	214	3	11	0
190	487	486	243	225	3	11	0

191	486	485	259	243	3	11	0
192	485	484	273	259	3	11	0
193	484	483	285	273	3	11	0
194	483	482	295	285	3	11	0
195	482	481	303	295	3	11	0
196	481	480	309	303	3	11	0
197	480	479	313	309	3	11	0
198	479	323	59	313	3	9	0
199	324	488	314	77	3	9	0
200	488	489	311	314	3	11	0
201	489	490	306	311	3	11	0
202	490	491	299	306	3	11	0
203	491	492	290	299	3	11	0
204	492	493	279	290	3	11	0
205	493	494	266	279	3	11	0
206	494	495	251	266	3	11	0
207	495	496	234	251	3	11	0
208	496	352	224	234	3	11	0
209	214	225	226	213	3	12	0
210	225	243	244	226	3	12	0
211	243	259	260	244	3	12	0
212	259	273	274	260	3	12	0
213	273	285	286	274	3	12	0
214	285	295	296	286	3	12	0
215	295	303	304	296	3	12	0
216	303	309	310	304	3	12	0
217	309	313	78	310	3	12	0
218	313	59	78	0	3	9	0
219	59	60	96	78	3	9	0
220	60	61	112	96	3	9	0
221	61	62	128	112	3	9	0
222	62	63	142	128	3	9	0
223	63	64	156	142	3	9	0
224	64	65	168	156	3	9	0
225	65	66	180	168	3	9	0
226	66	67	190	180	3	9	0
227	67	68	200	190	3	9	0
228	68	69	195	200	3	9	0
229	69	70	185	195	3	9	0
230	70	71	174	185	3	9	0
231	71	72	162	174	3	9	0
232	72	73	149	162	3	9	0
233	73	74	135	149	3	9	0
234	74	75	120	135	3	9	0
235	75	76	104	120	3	9	0
236	76	77	87	104	3	9	0
237	314	311	312	87	3	12	0
238	311	306	307	312	3	12	0
239	306	299	300	307	3	12	0
240	299	290	291	300	3	12	0
241	290	279	280	291	3	12	0
242	279	266	267	280	3	12	0
243	266	251	252	267	3	12	0
244	251	234	235	252	3	12	0
245	234	224	223	235	3	12	0
246	213	226	227	212	3	13	0
247	226	244	245	227	3	13	0
248	244	260	261	245	3	13	0
249	260	274	275	261	3	13	0
250	274	286	287	275	3	13	0
251	286	296	297	287	3	13	0
252	296	304	305	297	3	13	0
253	304	310	79	305	3	13	0
254	310	78	79	0	3	13	0
255	78	96	79	0	3	13	0
256	87	88	104	0	3	13	0

257	87	312	88	0	3	13	0
258	312	307	308	88	3	13	0
259	307	300	301	308	3	13	0
260	300	291	292	301	3	13	0
261	291	280	281	292	3	13	0
262	280	267	268	281	3	13	0
263	267	252	253	268	3	13	0
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287	236	222	221	237	3	14	0
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290	246	262	263	247	3	15	0
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306	302	293	294	90	3	15	0
307	293	282	283	294	3	15	0
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309	269	254	255	270	3	15	0
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311	237	221	220	238	3	15	0
312	210	229	230	209	3	16	0
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358	278	82	83	0	3	17	0
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382	108	91	92	109	3	17	0
383	91	284	92	0	3	17	0
384	284	271	272	92	3	17	0
385	271	256	257	272	3	17	0
386	256	239	240	257	3	17	0
387	239	219	218	240	3	17	0
388	208	231	232	207	3	18	0

389	231	249	250	232	3	18	0
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455	103	119	42	41	3	20	0
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747	542	543	551	550	4	3	0	0	0	0
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761	558	559	567	566	4	5	0	0	0	0
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768	566	567	575	574	4	6	0	0	0	0
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775	574	575	583	582	4	7	0	0	0	0
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955	742	743	751	750	4	7	0	0	0	0
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1052	852	853	861	860	4	20	0	1	1	8
1053	854	855	863	862	4	20	0	0	0	0
L1059	860	861	869	868	4	20	0	1	1	8

Boundary Conditions

Supports at the boundary of the in-situ soil are defined by restricting the deflections in any direction at the required nodes. It is necessary to know the boundary node numbers before writing the input for boundary conditions. Live load is applied to any desired node as a boundary condition force.

Input for boundary conditions is given below.

534	1	0.000	0	0.000	0.000	1	0	0	0.000	0.000
542	1	0.000	0	0.000	0.000	2	0	0	0.000	0.000
550	1	0.000	0	0.000	0.000	3	0	0	0.000	0.000
558	1	0.000	0	0.000	0.000	4	0	0	0.000	0.000
566	1	0.000	0	0.000	0.000	5	0	0	0.000	0.000
574	1	0.000	0	0.000	0.000	6	0	0	0.000	0.000
582	1	0.000	0	0.000	0.000	7	0	0	0.000	0.000
590	1	0.000	0	0.000	0.000	8	0	0	0.000	0.000
598	1	0.000	0	0.000	0.000	9	0	0	0.000	0.000
606	1	0.000	0	0.000	0.000	10	0	0	0.000	0.000
614	1	0.000	0	0.000	0.000	11	0	0	0.000	0.000
622	1	0.000	0	0.000	0.000	12	0	0	0.000	0.000
630	1	0.000	0	0.000	0.000	13	0	0	0.000	0.000
638	1	0.000	0	0.000	0.000	14	0	0	0.000	0.000
646	1	0.000	0	0.000	0.000	15	0	0	0.000	0.000
654	1	0.000	0	0.000	0.000	16	0	0	0.000	0.000
662	1	0.000	0	0.000	0.000	17	0	0	0.000	0.000
670	1	0.000	0	0.000	0.000	18	0	0	0.000	0.000
678	1	0.000	0	0.000	0.000	19	0	0	0.000	0.000
686	1	0.000	0	0.000	0.000	20	0	0	0.000	0.000
694	1	0.000	0	0.000	0.000	20	0	0	0.000	0.000
709	1	0.000	0	0.000	0.000	1	0	0	0.000	0.000
717	1	0.000	0	0.000	0.000	2	0	0	0.000	0.000
725	1	0.000	0	0.000	0.000	3	0	0	0.000	0.000
733	1	0.000	0	0.000	0.000	4	0	0	0.000	0.000
741	1	0.000	0	0.000	0.000	5	0	0	0.000	0.000
749	1	0.000	0	0.000	0.000	6	0	0	0.000	0.000
757	1	0.000	0	0.000	0.000	7	0	0	0.000	0.000
765	1	0.000	0	0.000	0.000	8	0	0	0.000	0.000
773	1	0.000	0	0.000	0.000	9	0	0	0.000	0.000
781	1	0.000	0	0.000	0.000	10	0	0	0.000	0.000
789	1	0.000	0	0.000	0.000	11	0	0	0.000	0.000
797	1	0.000	0	0.000	0.000	12	0	0	0.000	0.000
805	1	0.000	0	0.000	0.000	13	0	0	0.000	0.000
813	1	0.000	0	0.000	0.000	14	0	0	0.000	0.000
821	1	0.000	0	0.000	0.000	15	0	0	0.000	0.000
829	1	0.000	0	0.000	0.000	16	0	0	0.000	0.000
837	1	0.000	0	0.000	0.000	17	0	0	0.000	0.000
845	1	0.000	0	0.000	0.000	18	0	0	0.000	0.000
853	1	0.000	0	0.000	0.000	19	0	0	0.000	0.000
861	1	0.000	0	0.000	0.000	20	0	0	0.000	0.000
869	1	0.000	0	0.000	0.000	20	0	0	0.000	0.000
870	1	0.000	1	0.000	0.000	1	0	0	0.000	0.000
871	1	0.000	1	0.000	0.000	1	0	0	0.000	0.000

[illegible]

Material Properties

Properties for soil elements were determined by their location in the mesh. In-situ soil elements (material ID 1) were defined as simple linear-elastic-isotropic elements with a Young's modulus of 3 ksi and Poisson's ratio of 0.3. Footing elements (material ID 2) were defined as very stiff simple linear-elastic-isotropic elements with a Young's modulus of 3,500 ksi and Poisson's ratio of 0.18. Backfill elements (material ID 3) were defined using a nonlinear soil model called Duncan/Selig formulation. Well-graded sand compacted up to 95% of the Standard Proctor Density, identified as SW95, was used in the models.

Pipe elements were modelled as non-linear elements with modulus of elasticity of 29 ksi, Poisson's ratio of 0.3, and yield stress of 44 ksi. Cross sectional properties for the pipe elements used in the analysis are summarized in Table C-1. Input for strengthened portions of the plate are calculated using Eqs. C-1, C-2, C-3, and C-4. The beams are at 30-inch spacing.

$$A_{total} = \frac{A_{beam}}{s} + A_{plate} \quad \text{Eq. C-1}$$

$$I_{total} = \frac{I_{beam}}{s} + I_{plate} \quad \text{Eq. C-2}$$

$$S_{total} = \frac{S_{beam}}{s} \left(\frac{F_{y, beam}}{F_{y, plate}} \right) + S_{plate} \quad \text{Eq. C-3}$$

$$Z_{total} = \frac{Z_{beam}}{s} \left(\frac{F_{y, beam}}{F_{y, plate}} \right) + Z_{plate} \quad \text{Eq. C-4}$$

TABLE C-1 Sectional properties

Structural Member	Cross sectional area (in²/in)	Moment of Inertia (in⁴/in)	Section Modulus (in³/in)	Weight (lb/ft²)
3/8 Ga (0.380 in)	0.525	2.013	0.6423	20.21
5/16 Ga (0.318 in)	0.4375	1.6730	0.5389	16.98
5 Ga (0.218 in)	0.3003	1.1436	0.374	11.47
W10×12	0.1180	1.7933	0.3633	4.8
W8×15	0.148	1.6	0.3933	6
W10×19	0.1873	3.21	0.624	7.6

Weight of the beams is calculated by assuming that a single beam stiffens a 30-in. width of the plate.

For example, for a W10×12 with 30-in. spacing, weight of the beam = $\frac{12}{30} = 0.4 \frac{lb}{in} = 0.4 \times 12 = 4.8 \frac{lb}{ft}$

Live Load

Live load was calculated using the procedure recommended in CANDE tutorial manual (9).

AASHTO HL-93 truck loading was used in the analyses. A schematic figure depicting the spread of the load in the soil between the crown of the culvert and the tires is given in Figure C-1. The calculations for determining the equivalent truck load are provided below.

FIGURE C-1 Schematic diagram of wheel load (9)

Height of Cover (H_E): 72-in. (6-ft)

$P_{\text{axle}} = 32$ kips, $P_{\text{tyre}} = 16$ kips

Axle width = 6-ft, assumed contact area: 20-in. $\times$ 20-in.

Live load distribution factor (LLDF): 1.15

Multiple presence factor: 1.2

Live load factor: 1.0

Impact factor:

$$IM = 33 \times (1.0 - 0.125 \times 6) = 8.25$$

$$I_{impact} = \max\left(1, 1 + \frac{8.25}{100}\right) = 1.0825$$

$$E_{deep_wheel} = W_{tire} + H \times LLDF = 20 + 72 \times 1.15 = 102.8 \text{ in}$$

$$E_{deep_axle} = W_{axle} + H \times LLDF = 72 + 72 \times 1.15 = 174.8 \text{ in}$$

Equivalent wheel load:

$$LL = \frac{16 \times 1.0825 \times 1.2}{102.8} = 252.72 \text{ kips}$$

$$LL = \frac{16 \times 1.0825 \times 1.2}{174.8} = 148.62 \text{ kips}$$

Governing Live load = 252.72 kips.

Load was applied in 4 loading steps to avoid convergence problems. Hence, live load per step = 63.18 kips.

Live load was applied to nodes 48 and 49 in load steps 21-24.