ANALYSIS OF A CORRUGATED STEEL BOX-TYPE CULVERT

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ABSTRACT

The structural performance of a rib-reinforced, box-type culvert is investigated with regard to measured deflections and strains that occurred during the construction sequence and live loading. Calculated moments and thrusts, and deflections are compared to results obtained from a plane-strain finite element analysis using two available programs, CANDE and SEQCON. Higher order soil and beam elements were utilized in SEQCON computer runs to determine if greater accuracy could be obtained in predicting field response. Hyperbolic models are employed for the backfill material and the construction sequence is simulated as closely as possible. Symmetrical and unsymmetrical live loading are modeled. Soil elements are assumed to be fully bonded to the culvert-beam elements as no interface element was utilized. Compaction induced stresses are not incorporated into the finite element analysis.
Chapter 1

INTRODUCTION

1.1 Description Of Problem

Corrugated metal box-type culverts have become increasingly popular in recent decades. They were designed to meet the need of a structure capable of providing a large cross sectional area where vertical clearance is limited, and as such, have large spans compared to their rises. They are primarily used as replacements for short span bridges. Economy is the advantage to using low profile box culverts and is facilitated by using reinforcing ribs to strengthen and permit the use of light weight corrugated sheets.

Because box-type culverts have nearly flat crowns, standard design methods that are applicable to round culverts do not apply. This is primarily due to a box-type culverts inability to carry the major portion of it's load through arch action. In addition, they represent a complex, nonlinear soil-structure interaction problem and as such require the use of the finite element method to predict their response during construction sequences and live load.
To prevent excessive deflections from occurring during backfill and to improve buckling resistance, reinforcing ribs have been incorporated into the design of many box-type culverts. Reinforcing rib stiffeners are employed when shallow cover conditions prevail. Culverts subject to shallow cover conditions with inadequate flexural stiffness may experience failure when heavy live loads are imposed. Therefore, there is a need to investigate the performance of box-type culverts with reinforcing ribs, subject to shallow cover conditions, in order to establish realistic design procedures for them.

1.2 Objectives

The objectives of this study are to:

1) Evaluate results from field tests performed on the Lane Products Company low profile box culvert.

2) Compare field test results to culvert response as predicted by the finite element computer programs CANDE and SEQCON to verify their validity as they apply to the design of box-type culverts with reinforcing-ribs.

3) To determine which of the two computer programs (CANDE and SEQCON) is more suited for predicting culvert response during the construction sequence and live load application.

4) Determine the soil interaction between the #603 backfill material and the Lane culvert test-structure.
1.3 Literature Review

Box-type culverts have been the subject of numerous investigations in recent years. Duncan, Seed and Drawsky [1], performed finite element analyses on aluminum box type culverts with different spans and rises, subject to varying cover depths and live loads, to develop design equations for crown and haunch moment capacities. To verify the validity of their finite element analyses, they performed a full-scale field test on a box-type culvert having a span of 17 feet 6 in. and a rise of 6 feet 2 in. They concluded that:

1) Actual deflections due to live load are likely to be about 1/4 as large as deflections predicted from the FEM.
2) Actual bending moments in the crown sections are likely to be about the same as those predicted from the FEM.
3) Actual bending moments in the haunch sections are likely to be significantly smaller than those predicted from the FEM.
4) For cover depths less than five feet, the effects of axial forces are less significant than the effects of bending moments, and as such, bending moments need only be the major design consideration.
5) Culvert rise has minimal effects on live load moments— but culverts with small rises experience larger live load moments than do culverts with higher rises.
6) The critical live load position is always at or near center span of the culvert.
7) Good quality backfill and higher degrees of compaction result in smaller moments.

Selig and Musser [2], evaluated the performance of a rib-reinforced long span culvert by means of field tests and a limited finite element analysis and compared the results with current design practice. The computer analysis was performed with CANDE and NLSSIP. They assumed fully composite action between the rib stiffener and the corrugated shell. The soil was modeled with the Hyperbolic Model in the computer analysis and incremental backfill placement was done in six lifts. The results of their study indicate that:
1) Rib stiffeners decrease deflections, but increase moments. Thrusts are essentially unaffected by the ribs.
2) Allowing interface slip with limiting friction increased deflections.
3) Increasing soil stiffness decreased deflections and crown moments.
4) Reinforcing-ribs help to control the shape and magnitude of deflections during backfill placement and compaction, and increase the factor of safety against plastic hinge formation.

Another investigation conducted by Leonards, Juang, Wu
and Stetkar [3], utilized CANDE to predict conduit behavior with regard to using different soil models and soil-structure interface conditions and the importance of the sequence in which backfill layers are placed. Yielding and buckling of the conduit wall was also investigated.

The results of their study indicate that the Duncan-Chang soil model gives results that are compatible with field experience. They also concluded that soil-structure interface strongly affects conduit response and that a fully bonded interface condition is unrealistic. In addition, they concluded that the sequence of soil placement strongly affects conduit response and as such, there is a need to closely follow field construction sequences if meaningful comparisons are to be made.

Lane Metal Products Company [4] performed a load test on one of their manufactured low profile box culverts. The structure tested had a 20 ft. 10 in. span and 6 ft. 3 in. rise. Backfill lifts were kept at 6 inches and were compacted using a whacker and dual drum vibratory roller. Load tests were performed for 2 feet and 5 feet of cover respectively. The backfill material consisted of washed graduated aggregate from sands up to 2 inch nominal diameter stone and the live load was 33,830 pounds as measured on the rear tandem axle.
For 2 feet of cover, the maximum measured deflection was 1.308\" and for 5 feet of cover the maximum measured deflection was .336\". There was no yielding of material, bolt failure or plate slippage detected and the structure was considered satisfactory under the load test procedure.

Effects of compaction induced stresses and deformations were investigated by Seed and Ou [5] for various backfill levels placed about a long-span culvert. The culvert had a 38.7 ft. span and a 15.8 ft. rise. Stiffening ribs were spaced on 9" centers. Field measurements were compared with the results of two finite element analyses to investigate backfill compaction effects on culvert response. One FEM analysis had the capability of modeling compaction induced stresses and deformations, while the other one could not.

The results of their analysis indicates that modeling compaction effects significantly improves the agreement between actual field measurements and those determined from a finite element analysis. In addition, finite element analyses that do not model compaction induced effects are likely to underestimate culvert deflections, stresses due to bending, and to a lesser degree, axial thrust.

Buckling of culvert structures was investigated by Ghobrial and Abdel-Sayed [6]. In their study of this
phenomenon they took into consideration the formation of plastic hinges in the culvert walls. The model used to represent the soil consisted of springs (both normal and tangential) attached to the culvert at the nodal points of each beam element. Coefficients of the soil springs depended on the type of backfill, location of the springs in relation to the backfill height and the direction of displacement.

They observed sudden snap-through failure to occur when large spans and/or shallow cover conditions prevailed. Short span culverts with deep cover did not exhibit this response but, instead, experienced displacements with a larger margin of increase after each load step. In addition, they observed the first plastic hinge to form at the crown and the second hinge formation to be dependent on the culvert span and depth of cover. In culverts with large spans, the second hinge formation tends to occur further away from the crown.

The inelastic flexural stability of corrugations was examined by Raymond L. Cary [7]. The study investigated the local stability of arc-and-tangent corrugated profiles when they were subjected to inelastic bending. Empirically derived relationships from 24 flexural tests were presented and related critical corrugation dimensions to the minimum curving radius, the ultimate moment capacity and the
critical buckling strain. The results of their analysis indicates that as the radius of curvature to thickness ratio increases, the strain at which buckling occurs is reduced.
Chapter 2

DESCRIPTION OF FIELD INSTRUMENTATION

2.1 Introduction

In the sections that follow, a description of the culvert tested and the test site is given. A detailed description of the instrumentation scheme employed and the methods used to mount data measuring devices to the test-structure is also provided. In addition, figures which illustrate the locations of the data measuring devices along the culvert periphery and in the surrounding backfill are given.

2.1.1 Lane Culvert Description

The Lane Culvert selected for testing is a low profile box-type culvert of average size for this type of structure with a span of 15'-9" and a rise of 5'-0". The length of the culvert is approximately 40 feet. Unlike standard circular shaped culverts, the box-shaped geometry of this structure renders it capable of developing large compressive forces. To improve buckling resistance, excessive deflections during
backfill and the development of plastic hinges, corrugated reinforcing-ribs are included in its design.

The test-structure is constructed of 6x2 inch corrugated structural steel plate shell, reinforced with corrugated ribs on both the interior and exterior in the crown regions. In the haunch and side regions, rib-reinforcement is limited to the exterior only. The reinforcing-ribs are 6x3 inches and are placed on 24" centers. In Fig.(2.1) a photograph of the free-standing structure is shown before instrumentation and in Fig.(2.2) cross sections of the crown and haunch regions are provided.

Fig. 2.1 Free Standing Structure
Measurement of the culvert plate and rib thickness was done in the field using a micrometer and the corresponding values are given in Table 2.1. These values were used in calculations of the geometric properties that were needed to determine moments and thrusts from field data and were also incorporated into the CANDE program.

Table 2.1 Measured Rib And Plate Thicknesses

| Crown rib (exterior)  | .1624 |
| Crown rib (interior) | .2050 |
| Side-haunch rib (exterior) | .1620 |
| Crown and side plate  | .1215 |

2.1.2 Test Site Description

The test site was located on state route 260, adjacent Duck Creek in Noble County, Ohio. In-situ soil consisted of a brown silty, sandy-clay mixture to a depth of about 8 feet. Table 2.2 lists the physical characteristics of the in-situ soil at various depths.
Data provided in Table 2.2 was obtained from the Ohio Department Of Transportation Division Of Highways - Testing Division. The test-structure site is located in the highly dissected portion of the Allegheny plateau region, on the narrow flood plain of East Fork Duck Creek and over a branch of the same, in an area where moderately deep valley and alluvial deposits overlie shale bedrock of Pennsylvanian Age [8].

2.2 Strain Measurement

Prior to field instrumentation, it was assumed that the corrugated reinforcing-rib would result in a uniaxial stress-strain field with deformations occurring in the
direction of its longitudinal axis. Deformations transverse to this axis were assumed to be minimal and were not monitored with gages. Thus, uniaxial strain gages used to monitor strains occurring in the ribs and they were aligned with the longitudinal axis of the rib. Both the interior and exterior ribs were instrumented.

The structural plate shell represents a biaxial stress field and requires the use of biaxial gages to accurately determine plate stresses. The biaxial gages selected for instrumentation had 90° arms and these arms were aligned with the transverse and longitudinal axes of the culvert.

The cross section chosen for instrumentation was located at the exact center of the culvert length, 20 feet from either end. Seven locations in this cross section were instrumented with gages (see Fig.(2.3)). All electric strain gages were placed at the extreme fibers of the ribs and plate and were paired so that moments and thrusts could be determined with greater accuracy.

In addition to the uniaxial and biaxial gages, rosettes were placed in two different locations around bolts connecting the ribs to the plate. One of the locations was near the northern foundation at the connection of the outside rib to the plate. The other was near the northern
haunch region at the connection of the interior rib to the plate. The corrugated ribs are connected to the plate by a series of 3/4" high strength steel bolts spaced at approximately two foot intervals along the culvert periphery. Rosettes allow the determination of principal stresses and give an indication of shear flow. By placing them on different arms around the bolts it was hoped to gain an understanding of the nature of shear transfer occurring between the rib and plate. Fig.(2.4) shows the orientation of the rosettes around the bolts in both locations.

![Diagram of instrumented bolts and rosettes around the bolts in both locations.](image)

**Fig. 2.3** Lane Culvert Instrumentation Schematic
Figure 2.4 Rosette Orientation

Side Section At North Foundation

Top View

Instrumented Bolt

Rosettes

Side View

North Foundation

Crown Section

Exterior Rib

Side View

Interior Rib

To North Foundation

Top View
2.3 Electric Strain Gage Mounting Technique

Strain gages provide an accurate means of determining strain at a point in a material, however, due to the very small deformations involved, a very delicate means of placement is required. In addition, strain gages must be able to perform in prevailing conditions. Undesirable factors such as temperature, humidity changes and vibrations affect strain gage performance and it is very important that the proper gage type be selected for the conditions it must withstand.

All gages selected for instrumentation were a general purpose strain gage with a constantan grid completely encapsulated polyamide and with large, integral, copper-coated terminals. These gages offer safe operation in a temperature range of -100° to 400° Fareinhite and are accurate to strain levels of ± 1500 micro-strain. Gages providing 350 ohms of resistance were selected because long lead wires used (over 50 feet).

Precise strain measurements are dependent on the type of adhesive used and the gage placement procedure. The adhesive serves the function of transmitting strain from the test specimen to the strain-measuring gage without distortion. Thus, to ensure a good bond between the strain
gage and the test specimen, it is essential that the proper adhesive be used. The selection of the adhesive to be used is dependent on such factors as the carrier material, the operating temperature, the curing temperature and the maximum strain to be measured [9]. The adhesive chosen for bonding gages to the test-structure was cyanoacrylate cement. Although no catalyst was required to induce polymerization, one was used to increase reaction time.

Before strain gages could be mounted to the culvert, the surface had to be properly prepared. First the surface was sanded with a coarse sand paper to remove any rust or rough spots. Then the surface was sanded with a finer paper to ensure a smooth surface for bonding. Next solvents were applied to remove any grease or oil from the surface. And finally, the surface was treated with a basic solution to create desirable chemical conditions for the adhesive.

After the surface had been properly prepared, the location of the gage to be placed was marked and the gage was positioned in the correct orientation using transparent tape. Standard bonding techniques were then employed. If a good bond was ensured (as verified by removing the tape), the gage was checked for the rated 350 ohm resistance. Once the gage resistance was verified, the lead wires were placed at the base of the gage upon a strip of electrical tape that
acted as a barrier to the metal surface to prevent a short circuit from occurring. The wires were then bonded to the culvert surface with Duco-Cement to prevent gage damage in the event that a lead wire was pulled.

2.4 Protection Of Gages From Environmental Conditions

Strain gages placed on the interior of the culvert required protection from water, while gages placed on the exterior surface required protection from impact as well as water. A special coating (M-COAT D) was used to waterproof the gages. Then masking tape was placed over the tabs of the gage to further protect the soldered connection. To protect the gages from shock due to backfill or floating debris, a tar-based material and a then a rubber pad was placed over them. Finally, aluminum foil tape was placed over the gages to secure them and provide further protection and then an air-drying nitrile rubber coating was applied around the edges of the aluminum to prevent water penetration.

2.5 Vibrating Wire Gage Selection And Mounting Technique

Electric strain gages provide an accurate means of determining strain at a point on a specimen. In contrast, average strains over a prescribed length may be determined
with vibrating wires. A vibrating wire gage is basically a wire with a prescribed tension and naturally occurring frequency. When the wire is fixed between two points on the specimen, the variation of this distance can be measured from changes in the natural frequency of vibration in the wire.

The wire is set into vibration by an electromagnet which picks up and transmits the vibration to a frequency measuring apparatus. Characteristics of vibrating wires are long-term reliability and high accuracy. Common accuracies for wires about 20 centimeters long are within 5 microstrain. Accuracy can be improved by decreasing the length of the wire.

A gage length of 2" was selected for instrumentation. In most cases, the wires were spot welded to the culvert. However, in some cases the curvature of the culvert corrugations would not permit a secure spot weld. Thus, an epoxy cement was used to secure gages when a spot weld could not be obtained. A vibrating wire, spot welded to the structure is shown in Fig.(2.5).

2.6 Deflection Measurements

Deflections were obtained using a tape extensometer
which permitted the measurement of relative distances. Accuracy was limited to .03". This instrument is shown in Fig. (2.6).

Eleven reference points were established around the culvert periphery by drilling holes through the structural steel plate and fastening eye-bolts to it. Measurements were made with reference to two points located in the stream bed. These point were established by digging three foot holes and pouring a quick setting concrete mix into them. Then five foot reinforcing rods were driven down into the concrete to further prevent any movement of the reference points. Finally, eye-bolts secured to 4 inches of reinforcing rod were set in the concrete at these two locations. Reference point spacing along the culvert periphery was about two feet. The deflection measurement circuit is shown in Fig. (2.7)

Fig. 2.5 Vibrating Wire Fig. 2.6 Tape Extensometer
Figure 2.7  Deflection Measurement Circuit

Section C

Note: Eye bolts are located one section over from instrumented cross section.

Reference Points

Quick Crete
2.7 Soil Displacements

Displacement of the surrounding backfill was monitored with rod extensometers which were positioned horizontally in the backfill mass at the sides and above the crown of the culvert. Rod extensometers are essentially inductance strain meters consisting of a coil with a ferromagnetic core, the inductance of the core being variable with changes in the flux of the magnetic circuit.

Four rod extensometers were used to monitor soil displacements. The exact locations of the rods is shown in Fig.(2.8). A photograph of a rod extensometer is shown in Fig.(2.8.1). Cables from the rods were pulled through holes drilled in the culvert plate to permit readings to be taken.

2.8 Vertical Control

In an effort to verify that the reference points established in the stream bed were not settling or bulging upward, a survey circuit was established. Five control points were established and elevations were measured with a level. The survey circuit layout is shown in Fig.(2.9). Relative elevations that occurred during dead and live load are reported in Table 4.7.
Fig. 2.8 Rod Extensometer Locations

Fig. 2.8.1 Rod Extensometer
Figure 2.9 Survey Circuit Layout
A total of 130 strain measurements were provided by electric strain gages. Nineteen of the 130 were uniaxial gages, and 28 were biaxial gages. The remaining gages were rosettes. Uniaxial gages were placed on the stiffening ribs while biaxial gages were placed on the culvert plate. Rosettes were placed around two bolts (one at the haunch and the other at the foundation) to provide a measure of the shear transfer that occurred between the ribs and plate.

Nine vibrating wires were placed in the instrumented cross section. Five were located on the interior of the structure. The other four were located on the exterior rib. These gages were used as a backup in the event of electric strain gage damage.

A total of eleven reference points were established to permit the determination of culvert deflections during backfill and live load. Four rod extensometers were placed in the backfill to determine horizontal displacements. A survey circuit was established to monitor any movements of the reference points in the stream bed.
3.1 Introduction

Data representing strains and deflections were recorded in two phases during the loading of the Lane Culvert. The first phase was while backfill material was placed according to ODOT specifications. The second phase was during the application of live load at five selected positions on the pavement above the test-structure. The sections that follow describe the methods used to record data and the manner in which the loads were applied to the culvert.

3.2 Construction Sequence

Construction of the Lane Metals culvert began with the removal of the old existing bridge and roadway. A 10 feet deep by 40 feet wide trench, about 50 feet in length, was then excavated. Concrete strip footings for the culvert were constructed by erecting forms 3 feet deep and 3 feet wide, and then pouring concrete into the forms. Once the concrete had set, the forms were removed and insitu soil was
placed around the footers and compacted with a hand operated whacker.

The culvert was then brought to the construction site and erected by Lane Metals own construction crew. The culvert was placed in groves on the footers and secured to them by pouring concrete into the groves (grouting). All bolts were torqued to a minimum of 150 ft-lbs. At this point, instrumentation of the culvert began while the headwalls of the culvert were constructed.

Forms for the east and west headwalls were erected beginning with the east headwall first. Concrete was poured into them and allowed to set for about 3 days. Then the forms were stripped, and in situ soil was compacted around the headwalls with a whacker. Placement of the backfill material around the culvert then began.

Backfill was placed in approximately 6" lifts, was wetted, compacted and checked for density and moisture requirements. When the backfill reached a level about 20" over the culvert crown, a layer of limestone subgrade 8" thick was placed, wetted and compacted. One week after the completion of backfilling the culvert, 12" of asphalt pavement was placed over the limestone subgrade. The amount of time spent on various phases of the construction sequence
and instrumentation is shown in Fig. (3.1a).

Fig. 3.1a  Time Spent On Various Phases Of Construction Sequence

3.2.1 Placement Of Backfill Material

The backfill material used to bury the culvert was a standard # 603 fine sand. The backfill was placed in six inch lifts, alternately, on both sides of the conduit. Each lift was wetted with water from the existing stream bed and compacted with hand-operated vibratory plate compactors. In the early stages of backfill placement, each lift was
monitored with a Troxler nuclear density gauge to assure compaction at the desired 95% of the Standard Proctor Density. In the later stages of backfill placement, alternate lifts were monitored. Table 3.1 lists the values of density and moisture content of each lift monitored.

### Table 3.1 Compaction Data Taken During Backfill Placement.

<table>
<thead>
<tr>
<th>Backfill height (inches)</th>
<th>Dry density pcf</th>
<th>Wet density pcf</th>
<th>Moisture content %</th>
</tr>
</thead>
<tbody>
<tr>
<td>north footer</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.5</td>
<td>126.6</td>
<td>112.8</td>
<td>12.2</td>
</tr>
<tr>
<td>3.5</td>
<td>121.4</td>
<td>113.1</td>
<td>11.8</td>
</tr>
<tr>
<td>4.5</td>
<td>127.2</td>
<td>111.7</td>
<td>13.9</td>
</tr>
<tr>
<td>south footer</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td>127.6</td>
<td>112.3</td>
<td>13.6</td>
</tr>
<tr>
<td>4.0</td>
<td>125.4</td>
<td>111.7</td>
<td>12.3</td>
</tr>
<tr>
<td>5.0</td>
<td>127.3</td>
<td>113.0</td>
<td>12.6</td>
</tr>
</tbody>
</table>

Note: All data was taken with a Troxler nuclear gauge and passed ODOT inspection.

### 3.3 Deflections And Strains During Backfill

Before the first lift of backfill was placed, initial values of strains were recorded and the shape of the free-standing structure was measured. Table 3.2 lists the coordinates of the eleven reference points as measured with a tape extensometer. Fig.(3.1) is a plot of the measured
Strains were monitored with an HP 3497A model data acquisition system in conjunction with an HP personal computer. Deflections were determined from field measurements that were obtained with a tape extensometer and were accurate within .03". Initial values of strains and deflections served as a reference for strains and deflections occurring during backfill placement.

FIG. 3.1 DESIGN SHAPE VERSUS MEASURED SHAPE
Strains and deflections were recorded for each lift of backfill material placed. Five readings per strain gage were recorded and were represented as an average value for the load increment. Two days were needed to complete the backfill process which included the placement of an 8" layer of limestone subgrade. At the onset of the second day, new initial values of strains and deflections were recorded. This permitted the calculation of strains and deflections that may have occurred due to relaxation of the test-structure overnight.

Table 3.2 Measured Shape Of Test Structure

<table>
<thead>
<tr>
<th>Reference Point</th>
<th>Coordinates</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
</tr>
<tr>
<td>A</td>
<td>0.000</td>
</tr>
<tr>
<td>B</td>
<td>95.495</td>
</tr>
<tr>
<td>1</td>
<td>-46.774</td>
</tr>
<tr>
<td>2</td>
<td>-40.555</td>
</tr>
<tr>
<td>3</td>
<td>-23.146</td>
</tr>
<tr>
<td>4</td>
<td>-0.128</td>
</tr>
<tr>
<td>5</td>
<td>23.724</td>
</tr>
<tr>
<td>6</td>
<td>47.857</td>
</tr>
<tr>
<td>7</td>
<td>71.727</td>
</tr>
<tr>
<td>8</td>
<td>95.483</td>
</tr>
<tr>
<td>9</td>
<td>118.259</td>
</tr>
<tr>
<td>10</td>
<td>135.337</td>
</tr>
<tr>
<td>11</td>
<td>141.550</td>
</tr>
</tbody>
</table>

Note: Bracketed distances are to the neutral axis.

3.4 Soil Displacements During Backfill
Soil displacements were monitored with four rod extensometers buried in the backfill material at selected positions. Two of the rod locations were 10" above the north and south foundations in the plane of the instrumented cross section. The remaining two rods were placed 6-8" above the crown of the culvert in the instrumented cross section. Rod locations are shown in Fig.(2.9). Soil displacements were monitored manually with a portable indicator box after each lift was placed.

3.5 Placement Of Asphalt

Three days after backfill completion, asphalt material was placed according to ODOT specifications. Pavement surface treatment included the placement of three layers of asphalt. The first two layers consisted of 4 inches of 3/4"-3/8" nominal size aggregate. The final layer consisted of 4 inches 1/2" nominal size aggregate.

Asphalt was placed with a paver that covered the width of one traffic lane. When the first layer of asphalt had been placed, strains, deflections and soil displacements were monitored. After the data had been recorded, a roller was driven across the freshly placed surface.

When sufficient time had elapsed for the surface to
cool, the next layer of asphalt was placed. However, data reflecting strains was not available due to a problem that developed with the voltage supply source to the strain gages.

3.6 Application Of Live Load

When ample time for the asphalt surface to harden had elapsed (9 days), a series of five live load positions were located and marked on the pavement surface. The load positions were chosen so that critical moments and thrusts would develop in the crown and haunch regions upon the application of live load to the test-structure.

The load positions were measured using a surveyor's tape with reference to the culvert headwalls. First the transverse centerline was determined and marked on each headwall. Then the longitudinal centerline was marked across the pavement surface. The exact center along the longitudinal centerline marked load position #1. The remaining load positions were marked along the longitudinal centerline out of the instrumented cross section. See Fig.(3.2) for the exact locations of the load positions.

Live load was applied to the culvert by means of a dump truck loaded with crushed limestone. The truck was loaded
and weighed at a nearby quarry to produce the specified live load on the 8-wheel rear tandem axle. A series of three live loads were applied to the test-structure and were 16, 32, and 42 kips respectively. The 32 kip live load was intended to simulate the standard AASHTO HS20 live load.

FIG. 3.2 LIVE LOAD POSITIONS

- Asphalt Surface
- Critical Position
- Berm

EAST HEADWALL

WEST HEADWALL
The loaded truck was positioned over all five load positions and the tire on the central axle were deflated for each live load. The removal of air from the tires on the center axle was intended to produce a more effective "point load" at each load position. Strains, deflections and soil displacements were recorded for all three live loads at the five load positions. Fig.(3.3) shows the loaded dump truck with a 32 kip load at load position #1.

Fig. 3.3 Loaded Dump Truck At Load Position #1

3.7 Backfill Samples

Samples of the #603 sand used to bury the test-structure were collected during the construction sequence for laboratory testing purposes. The samples were wrapped in
several plastic bags to maintain their field moisture content and were taken to the lab for testing. Several multiaxial tests are to be performed to determine material parameters for incorporation into the CANDE finite element computer program. The results of the laboratory testing are not available to date and have not been included in this investigation.

3.8 Summary

Strains and deflections were monitored during the placement of #603 backfill material and during the application of 16, 32 and 42 kip live loads. During backfill, each lift was checked with a Troxler nuclear gauge to assure compaction at the proper 95% of the Standard Procter Density. The asphalt surface was then placed and when it cooled and was hard enough for live load application, five load positions were located and marked. Live load was applied with a dump truck having two four wheel rear tandem axles. The air was removed from the central axle so that the measured live load was applied to the pavement by the four wheel rear tandem axle. The 16, 32 and 42 kip live loads were obtained at a nearby quarry using crushed limestone.
Chapter 4

FIELD TESTING RESULTS

4.1 Introduction

In this chapter results from field tests are presented and discussed. Strains and deflections recorded during backfill and the application of live load to the test-structure were analyzed. Stresses were computed from strains and the results were used to compute moments and thrusts at seven locations in the instrumented cross section. Fully composite action was assumed to occur between the corrugated rib stiffeners and plate. Principal stresses were determined from rosettes placed around two bolts connecting the rib stiffeners to the culvert plate. One of the bolt locations was in the haunch region and the other was near the foundation. Shear transfer that occurred between the ribs and plate was investigated. Deflection components were projected on axes normal to the culvert at reference points located around the culvert periphery.

4.2 Calculation Of Stresses From Strains
The incorporation of corrugated rib stiffeners, connected to the corrugated shell at 24" spacings along the culvert length, reduces deflections but increases stresses due to bending [2]. It was assumed that the corrugated ribs would result in one-dimensional stress fields under load. Thus, strain gages placed on the ribs were aligned with the ribs longitudinal axis and strains transverse to this axis were not monitored with gages (with the exception of one gage located on the exterior rib at the crown).

During the analysis of field results, it was discovered that strains in the longitudinal direction were significant. Biaxial gages placed on the inside plate at the crown of the instrumented cross section and indicated that longitudinal strains (in the direction of the culverts longitudinal axis) were significant. Thus, it is possible that the ribs and plate were flattening (see Fig. (4.1)). The effects of rib and plate flattening were investigated and are discussed in section 4.15.

The corrugated plate between rib spacings most nearly results in a two-dimensional stress field under load. A plane stress condition was assumed for the plate. Stresses on the surface and throughout the plate thickness were assumed to be constant or zero.
All stress calculations were based on Hooke's Law which is valid for linear-elastic deformations of an isotropic, homogenous material. The appropriate forms of the constitutive relations become:

- for uniaxial gages located on the ribs,

\[ \sigma_t = E \epsilon_t \]  \hspace{1cm} \text{Eq. (4.1)}

- for biaxial gages located on the plate,

\[ \sigma_t = \frac{E}{1 - \nu^2} \left( \epsilon_t + \nu \epsilon_1 \right) \]  \hspace{1cm} \text{Eq. (4.2)}

\[ \sigma_1 = \frac{E}{1 - \nu^2} \left( \epsilon_1 + \nu \epsilon_t \right) \]  \hspace{1cm} \text{Eq. (4.3)}

where, \( E = " \text{Modulus Of Elasticity} \" = 30 \times 10^6 \)

\( \nu = " \text{Poisson's Ratio} \" = .30 \)

\( \sigma_t \) = transverse stress
\( \varepsilon_t = \text{transverse strain} \)
\( \sigma_1 = \text{longitudinal stress} \)
\( \varepsilon_1 = \text{longitudinal strain} \)

Note: Effects Of Rib Flattening Are To Reduce Distance To Neutral Axis And Reduce Stresses Due To Bending

**Fig. 4.1 Flattening Of Corrugated Rib Stiffener**

### 4.3 Calculation Of Moments And Thrusts

Moments and thrusts were determined from the simultaneous solution of two equations representing states of stress at points in the cross section. A total of eight equations were available for the crown sections in the event that all the gages were working properly. Six to seven
equations were available for the haunch and sides sections provided all the gages were operating. Whenever possible, stresses from gages located the same distance from the longitudinal centroid were used in computations. In all cases, stresses from gages on opposite sides of the transverse neutral axis were used in the simultaneous solution of moments and thrusts (refer to Fig. (4.2)). Average values of moments and thrusts for each of the seven sections were then determined for each load condition.

Stresses were assumed to be due to bending moments and axial forces (stresses due to shearing forces were not included in the analyses). Superposition of these components yields the familiar expression from mechanics of materials shown below:

\[ \sigma_x = \pm \frac{M_z y}{I_z} + \frac{P}{A} \quad \text{Eq. (4.4)} \]

where, \( I_z \) = moment of inertia, \( M_z \) = moment, \( P \) = thrust
\( A \) = cross sectional area, \( \sigma_x \) = total stress.

Equation (4.4) was used exclusively in moment and thrust calculations for the haunch and side regions due to the symmetrical nature of those sections. However, in the
crown region, sections were asymmetrical due to the thicker stiffening rib on the interior as compared with the exterior rib. Thus, the generalized form of the flexure formula was combined with the axial stress to compute moments and thrusts in the crown regions. The resulting expression is shown below:

\[
\sigma_x = \pm \frac{(M_y I_z + M_z I_{yz})z - (M_y I_{yz} + M_z I_y)y}{I_y I_z - I_{yz}} + \frac{P}{A}
\]

Eq. (4.5)

where, \( I_z \) = moment of inertia about the weak axis
\( I_y \) = moment of inertia about the strong axis
\( I_{yz} \) = product of inertia
\( y, z \) = distances from the neutral axes to the point under investigation
\( A \) = cross sectional area
\( M_y \) = moment about the weak axis
\( M_z \) = moment about the strong axis

In Eq. (4.5), \( M_y \) is equated to zero because longitudinal bending is minimized due to the "bellows" type action of the pipe wall circumferential corrugation [10].
The orientation of the principal axes is given by the following expression:

$$\tan(\beta) = \frac{2 I_{yz}}{(I_Y - I_Z)}$$  \hspace{1cm} \text{Eq. (4.6)}$$

and the measured distances (y and z) to the points of known stress were transformed to this new coordinate system using a standard coordinate transformation.

The sign convention adopted for bending was positive for compressive stresses above the neutral axis and negative for compressive stresses below the neutral axis. Fully composite action was assumed in moment and thrust calculations which is to say that the ribs and plate acted together as one member. Verification of this assumption is given in section 4.12. Values of the geometric properties I, A and S were obtained from [11] for the ribs and plate separately and these values were combined to yield properties for the entire section. Table 4.1 lists values of the geometric properties for the crown, side and haunch regions. A sample calculation of moment and thrust is provided in section A of the appendix.
4.4 Calculation Of Deflections

Vertical and horizontal deflections were obtained by triangulation between reference points located on the interior of the test structure for subsequent load conditions. The difference in the measured geometry for each load condition represents the deflection of the structure due to the imposed load. The components of deflection $(x, y)$ were projected on an axis normal to the culvert at the reference points. The appropriate angles used in this transformation were obtained graphically from the culvert geometry. A sample calculation for a normal deflection is
Table 4.1 Geometric Properties Of Crown And Haunch Sections

<table>
<thead>
<tr>
<th>Section</th>
<th>t in.</th>
<th>A in.²</th>
<th>I in.⁴</th>
<th>S in.³</th>
<th>y in.</th>
<th>z in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plate</td>
<td>.1216</td>
<td>3.690</td>
<td>1.692</td>
<td>1.594</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Outside rib</td>
<td>.1644</td>
<td>1.479</td>
<td>1.425</td>
<td>0.901</td>
<td>2.418</td>
<td>0.0</td>
</tr>
<tr>
<td>Inside rib</td>
<td>.2050</td>
<td>1.845</td>
<td>1.781</td>
<td>1.111</td>
<td>-2.396</td>
<td>-9.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Section</th>
<th>A in.²</th>
<th>I_z in.⁴</th>
<th>I_y in.⁴</th>
<th>I_{zy} in.⁴</th>
<th>y in.</th>
<th>z in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crown</td>
<td>7.014</td>
<td>24.034</td>
<td>814.50</td>
<td>39.785</td>
<td>-.120</td>
<td>-2.37</td>
</tr>
<tr>
<td>Haunch &amp; Side</td>
<td>5.169</td>
<td>9.300</td>
<td>0.0</td>
<td>0.0</td>
<td>0.692</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Note: Sample calculations for the above values are provided in section A of the Appendix. All thicknesses were measured in the field. In the crown sections, the orientation of the principal planes is given \( \beta = 2.87^\circ \).

4.5 Deflections Due To Backfill

As backfill material was placed at the sides of the culvert, the structure bended inward at the sides and slightly upward at the crown. This response is shown in Figures (4.3) and (4.4). The largest transverse deflection occurred at point #1 in Fig.(4.3) and was .237" inward under a backfill height of 55 inches.
As backfill material was placed over the crown of the culvert, these effects were reversed; the sides bended outward and the crown deflected downward. The maximum vertical deflection occurred at point #6 in Fig.(4.4) and was .730" downward under 87 inches of backfill. This backfill level represents 19" of cover above the crown.

From Fig.(4.4) it is evident that the crown regions experienced small upward deflections during the early stages of backfill placement. This is most likely the result of the greater stiffness of the crown region as compared to the haunch and side regions ($I_C=2.5\ I_h$) due to the incorporation of the corrugated rib stiffener on the interior of the crown region. Analyzing the deflections that occurred during backfill placement, one would expect small negative moments to develop in the crown region when the backfill height is below the crown level (61") and large positive moments to develop when the backfill height reaches levels above the crown. In the section that follows, it is shown that this is in fact the case. Table 4.2 lists the values of deflections that occurred during backfill placement.

4.6 Moments Due To Backfill

During the early stages of backfill placement, negative moments developed in the crown region and positive moments
developed in the haunch and side regions. When backfill was placed over the crown, the effects were reversed in the crown and haunch regions, however, negative moments did not develop at both of the side regions (near the foundations) until 4.5" of asphalt was placed. The signs of the moments that developed during backfill are in accordance with the deflections that occurred in most cases (see Figures 4.19).
Table 4.2: Deflections Due To Backfill

<table>
<thead>
<tr>
<th>Reference point</th>
<th>X component</th>
<th>32</th>
<th>49</th>
<th>55</th>
<th>67</th>
<th>71</th>
<th>87</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Y component</td>
<td>0.089</td>
<td>0.163</td>
<td>0.237</td>
<td>0.168</td>
<td>0.082</td>
<td>0.057</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.121</td>
<td>0.411</td>
<td>0.479</td>
<td>0.088</td>
<td>-0.289</td>
<td>-0.398</td>
</tr>
<tr>
<td>2</td>
<td>X component</td>
<td>0.060</td>
<td>0.086</td>
<td>0.169</td>
<td>0.117</td>
<td>0.035</td>
<td>-0.015</td>
</tr>
<tr>
<td></td>
<td>Y component</td>
<td>-0.014</td>
<td>0.053</td>
<td>0.100</td>
<td>-0.162</td>
<td>-0.394</td>
<td>-0.583</td>
</tr>
<tr>
<td>3</td>
<td>X component</td>
<td>0.064</td>
<td>0.059</td>
<td>0.089</td>
<td>0.126</td>
<td>0.036</td>
<td>0.031</td>
</tr>
<tr>
<td></td>
<td>Y component</td>
<td>0.032</td>
<td>0.026</td>
<td>-0.005</td>
<td>-0.207</td>
<td>-0.406</td>
<td>-0.548</td>
</tr>
<tr>
<td>4</td>
<td>X component</td>
<td>0.040</td>
<td>0.050</td>
<td>0.064</td>
<td>0.083</td>
<td>0.018</td>
<td>0.010</td>
</tr>
<tr>
<td></td>
<td>Y component</td>
<td>0.012</td>
<td>0.012</td>
<td>-0.033</td>
<td>-0.299</td>
<td>-0.433</td>
<td>-0.659</td>
</tr>
<tr>
<td>5</td>
<td>X component</td>
<td>0.030</td>
<td>0.053</td>
<td>0.052</td>
<td>0.064</td>
<td>0.052</td>
<td>0.052</td>
</tr>
<tr>
<td></td>
<td>Y component</td>
<td>-0.005</td>
<td>-0.012</td>
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Moments are plotted against the unfolded length of the culvert for two backfill conditions that produced maximum effects in Fig. (4.5). Maximum positive moments developed in the side regions when the backfill level reached 67". They
were .928 and .665 (ft-kips/ft) at points 1 and 7 respectively (see Fig.(4.5)). A maximum positive moment of 2.81 (ft-kips/ft) developed at the crown under a backfill level of 92". Moments that occurred during backfill are listed in Table 4.3.

**FIGURE 4.4 VERTICAL DEFLECTIONS AT CROWN DUE TO BACKFILL**

![Figure 4.4](image)

<table>
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<th>BACKFILL HEIGHT (INCHES)</th>
<th>VERTICAL DEFLECTION (INCHES)</th>
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<td>50</td>
<td>-0.45</td>
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<td>60</td>
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<td>70</td>
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<tr>
<td>90</td>
<td>-0.85</td>
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</table>

**Legend**
- ■ POINT 4
- □ POINT 5
- ● POINT 6
- ○ POINT 7
- △ POINT 8

4.7 Thrusts Due To Backfill

Thrusts that occurred during backfill were not very uniform along the culvert periphery. In some instances tensile thrusts developed. At points where the moment of inertia of the cross section changed, thrusts were
significantly large. Table 4.3 lists the thrusts that occurred during backfill.

Sand is a frictional material and it has been shown that thrust distributions become more uniform with decreased friction [13]. This may explain the non-uniform thrust distribution that occurred during backfill. In addition, although backfill levels are reported as a single value, these values represent the average of backfill levels on opposing sides of the culvert. Furthermore, lifts were placed alternately on either side of the culvert and in some cases they exceeded the 6 inch requirement. This may explain why tensile thrusts developed (refer to Fig.(4.6)). Fig. (4.7) shows the thrust distribution that occurred under the maximum dead load (87" of backfill plus 4.5" of asphalt).

![Diagram of Deflected Structure and Thrust Distribution](image.png)

**Fig. 4.6 Tensile Thrust Development**
FIGURE 4.5  MOMENTS DUE TO BACKFILL

FIGURE 4.7  THRUSTS DUE TO BACKFILL
Table 4.3 Moments And Thrusts Due To Backfill

<table>
<thead>
<tr>
<th>Backfill level (inches)</th>
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<th>4</th>
<th>5</th>
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<th>7</th>
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<td>55 M 0.714</td>
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</table>

Note: *t* represents 87" of backfill plus 4.5" of asphalt.

4.8 Deflections Due To Live Load

The deflections that occurred during the application of live load to the test-structure were much smaller than deflections that occurred during backfill (by a factor of 1/10 in most cases). This effect was expected since the backfill material was of good quality and was properly compacted around the culvert. Live load position #3 caused maximum deflections in the culvert. The variation of
deflections with respect to live load is shown in Fig. (4.8) for live load position #3. From this graph it is evident that deflections increased with increasing live load. The variation of deflections with respect to live load position is shown in Fig. (4.9) for a 42 kip live load. From this graph it is evident that load position #3 caused maximum deflections at the crown and load position #2 caused minimum deflections. Table 4.4 lists the deflections that occurred during live load. Fig. (4.10) shows the deflection the structure experienced under a 42 kip live load at load position #3.

**FIGURE 4.8 NORMAL DEFLECTIONS DUE TO LIVE LOAD AT POSITION NO. 3**

![Graph showing deflections due to live load](image)

**Legend**
- ■ 16 KIP
- □ 32 KIP
- ● 42 KIP
Fig. 4.10 Deflected Shape Due To 42 Kip Live Load At Position #3

Note: Moment values are boxed. Units are in Ft-Kips/Ft. Thrusts values at footers are shown by arrows. Units are in Kips/Ft.
Table 4.4 Deflections During Live Load

16 Kip Live Load

<table>
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<th>Deflection Component</th>
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<th>Position 3</th>
<th>Position 4</th>
<th>Position 5</th>
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Table 4.4  Continued...

32 Kip Live Load

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Moments Due To Live Load

Bending moments vary significantly with cover depth. At greater cover depths, the moments induced by a given vehicle load are smaller [1]. This effect is the result of load distribution that occurs radially. Shallow cover is defined by the following criterion:

\[ \frac{H}{S} \leq 0.25 \]  \hspace{1cm} \text{Eq. (4.7)}

where, \( H \) = depth of backfill cover
\( S \) = culvert span

The \( H/S \) ratio for the test structure is 0.21 for 40" of cover and a 15 foot span thus meeting the criterion for shallow cover. As such, load spreading is probably minimal during live load application and the culvert, being stiffer than the surrounding soil, experiences larger bending moments than would occur at greater depths of cover.

Moments were greatest for live load position #3 which was located directly above the instrumented cross section. Positive moments developed at the crown and near the foundations and negative moments developed at both haunches for all five load positions. Load position #2 caused minimal effects. This load position was located approximately 7 feet
south of the longitudinal centerline. Moments are plotted against the unfolded length of the culvert in Fig.(4.11) for all five load positions experiencing a 42 kip live load. The variation of moments due to a 16,32 and 42 kip live load is shown in Fig.(4.12) for load position #3. A maximum moment of 2.06 (ft-kips/ft) occurred at the crown and was due to a 42 kip live load at load position #3. Table 4.5 lists the values of moments that occurred during the application of live load.

Figure 4.11: Moments with respect to load position for 42 kip LL.
4.10 Thrusts Due To Live Load

The distribution of thrusts during live load were considerably uniform compared to the thrusts that developed during backfill. In addition, there was not much variation in thrust magnitude with respect to load position. Thrusts were largest at the end of the interior rib on the northern side of the culvert (see Fig.(4.13)). This was a point where the area increased from 5.17" in the haunch to 7.01" in the crown. It should be mentioned that in all of the live load applications, the front of the test-vehical was oriented over the northern half of the culvert, except for load position #2 - where the front of the vehical was directly above the culvert centerline.

The nature of this unsymmetrical load application produced thrusts that were larger in the northern half of the conduit. At the southern end of the crown region, tensile thrusts developed which are difficult to explain. However, thrusts occurring at the haunches and sides were very resonable. In all cases, thrusts at the northern foundation were greater than those at the southern foundation and in most instances they were about three times as large.

Thrusts are plotted against the unfolded length of the culvert in Fig.(4.13) for a 42 kip live load placed at all
five load positions. From this figure it can be seen that thrust distributions did not vary much with respect to live load position. Thrusts due to live load are listed in Table 4.5.

The large magnitudes of the thrusts that developed at the northern portion of the crown are most likely the results of unsymmetrical live loading. Three rosettes placed around a bolt on the interior rib at this location allowed principal stresses to be determined there. A study conducted by Duncan [15] indicates that the stiffness of the structure and the surrounding backfill have little effect on the magnitude or the distribution of axial force in the arch. Thus, the possibility of large axial forces developing in the crown due to the increased stiffness there has been ruled out.

FIGURE 4.12 MOMENTS DUE TO LIVE LOAD AT LOAD POSITION 3
### Table 4.5 Moments And Thrusts Due To Live Load

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![Figure 4.13 Thrusts Due to Live Load of 42 Kips](image)

**Legend**
- ■ POSITION 1
- □ POSITION 2
- ● POSITION 3
- ○ POSITION 4
- △ POSITION 5
Table 4.5  Continued...

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4.11 Soil Arching Analysis

Axial forces are important considerations in the design of metal culvert structures because the corrugated metal and bolted seams must be able to withstand compressive forces that may cause buckling and seam compression failure, respectively. A simple procedure for predicting axial forces in culvert structures has been developed by White and Layer [12] and is based on ring compression theory. This theory assumes that the culvert carries the weight of the backfill above the crown through pure ring compression. The maximum axial force at the footing of the culvert is given by:

\[
P = \frac{\lambda H S}{2 \cos(\alpha)} \quad \text{Eq. (4.8)}
\]

where, 
- \( P \) = axial force (kips/ft)
- \( H \) = depth of cover over crown (ft)
- \( S \) = culvert span (ft)
- \( \alpha \) = angle between culvert wall and vertical
- \( \lambda \) = unit weight of backfill (kips/cubic foot)

Axial forces were calculated using Eq. (4.8) for different depths of cover above the crown of the test-structure and were then compared with measured values. The measured values of axial thrust were 2-3 times larger than
those based on ring compression theory.

Similarly, measured axial thrusts that occurred during live load were also greater than the imposed loads. Live loads were converted to equivalent line loads using the following equation obtained from Duncan, Seed and Drawsky [1]:

\[ LL = \frac{AL}{K_4} \]

Eq. (4.9)

where, \( LL \) = equivalent line load (kips/ft)
\( AL \) = axle load (kips)
\( K_4 \) = load spread factor (ft) = 6.4

Axial thrusts that developed during backfill and live load are plotted against thrusts obtained from Equations (4.8) and (4.9) in Fig.(4.14). From this plot it is evident that "negative arching" is taking place during backfill and live load (see Fig.(4.15)). If the total springline thrust 11.0 kips/ft during backfill (north plus south foundation thrusts) is compared to the weight of the soil column above the crown 3.78 kips/ft (for 2 feet of cover) and an equilibrium check is performed, then a total down drag force of 7.2 kips/ft is needed to satisfy equilibrium. Negative arching seems reasonable as the sides of the culvert are inclined inward and the top of the structure is nearly
flat causing the soil to hang itself on the culvert sides. Due to the shallow cover above the crown and the box-shape of the structure it is not likely that a soil compression arch would develop during live load and the load is transmitted directly to the structure.

**Fig. 4.14 Negative Arching Verified**

![Graph showing predicted axial thrust vs. measured axial thrust (kips/ft)]

**Legend**
- Backfill
- Position 1
- Position 2
- Position 3
- Position 4
- Position 5

**Neutral Arching**

**Fig. 4.15**

![Diagram showing negative arching and culvert](image-url)
4.12 Composite Section Verification

Vibrating wire strain gages placed in the crown region at sections 4 and 5 indicate that the structure behaved linear-elastically and compositly. In Fig.(4.16), vibrating wire strain is plotted against the gage distance from the neutral axis for load position #1 subjected to 16, 32, and 42 kip live loads. It is evident from this figure that compressive and tensile strains develop on opposite sides of the neutral axis indicating a large degree of composite action. Vibrating wire #8 placed .12" from the neutral axis experienced very small compressive strains indicating that the neutral axis shifted downward slightly.

Figure 4.16 Composite Section Verification
4.13 Backfill Displacements

During the placement of backfill material around the culvert, four rod extensometers were positioned at different locations (see Fig.(2.9)). Results from field studies indicate that as backfill material was placed at the sides of the culvert, rod extensometers #1 and #2 extended in length. This effect is supported by deflection results of the test-structure which verified that the sides of the culvert deflected inward during the early stages of backfill placement.

Rod extensometers #1 and #2 continued to extend in length as backfill material was placed over the crown of the culvert until there was 6-8" of cover. At this point, rod extensometers #3 and #4 were placed and the backfill process was halted until the next day. Overnight, rod #1 and rod #2 contracted indicating that the sides of the culvert continued to deflect outward and that soil-structure equilibrium was not immediately reached.

When the limestone subgrade material was placed, rod #1 continued to contract, however, rod #2 extended along with rods #3 and #4. As asphalt was placed, rod #1 and rod #2 contracted and rod numbers 3 and 4 extended.
During the application of live load, the rods placed at the sides of the culvert contracted and the rods placed over the crown extended. The order of magnitude of these displacements were 10 times less than those that occurred during backfill in most cases. In addition, live load position did not have much effect on rod displacements.

Horizontal displacements and strains of the backfill embankment are tabulated in Table 4.6. Consistent with the standard geotechnical engineering sign convention, negative values of displacement indicate extension of the soil mass. Fig.(4.17) is a plot of backfill displacement versus backfill height.

Fig. 4.17 Backfill Displacements
4.14 Plate-Rib Shear Transfer

Rosettes placed around a bolt located in the haunch and another one located near the northern footer provided a means of calculating principal stresses and the orientation of the planes upon which they acted. When the principal stresses are plotted on their principal planes, an indication of the shear flow occurring around the instrumented bolt is evident. Several factors contribute to the magnitude and orientation of principal stresses obtained in the two previously mentioned regions. At the foundation, it is expected that moments will not contribute to stresses as much as thrusts (excluding the early stages of backfill placement). However, at the haunch bolt, where moments are considerably larger it is expected that stresses will be primarily due to bending. Thus, it is important to keep in mind where the rosette is placed in relation to the neutral axis of the section and the distance from the bolt.

Principal stresses are plotted in Figures (4.18a) through (4.18d) for two backfill levels and in Figures (4.18e) and (4.18f) for live loads at load position #1. In Figures (4.18g) through (4.18r) the variation of transverse and lateral stresses with the distance from the bolt in both the haunch and footer locations is plotted for 33" of backfill at the culvert stringline, for 40" of cover and for
a 42 Kip live load at position #1. Fig.(4.18s) shows the variation of lateral and transverse stresses directly above and below the bolt at the footer during the course of backfill.

Refering to Fig.(4.18a), it is evident that the plate and bolt came into contact on the bolts upper surface as verified by compressive stresses nearest the bolt on arms 1 and 2 and tension directly beneath the bolt. This effect was most likely due to bending. Compressive lateral stresses occured in regions below the bolt and are probably the result of longitudinal bending moments that resulted from uneven backfill placement. Compressive stresses could develop because as the longitudinal moment attempted to bend the corrugated plate, it would be resisted by the fixed support condition at the footer.

In Fig.(4.18b) there is primarily tension occuring at nearly every location in the region. The larger tensile stresses below the bolt are the result of a less stiff section resisting the same moment. In addition, the change in moments that occurs between the haunch and crown produces large shearing forces in this region, but it is not clear if there was much shear transfer to the interior rib at this stage of backfill placement. Compression directly above the bolt is due to the bolt coming in contact with rib
on its upper surface.

Principal stresses at the footer that resulted from 40" of cover are shown in Fig.(4.18c). Below the bolt tension is due to bending (the plate bends around the rib) and compression is caused by the large axial thrust that developed. Above the bolt there is pure compression again indicating that the plate and bolt are in contact on the upper surface of the bolt. Unlike the case of 33" of backfill at the culvert springline, longitudinal bending stresses are not as significant below the bolt. This is due to more uniform compaction stresses that were obtainable at this point because the backfill level was above the culvert crown and the lifts were not loading the structure unsymmetrically as in the case of 33" of backfill.

In Fig.(4.18d) principal stresses occurring at the haunch region are shown for the case of 40" of cover. Pure tension occurs above the bolt on the interior rib, and at the bolt, both above and below, stresses are very small. Possibly, concave upward bending forced the interior rib to push the bolt into contact with the plate. This explains compressive stresses above and below the bolt in the near vicinity. However, the stresses are considerably small in comparison to the majority of the stresses that developed. It is apparent that there is not much shear transfer
occurring from the plate to the interior rib though this is not to say that large shearing forces do not exist and it could be that most of the shear transfer is occurring between the plate and the outside rib. Compressive stresses that occurred just at the right of the bolt may have resulted from assymetrical bending caused from the non-symmetrical cross section of the crown region.

During live load, the bolt at the footer again was bearing against the plate above it. There is pure compression above the bolt as shown in Fig.(4.18e). Because the rib is stiffer than the plate, the plate bends around the rib causing compressive stresses above the bolt. Tensile stresses that developed below the bolt are the result of this "pinching" type action. Stresses are smaller than those which occurred due to 40" of cover and this indicates that the backfill material is carrying the majority of the load.

The response that the haunch region experienced due to a 42 kip live load is shown in Fig.(4.18f). Below the bolt at the end of the interior rib, there are compressive stresses on horizontal planes that were probably caused from the plate bending around the interior rib. In addition, due to the assymetrical cross section, there may have been a twisting effect which would explain compressive stresses on
vertical planes. Tensile stresses above the bolt imply that the bolt was not in contact on its upper surface. Since the imposed loads would cause compression below the neutral axis, this implies that the bolt is being forced downward into the plate on its lower surface, and compressive stresses below the bolt verify this.

In Figures (4.18g) through (4.18r) it is very clear that the effect of the bolt in nearly every case was to increase the compressive stresses above the bolt. Included on these figures is the magnitude of stress that would have occurred had there been no bolt present. This level of stress was obtained from biaxial gages located at the same elevation but over one corrugation away from the instrumented rib. An increase in the compressive stress was most noticeable on rosette arm #1.

The variation of lateral and transverse stresses during backfill directly above and below the bolt in the footer location is shown in Fig.(4.18s). Transverse and lateral stresses above the bolt ( arm #1 ) increased compressively during backfill. Below the bolt transverse stresses also increased compressively and to an even greater extent. Lateral stresses below the bolt increased in tension though the effect was minimal.

The distribution of stresses along the unfolded length
for 33" of backfill, 40" of cover and a live load of 42 Kips is shown in Figures (4.18t) through (4.18v). Included on these diagrams are transverse stresses just above and below the bolts at the haunch and footer. From these plots it is evident that large tensile stresses are occurring due to the plate bending around the rib.

Fig. 4.18a Principal Stresses At Footer Due To 33" Of Backfill

Free Body Of Section

ARM 1

SCALE

1000 PSI

ARM 2

ARM 3

ARM 4

ARM 5

M = .402 Ft-Kips/ft

T = 2.29 Kips/ft
Fig. 4.18b. Principal Stresses At Hunch Due To 33° Of Backfill

Free Body Of Section

ARM 2

ARM 3

Fig. 4.18c. Principal Stresses At Footer Due To 40° Of Cover

Free Body Of Section

ARM 1

ARM 3

ARM 5

ARM 4
Fig. 4.18d Principal Stresses At Haunch Due To 40° Of Cover

Fig. 4.18e Principal Stresses Due At Footer Due To 42 Kip Live Load At Pos. *1
Fig. 4.18f Principal Stresses At Haunch Due To 42 Kip Live Load At Pos. 3

Fig. 4.18g Lateral Stress Variation With Distance From Bolt
Fig. 4.18h Transverse Stress Variation With Distance From Bolt

![Graph showing transverse stress variation with distance from bolt.](image)

Legend
- ARM 1
- ARM 3
- ARM 5

Fig. 4.18i Lateral Stress Variation With Distance From Bolt

![Graph showing lateral stress variation with distance from bolt.](image)

Legend
- ARM 1
- ARM 2
- ARM 3
- ARM 4
Fig. 4.18j  Transverse Stress Variation With Distance From Bolt

![Graph showing transverse stress variation with distance from bolt.]

Legend:
- ARM 1
- ARM 2
- ARM 3
- ARM 4

33° of backfill / bolt at haunch

Fig. 4.18k  Lateral Stress Variation With Distance From Bolt

![Graph showing lateral stress variation with distance from bolt.]

Legend:
- ARM 1
- ARM 4
- ARM 5

40° of cover / bolt at footer
Fig. 4.18l Transverse Stress Variation With Distance From Bolt

Fig. 4.18m Lateral Stress Variation With Distance From Bolt
Fig. 4.18n Transverse Stress Variation With Distance From Bolt

![Graph showing transverse stress variation with distance from bolt.](image)

Legend:
- ARM 1
- ARM 2
- ARM 3
- ARM 4

40" of cover / bolt at haunch

without bolt

Fig. 4.18o Lateral Stress Variation With Distance From Bolt

![Graph showing lateral stress variation with distance from bolt.](image)

Legend:
- ARM 1
- ARM 2
- ARM 3
- ARM 4
- ARM 5

42 Kip live load at Pos. #1 / bolt at footer

without bolt
Fig. 4.18s Stress Variation At Bolt During Backfill

![Graph showing stress variation at bolt during backfill.](image)

Legend:
- ARM 1 Trans.
- ARM 1 Lat.
- ARM 5 Trans.
- ARM 5 Lat.

Fig. 4.18t Stresses Due To 33" Of Backfill

![Graph showing stresses due to 33" of backfill.](image)

Legend:
- PLATE
- BOLT
- RIB

Stresses on plate below bolt are tensile due to pinching type action.
Fig. 4.18u Stresses Due To 40" of Cover

Fig. 4.18v Stresses Due To A 42 Kip Live Load
Table 4.6 Backfill Displacements And Strains

<table>
<thead>
<tr>
<th>Load Condition</th>
<th>Rod #</th>
<th>Displacement (inches)</th>
<th>Strain (micro-strain)</th>
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<tr>
<td>Backfill Height</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24&quot;</td>
<td>1</td>
<td>-.002</td>
<td>-15</td>
</tr>
<tr>
<td></td>
<td>2</td>
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<tr>
<td></td>
<td>2</td>
<td>-.012</td>
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<td>Limestone Subgrade</td>
<td></td>
<td></td>
<td></td>
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4.15 Effects Of Rib And Plate Flattening

Several tests conducted on arc-and tangent corrugated profiles by Cary [7], allowed the development of empirically derived equations to ensure local stability of the corrugated plate during inelastic bending. It is not intended to present these equations here but rather to discuss the results of his findings. The ratio of the corrugations radius of curvature to the plate thickness is the controlling factor. As this ratio increases, the flexural strain at which buckling will occur decreases.

If the corrugated rib-stiffeners connected to the culvert are analyzed during the case of live load and the ribs and plate are assumed to experience flattening, several effects are possible:

1) As the corrugated rib flattens, its radius of curvature will increase, its moment of inertia will decrease and bending stresses will increase. In addition, local buckling may occur which would render it a discontinuous member and a less effective factor of safety against buckling of the culvert structure.

2) Stresses due to imposed loads are concentrated in the ribs, especially stresses due to bending. These stresses are largest at or near center span. If a rib were to experience local buckling, the load it carried may be transferred to neighboring ribs in addition to the load they already carry,
and propagation of buckling would occur until all ribs in the immediate area failed. At this point, the plate would carry the entire load and a sudden snap through failure would be possible.

In Fig. (4.18w) strains that occurred on the exterior rib at center span (in both the transverse and lateral directions) during backfill and live load are shown. This plot suggests that transverse strains in the rib influence longitudinal strains in the plate in a nearly one to one relationship. If this is the case, longitudinal stresses are likely to be as large as transverse stresses at the crown and a plane strain approximation in the finite element analysis will be more in error in predicting culvert response as was initially thought. As a result, moments and deflections would be larger than field values.

The degree of composite action can also be seen in Fig. (4.18w). Strains become larger with the distance from the neutral axis. In addition, longitudinal strains in the rib are not very significant and remain fairly constant, thus verifying the assumption of a one-dimensional stress-strain field.
Fig. 4.18w Rib And Plate Strains During Backfill And Live Load

Legend
- RIB (trans.)
- RIB (lat.)
- PLATE (trans.)
- PLATE (lat.)

Transverse strain on rib strongly influences lateral strain on plate.
4.16 Survey Circuit Results

The results of the level circuit indicate that there was virtually no movement of bench marks A, B, C and D. The largest vertical movement was detected at bench mark A and was .009" downward. Table 4.7 lists the elevations of the bench marks with respect BM1 (refer to Fig.(2.9)).

4.16 Summary

In an attempt to gain a better understanding and more clearly visualize the physical response the test-structure underwent during the construction sequence and live load, the deflected shape of the culvert has been exaggerated and included on these diagrams are corresponding moments, thrusts and backfill displacements. The intent is to piece together all of the experimental findings at various stages during backfill and live load on one diagram, and then determine the nature of the structural response. Figures (4.19a) through (4.19d) show the culvert response during three stages of backfill placement and for the case of a 42 kip live load at load position #1.
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Date: 10/6/86  Closure: .003

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Date: 10/10/86  Closure: .000

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In Fig. (4.19a) the deflected shape of the culvert is shown for the case of backfill level with the springline. Note that the backfill level on the northern side is 6" higher than that on southern side. From this diagram it is evident that the culvert shifted laterally toward the southern footer as a result of the uneven backfill placement. At the side and haunch regions, positive moments developed that were consistent with the curvature of the culvert in those regions. Negative moments occurred in the crown region as it deflected upward.

It is interesting to note that moments were larger at the southern side of the culvert. This is consistent and reasonable as the northern haunch is opening outward while the southern haunch is closing inward.
The largest thrust occurred at the northern foundation and this is reasonable as there was more backfill material on this side of the culvert. Displacements of the backfill mass agree with the deflected shape of the structure, however the magnitudes of backfill displacement are 100 times less than culvert deflections in most cases and it is difficult to make a comparison. However, the trend was consistent in that as the culvert sides deflected inward, the rods extended in length and as the culvert sides deflected outward, the rods contracted.

In Fig.(4.19b) the deflected shape of the culvert is shown for the case of a backfill height just level with the crown of the test-structure. In this case the backfill levels are nearly equal with the northern side having approximately 2-3 inches more material. Again the northern haunch is opening up while the southern haunch is closing and, as a result, moments are greater at the southern haunch. The crown of the culvert is moving downward slightly as a rigid member, but maintains a concave downward configuration, and thus experiences negative moments consistent with the standard sign convention ( compressive strains on the inside fibers are the result of negative moments ).
A possible explanation for the rigid-member downward movement of the crown region could be that the increased stiffness there, due to the interior rib stiffener. As backfill levels increased so did the frictional forces at the sides and this downward drag force could have been sufficient to cause a downward deflection of the crown.

The deflected shape of the test-structure with 32" of cover including 4.5" of asphalt is shown in Fig.(4.19c). The culvert crown deflected concave upward and the sides concave inward, except for the point located nearest the northern foundation. Bending moments are consistent with the culvert curvatures in all cases with the maximum moment occurring at the culvert crown.

The small positive moment at the northern footer region may have been the result of large inward deflections that occurred there during the early stages of backfill placement. This curvature may have been so pronounced that a complete reversal was not possible, and thus, resulted in smaller moments.

Structural response due a live load of 42 Kips at position #1 is shown in Fig.(4.19d). The crown deflected downward .119". Due to the non-symmetrical nature of load application, the northern side deflected outward at all
points up to the haunch. However, the southern side did not undergo the same response. Instead, the point nearest the southern footer deflected outward while the point midway between the haunch and footer deflected inward. Moments, again are consistent with the deflected shape of the structure. The southern haunch experienced larger moments than the northern haunch because it was closing up due the unsymmetrical load condition.

Thrusts were largest at the northern footer by a factor of about 3. This was undoubtedly due to the front of the test-vehicle being positioned over the northern half of the culvert.

In all cases of backfill and live load, thrusts were greatest at the northern portion of the crown, near the end of the interior rib. This was most likely due to the unsymmetrical load condition.

Shear stresses on the plate were significantly higher than those on the interior rib at points nearest the bolt. Thus, there was very little shear transfer occurring between the plate and interior rib. Shear forces are directly related to the change in moment i.e. \( V = \frac{dM}{dx} \). Changes in moments occurring between the haunch and crown are appreciably large and there is reason to believe that large
shearing forces would develop at these locations.

No shearing stress term was included in the calculations of moments and thrusts. It is quite possible that the large thrusts determined in this region (as calculated from strain data) include the effects of shear stresses. In addition, because there was very little shear transfer from plate to interior-rib, stresses on the plate were higher than would be had shear transfer occurred. Thus, calculated moments and thrusts are likely to be greater due to the larger differences in stresses on the ribs and plate.
5.1 Introduction

Two finite element programs were used to predict the response of the test-structure during the construction sequence and during the application of live load. One analysis was performed using the FHWA "CANDE" computer program which was introduced in 1976 for the structural design and analysis of buried culverts. The other finite element analysis was done with the finite element program "SEQCON", which was developed for purposes of simulating construction sequences typical of soil-structure interaction problems. The results of each finite element analyses were compared to field test results to determine which solution provided the closest approximation to the actual, experimental culvert response.

In cases where loading was symmetrical, half meshes were used. Full meshes were used for the case of unsymmetrical loading (live load position #2). The Duncan soil model was used to simulate backfill material response.
5.2 CANDE Background

The CANDE finite element program is a soil-structure interaction analysis program. Incorporated into the program are three solution levels: level 1 - an elasticity solution, level 2 - an automated finite element solution, and level 3 - a standard finite element solution. In addition, CANDE offers the choice of several different culvert types including, corrugated steel and aluminum and reinforced concrete as well as plastic materials. The program user has the option of a design or analysis solution. Other features of CANDE include soil-structure interface elements, incremental construction sequences and nonlinear soil models.

CANDE was updated in 1980 to include new options for reinforced concrete box culverts and to include the Duncan soil model and parameters applicable to it. At present, there are five soil models:
- Elastic
- Ortho-Elastic
- Over-Burden Dependent
- Hardin Model
- Duncan Model
and the capability of establishing interface elements between the culvert and soil mass to permit relative movement or "slip" and debonding.

CANDE utilizes a four node quadrilateral soil element that is composed of two triangles with complete quadratic interpolation functions. The interface element available in CANDE permits two subassemblies to meet at a common interface, such that, during the application of load to the soil-structure system the subassemblies may separate, slip or rebond. "Slip" is characterized by the Mohr-Coulomb failure criterion. The culvert structure is modeled by beam elements in a plane strain formulation. Each beam element has two nodes with six degrees of freedom per element (x, y translation and rotation). Fig.(5.1) shows the three element types available in CANDE and the corresponding interpolation functions.

5.2.1 CANDE Finite Element Formulation

Solution levels 2 and 3 share the same finite element solution program but differ in that level 2 generates the input data and level 3 input data is user-defined. Level 3 shows non-standard geometries and complex boundaries to be defined, however, the finite element formulation for both levels is identical.
Fig. 5.1 CANDE Quadrilateral Element And Beam Element

\[ U = U_i E'_i + B_i E'_j E'_k \]
\[ V = V_i E'_i + C_i E'_j E'_k \]

Beam element Interpolation Functions:

\[ u(x) = (1 - x/L)u_1 + (x/L)u_2 \]
\[ v(x) = (1 - \frac{3(x/L)^2}{2} + \frac{2(x/L)^3}{3})v_1 \]
\[ x(1 - x/L)\theta_2 \]
\[ (3(x/L)^2 - 2(x/L)^3)v_2 \]
\[ (x - L)(x/L)^2 \theta_2 \]

\[
\begin{bmatrix}
E_1 \\
E_2 \\
E_3
\end{bmatrix} = \frac{1}{2A} \begin{bmatrix}
A_{23} & b_1 & a_1 \\
A_{31} & b_2 & a_2 \\
A_{12} & b_3 & a_3
\end{bmatrix} \begin{bmatrix}
1 \\
x \\
y
\end{bmatrix}
\]

where, \( A_{ij} = \frac{(X_i Y_j - X_j Y_i)}{2} \)

\[ b = Y_j - Y_k \]
\[ a = X_k - X_j \]
\[ A = \frac{1}{2}X \begin{bmatrix} b & a \end{bmatrix} \]
\[ a = \text{indicial summation from 1 to 3} \]
The static formulation of the finite element method employed in CANDE is derived from virtual work principles and is given below:

The virtual work a structural system experiences can be expressed as the sum of the virtual external work of traction and body forces undergoing virtual displacements that are compatible with the kinematic constraints of the system. When the virtual internal strain energy is equated to the virtual external work, the following expression results for the structural system:

\[
\int_V \delta(\varepsilon)^T \sigma \, dv = \int_S \delta(u)^T \mathbf{t} \, ds + \int_V \delta(u)^T \mathbf{f} \, dv \quad \text{EQ. (5.1)}
\]

where, \( \{\sigma\} = \text{stress vector} \)
\( \{\varepsilon\} = \text{strain vector} \)
\( \{u\} = \text{displacement vector} \)
\( \{\mathbf{t}\} = \text{surface traction vector} \)
\( \{\mathbf{f}\} = \text{body force vector} \)
\( \{\cdot\}^T = \text{transpose of vector} \)
\[ \delta(\cdot) = \text{virtual vector} \]
\[ S = \text{surface of body} \]
\[ V = \text{volume of body} \]

To arrive at the finite element displacement formulation, the strain energy term of Eq.(5.1) is written in terms of displacements using the constitutive and displacement relationships listed below:

\[ \{\sigma\} = [C] \{\epsilon\} \quad \text{Eq.}(5.2) \]
\[ \{\epsilon\} = [Q] \{u\} \quad \text{Eq.}(5.3) \]

where, \([C]\) = constitutive matrix
\([Q]\) = derivative matrix

Eq.(5.1) can now be expressed in the following form:

\[
\int_V \delta(\epsilon)^T \{\sigma\} \, dV = \int_V \{ [Q] \delta(u) \}^T [C] [Q] \{u\} \, dV \quad \text{Eq.}(5.4)
\]

The terms of \([C]\) and \([Q]\) depend on the material and kinematic relations (plane stress, plane strain etc.) of the problem under investigation. The form of the constitutive matrix used in the CANDE analysis is given below:
Element displacements are characterized by a specified interpolation function and are selected as the primary dependent variable with unknown nodal displacements on the element exterior. The form of this relation is given below:

\[
\begin{bmatrix}
\Delta \sigma_x \\
\Delta \sigma_y \\
\Delta \tau
\end{bmatrix}
= \begin{bmatrix}
\frac{E(1-\nu)}{(1+\nu)(1-2\nu)} & \frac{E\nu}{(1+\nu)(1-2\nu)} & 0 \\
\frac{E\nu}{(1+\nu)(1-2\nu)} & \frac{E}{2(1+\nu)} & 0 \\
\text{SYM} & 0 & 0
\end{bmatrix}
\begin{bmatrix}
\Delta \epsilon_x \\
\Delta \epsilon_y \\
\Delta \gamma
\end{bmatrix}
\]  

Eq.(5.4.1)

When Eq. (5.5) is substituted into Eq. (5.1) and an integration is performed over the entire domain \( V \) (represented as the summation of element integrations), the virtual work equation yields the global equilibrium equation:

\[
[K] \{\bar{u}\} = \{P\}
\]  

Eq.(5.6)
where \([K] = \text{the summation of element stiffness matrices}\)

\((P) = \text{the summation of element forces}\)

The element stiffness matrix is the heart of the finite element formulation and is given by:

\[
[K]_e = \int_{V_e} [B]^T_e [C]_e [B]_e \text{d}V_e \quad \text{Eq. (5.7)}
\]

The element load vector is given by:

\[
(P)_e = \int_{S_e} [B]^T_e (t) \text{d}S_e + \int_{V_e} [B]^T_e (f) \text{d}V_e \quad \text{Eq. (5.8)}
\]

where,

\[
[B]_e = [Q]_e [h]_e \quad \text{Eq. (5.9)}
\]

5.2.2 Incremental Form

Many systems are nonlinear and to account for this response CANDE utilizes an incremental approach to approximate nonlinear behavior. The summation of several linear solutions after the load has been applied in small increments provides this means. The solution increment is added to the previous increments so that a running total, representing the complete response, is available after each loading increment. In addition, material nonlinearity is
accounted for by "up-dating" the constitutive matrix. This is accomplished by re-evaluating the material properties in accordance with the selected nonlinear model and the current state of stress (i.e. tangent method). Small load increments must be used for an accurate solution.

5.3 SEQCON Background

SEQCON is a displacement FEM formulation model that simulates construction sequences typical of soil-structure interaction problems. The construction sequences that SEQCON can model are listed below:

- Embankment
- Dewatering (consolidation)
- Deposition or Embankment
- In Situ Stresses
- Tie-backs and other structural components

Material nonlinearity is accounted for by utilizing four material models. The models provided in SEQCON are listed below:

- Linear Elastic
- Hyperbolic Model
- Drucker-Prager
- Cap Model

SEQCON utilizes an eight-node isoparametric quadrilateral soil element and the option of 2 or 3 node structural bar elements. In addition, interface elements may be used. The element types and the corresponding interpolation functions are shown in Fig.(5.2).

Fig. 5.2 SEQCON Quadrilateral And Beam Element
Quadralateral interpolation functions:

\[ X = \sum_{i=1}^{8} N_i X_i \]

\[ Y = \sum_{i=1}^{8} N_i Y_i \]

\[ N = \frac{1}{2} \left( (1-s)(1+s) \right) \]

5.3.1 SEQCON Finite Element Formulation

SEQCON uses the displacement finite element model which was derived in section 5.2.1. However, in contrast to CANDE, SEQCON utilizes an eight node isoparametric element with quadratic variations in displacements. This leads to linear stress distributions within the element, whereas, in CANDE stresses are constant within the element.

5.4 Geologic Material Modeling

Soils are highly nonlinear materials and require extensive laboratory testing to determine material parameters for incorporation into the governing constitutive stress-strain relationship. The FEM can account for nonlinearity in a soil-structure system when it is used in
an incremental and/or iterative manner, however, this is not enough alone to account for material nonlinearities. To account for material nonlinearities, a realistic stress-strain relationship must be utilized. Constitutive models are functions of many variables such as, density, moisture content, stress history, and stress paths. Through numerous laboratory tests it can be determined which variables are significant and which constitutive model should be utilized.

5.4.1 Hyperbolic Stress-Strain Relationship

The hyperbolic stress-strain relationship was developed by Duncan and Chang [13] for use in the finite element analyses of soil deformations where nonlinear behavior is modeled by a series of linear increments. The hyperbolic model offers generality and may be used to represent a wide range of soils (clays, silts, sands and gravels, and rockfills). This model can be used for fully or partially saturated soils, and for drained or undrained conditions. In recent years it has been exclusively used in a variety of soil-structure interaction problems. The constitutive relation is assumed to be governed by the generalized form
of Hooke's Law of elastic deformations. For plane strain conditions the constitutive relation has the following form:

\[
\begin{pmatrix}
\Delta \sigma_x \\
\Delta \sigma_y \\
\Delta \tau_{xy}
\end{pmatrix} = \begin{pmatrix}
\frac{3B}{9B - E} & \frac{3B}{9B - E} & \frac{3B}{9B - E} \\
(3B + E) & (3B - E) & 0 \\
(3B - E) & (3B + E) & 0
\end{pmatrix} \begin{pmatrix}
\Delta \varepsilon_x \\
\Delta \varepsilon_y \\
\Delta \gamma_{xy}
\end{pmatrix}
\]

Eq. (5.10)

where, \( E \) = Young's modulus

\( B \) = Bulk modulus

\( \Delta \) signifies an increment of stress or strain

Stress dependency, material nonlinearity and inelasticity may be modeled by re-evaluating the values of \( B \) and \( E \) after each load increment, consistent with the current state of stress, and "up-dating" the constitutive matrix.

Stress-strain curves for many soils may be approximated by hyperbolas with resonable accuracy as was shown by Kondner [14]. Characteristics which make the use of hyperbolas appealing in modeling stress-strain response of soils is that the parameters that appear in the hyperbolic equation have physical significance and that these parameters may be easily determined. Refering to Fig.(5.3a),
E₁ is the initial tangent modulus of the stress-strain curve and \((σ₁ - σ₃)_{ult}\) is the asymptotic value of the stress difference which is always greater than the compressive strength of the soil. The equation of the hyperbola used to model soil response is given below:

\[
(σ₁ - σ₃) = \frac{ε}{1 + \frac{ε}{E₁ (σ₁ - σ₃)_{ult}}} \quad \text{Eq.}(5.11)
\]

If Eq.(5.11) is normalized by dividing both sides of the equation by \((σ₁ - σ₃)\), and then plotted, this transformation will represent a linear relationship (see Fig. 5.4b). Test data plotted on the transformed plot are curve fitted with the best-fit straight line through them. The slope and intercept of this straight line represent the material parameters of the hyperbolic model.

An increase in confining pressure will result in a steeper stress-strain curve for most soils (excluding fully saturated soils in undrained-unconsolidated conditions) and will also increase the strength of the soil being tested.
Thus, there is a stress dependency that must be accounted
for in the soil model. The stress dependency can be accounted for by relating the material parameters \( E_i \) and \((\sigma_1 - \sigma_3)_{ult}\) to the confining pressure \( \sigma_3 \) using empirical relations. The variation of \( E_i \) with \( \sigma_3 \) is given by the following equation suggested by Janbu [13]:

\[
E_i = K P_a \left( \frac{\sigma_3}{P_a} \right)^n \quad \text{Eq.(5.12)}
\]

where, \( K = \) modulas number (dimensionless)

\( n = \) modulas exponent (dimensionless)

\( P_a = \) atmospheric pressure

The variation of \((\sigma_1 - \sigma_3)_{ult}\) with the confining pressure is accounted for by relating it to the compressive strength at failure, and then relating \((\sigma_1 - \sigma_3)_f\) to \( \sigma_3 \) by using the Mohr-Coulomb criterion i.e:

\[
(\sigma_1 - \sigma_3)_f = R_f (\sigma_1 - \sigma_3)_{ult} \quad \text{Eq.(5.13)}
\]

where, \( R_f = \) failure ratio, usually, \( 0.5 < R_f < 1.0 \)

\[
(\sigma_1 - \sigma_3)_f = \frac{2c \cos \phi + 2\sigma_3 \sin \phi}{1 - \sin \phi} \quad \text{Eq.(5.14)}
\]
where, $c = \text{cohesion intercept}$

$\phi = \text{friction angle}$

An expression for the tangent modulus may be derived by differentiating Eq.(5.11) with respect to and substituting Equations (5.12), (5.13) and (5.14) into the resulting expression for $E_t$. The resulting form of the equation for $E_t$ is given below:

$$E_t = \left[ 1 - \frac{R_f (1 - \sin \phi)(\sigma_1 - \sigma_3)}{2c \cos \phi + 2 \sigma_3 \sin \phi} \right]^2 K P_a \left( \frac{\sigma_3}{P_a} \right)^n$$

Eq.(5.15)

Eq.(5.15) permits the evaluation of stresses after displacements and strains have been determined in the finite element solution.

5.5 Deposition Modeling

Deposition occurs in the field when lifts or layers of backfill material are placed and compacted to some specified density to form the proposed embankment. Lifts have little strength until the compaction procedure is performed and so the main effect of adding lifts is to add surface traction to the layers or lifts beneath it.
Surface traction due to a lift of backfill may be calculated from the following equation:

\[ F = \int \lambda dV_1 \]  \hspace{1cm} \text{Eq. (5.16)}

where, \( \lambda \) = unit weight of soil
\( V_1 \) = volume of lift
\( F \) = surface traction due to lift of backfill

5.6 Live Load Modeling

There are four load distributions common to culvert-soil systems. They are:

- uniform pressure loads
- gravity loads
- strip or line loads
- concentrated loads

The vertical stress at the culvert crown due to a concentrated load is given by the following equation after Boussenesq:

\[ p_Q = \frac{3}{2} \frac{Q}{\pi} \left( \frac{H^3}{L^5} \right) \]  \hspace{1cm} \text{Eq. (5.17)}
where, \( H \) = height of cover over culvert crown
\( L \) = distance from load to culvert crown
\( P_Q \) = vertical stress due to concentrated load

Similarly, the effect of a strip load of line intensity \( q \) at the culvert crown may be described by:

\[
P_Q = \frac{2q}{\pi} \left( \frac{H^3}{L^4} \right)
\]

Eq. (5.18)

The overburden pressure due to gravity loads is expressed by the following relation:

\[
P_g = \lambda H
\]

Eq. (5.19)

where, \( \lambda \) = unit weight of soil

In CANDE and SEQCON concentrated loads must be converted to equivalent strip loads. This is done by equating Eq. (5.17) to Eq. (5.18) and solving for \( q \) to yield:

\[
q_Q = \frac{3}{4} \left( \frac{Q}{L} \right)
\]

Eq. (5.20)
5.7 Assumptions

Simulation of the actual field problem is a very complicated task. The majority of finite element formulations lack the capability of completely describing all of the factors which influence soil-structure interaction behavior. In addition, there is a trade off between the degree of accuracy of a problem solution and economy. Simplification of the problem with a reasonable level of accuracy is the objective. Several assumptions common to both CANDE and SEQCON in the analysis of the Lane culvert are listed below:

1) Plane strain condition
2) Time independency
3) Constant intrinsic material properties
4) Finite displacements
5) Fully bonded condition between culvert and backfill material
6) Lateral stresses due to compaction are ignored

5.8 Results Of Finite Element Analyses

Results from CANDE and SEQCON computer runs are
compared to the field findings. The construction sequence and live load are simulated with both full and half meshes. In the case of symmetric loading, half meshes are used because they employ more elements and provide greater accuracy. In the case of unsymmetrical loading, full meshes are used. A total of three meshes were used. The half meshes used for the CANDE and SEQCON analyses are shown in Figures (5.4.1) and (5.4.2), respectively. A total of 13 construction increments were employed in the simulation of backfill placement in CANDE; twelve were employed in SEQCON. Nonsymmetry was modeled with CANDE for the case of a live load placed at position #2. This mesh is shown in Fig. (5.4.3). A total of 11 construction increments were used to model backfill placement in the full mesh.

In the full mesh analysis, 22 beam elements are used to model the culvert structure. The half mesh analysis performed with CANDE utilized 25 beam elements, while in the SEQCON analysis only 17 were used. Fewer beam elements were used in SEQCON because three node elements offer greater accuracy in modeling non-linear response.

The actual field problem is described by five soil material zones. These materials, their properties and the soil models used to govern their response are listed in Table 5.1 below:
<table>
<thead>
<tr>
<th>Material</th>
<th>Soil Model</th>
<th>Material Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>In-situ soil</td>
<td>Linear elastic</td>
<td>$E = 5000 \text{ psi}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$v = 0.34$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\lambda = 135 \text{ pcf}$</td>
</tr>
<tr>
<td>Concrete footer</td>
<td>Linear elastic</td>
<td>$E = 3 \times 10^6 \text{ psi}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$v = 0.34$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\lambda = 150 \text{ pcf}$</td>
</tr>
<tr>
<td>#603 backfill</td>
<td>Duncan Model</td>
<td>$c = 0.0$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\phi = 33.0$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\Delta\phi = 3.0$</td>
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<td></td>
<td></td>
<td>$n = 0.4$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$R_f = 0.7$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$K_B = 50$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$m = 0.2$</td>
</tr>
<tr>
<td>Limestone subgrade</td>
<td>Duncan Model</td>
<td>$c = 0.0$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\phi = 42.0$</td>
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<tr>
<td></td>
<td></td>
<td>$\Delta\phi = 9.0$</td>
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<td></td>
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<td></td>
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<td></td>
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<td></td>
<td></td>
<td>$K_B = 175$</td>
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<tr>
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<td>$m = 0.2$</td>
</tr>
<tr>
<td>Asphalt</td>
<td>Linear elastic</td>
<td>$E = 3.5 \times 10^5$</td>
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<tr>
<td></td>
<td></td>
<td>$v = 0.34$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\lambda = 150 \text{ pcf}$</td>
</tr>
</tbody>
</table>
In-situ soil was modeled to a depth of 5 feet below the upper surface of the concrete footing. Boundary conditions applied to the nodes at the bottom most portion of this
layer were to fix the x,y deflections as well as the nodal rotations. A trench 10 feet from either side of the culvert and 6.6 feet high was restricted for the #603 backfill material. At the end of this 10 foot zone, 3 additional feet of the in-situ soil was included in the mesh. Boundary conditions applied to the nodal points located on vertical planes at the extremities of the mesh fixed the horizontal deflections and the rotations and permitted vertical deflections. An 8" layer of elements representing the limestone subgrade were placed on top of the last lift of backfill, and then, a 12" layer of elements representing asphalt were placed on top of it. Boundary conditions applied to the upper most surface left it free to deflect and rotate.

5.8.1 Backfill Response

In the CANDE analysis, the construction sequence is simulated by placing the elements representing the in-situ soil first along with elements representing the concrete footers. Then the culvert beam elements are placed on the footers and the node in contact with the footers is restricted from any horizontal displacement in the boundary condition coding. Next the elements representing backfill material are placed in layers until the total height reaches a level of 80". At this point the layer of elements
representing the 8" layer of limestone subgrade are placed and then, the 12" layer of asphalt elements are placed. The construction sequence is then complete for the computer run.

In SEQCON, the entire mesh is recognized at the very beginning of the construction sequence. All of the elements that are to be placed during the backfill process must first be excavated. Then they are placed again, layer by layer, until the backfill process is complete.

Crown moments during backfill are shown in Fig.(5.5.1). From this plot it is clear that both CANDE and SEQCON underestimate crown moments during the early stages of backfill. This most likely resulted from not including compaction pressures in newly added element layers. SEQCON and CANDE came close to predicting crown moments during the later stages of backfill placement with SEQCON coming closer to the actual field values.

Thrusts at the footer are shown in Fig.(5.5.2) where it can be seen that CANDE underestimates thrusts and SEQCON comes close to predicting the actual field values.

Haunch moments that occurred during backfill are shown in Fig.(5.5.3). Again, during the early stages of backfill, the predicted haunch moments were very small compared to
experimental values. But once the backfill reached crown level, the moments increased consistent with experimental findings. CANDE came closest in predicting the actual field moments.

Crown deflections are shown in Fig.(5.5.4). Both SEQCON and CANDE underestimated crown deflections with SEQCON to a lesser degree. CANDE predicts a very sudden drop after the backfill reaches a level of 74" which is most likely the result of not including compaction forces in the analysis. This effect is less severe in SEQCON which is probably due to the greater accuracy obtained with higher order elements.

Moments are plotted against the unfolded length of the culvert in Fig.(5.5.5) for 40" of cover. It is evident from this figure that better agreement is obtained in the haunch and side regions and that the crown moments are slightly overestimated. In Fig.(5.5.6) thrust distributions along the unfolded length are plotted for 40" of cover. There is very little agreement between predicted and field values, however, CANDE and SEQCON predict similar distributions with SEQCON yielding larger magnitudes of thrust. The deflected shape of the structure with the settlement at the footer subtracted from field values of deflections are shown in Fig.(5.5.7). Better agreement between crown deflections is obtained in this way.
Fig. 5.5.1 Crown Moments During Backfill

CANDE and SEQCON underestimate crown moments during the early stages of backfill.

Fig. 5.5.2 Thrusts At Footer During Backfill

SEQCON comes closest to predicting field values.
Fig. 5.5.3 Haunch Moments During Backfill

CANDE and SEQCON both come close to predicting field values.

Legend
- SEQCON
- CANDE
- FIELD

Fig. 5.5.4 Crown Deflections During Backfill

SEQCON comes closest to predicting field crown deflections.

Legend
- SEQCON
- CANDE
- FIELD
Fig. 5.5.5 Moment Distribution Due To 40" Of Cover

Fig. 5.5.6 Thrust Distribution Due To 40" Of Cover
5.8.2 Live Load Response

In Figures (5.5.8) through (5.5.21) the predicted response during live load is compared to field values. These figures may be summarized as follows:

1) CANDE and SEQCON overestimate crown moments and deflections but are in good agreement with haunch moments.
2) SEQCON predicts a uniform moment distribution around the
haunch, whereas CANDE predicts a more rapid increase moments in the top haunch region.

3) Thrust distributions are not close to field values. SEQCON predicts larger thrusts than does CANDE and is closer to field values at the footer.

4) Both CANDE and SEQCON predict large shear forces at the change in cross sectional properties in the crown region (at the ending of the interior rib).

5) Moments due to unsymmetrical loading at load position #2 as predicted by CANDE were in very good agreement with field values. Crown moments were overestimated at center span, but everywhere else along the unfolded length moments were in almost exact agreement. Thrusts and deflection distributions were very similar, but thrusts were underestimated and deflections were overestimated.

Fig. 5.5.8 Moments Due To 32 Kip Live Load At Position #1
Fig. 5.5.9 Shear Due To A 32 Kip Live Load At Position #1

Fig. 5.5.10 Thrust Due A 32 Kip Live Load At Position #1
Fig. 5.5.11 Deflections Due To a 32 kip Live Load At Position #1

Fig. 5.5.12 Moments Due To A 42 Kip Live Load At Position #1
Fig. 5.5.13 Shear Due To A 42 Kip Live Load At Position #1

Fig. 5.5.14 Thrusts Due To A 42 Kip Live Load At Position #1
Fig. 5.5.15 Deflections Due To A 42 Kip Live Load At Position #1

Fig. 5.5.16 Moments Due To A 16 Kip Live Load At Position #1
Fig. 5.5.17 Thrusts Due To A 16 Kip Live Load At Position #1

Legend
- SEOCON
- CANDE
- FIELD

Fig. 5.5.18 Deflections Due To A 42 Kip Live Load At Position #1

Legend
- SEOCON
- CANDE
- FIELD
Fig. 5.5.19 Moments Due To A Nonsymmetrical Live Load Of 32 Kips

![Graph showing moments with position #2 highlighted.]

Legend:
- FIELD
- CANDE

Fig. 5.5.20 Thrusts Due To An Unsymmetrical Load Of 32 Kips

![Graph showing thrusts with CANDE predicting small thrusts.]

Legend:
- FIELD
- CANDE

Note: CANDE predicts small thrusts, distribution is similar.
5.9 Summary

A comparison of CANDE and SEQCON with field values was performed to determine which would come closest to field values. Hyperbolic soil models were used in both instances and the soil parameters were identical for both cases. The soil elements were assumed bonded to the culvert so that 100% shear transfer would occur. The effects of compaction forces were not included in the analysis. The construction sequence occurring in the field was simulated as closely as possible. Live loading was simulated for both symmetrical and nonsymmetrical loading conditions.

Results from the analysis underestimated crown moments, and deflections during the early stages of backfill
placement. As backfill reached levels higher than the crown, moments were overestimated and deflections were underestimated. SEQCON came very close to predicting field values of thrust at the footer. During live load moments and deflections were overestimated – but thrusts at the footer were in close agreement. SEQCON came closest to predicting field values in the symmetrical analyses. CANDE came very close to predicting structural response due nonsymmetrical loading at load position #2. Both CANDE and SEQCON were in good agreement with haunch moments but both did not predict the large values of thrust occurring in the crown.
6.1 Review Of Work

A Lane Metals rib-reinforced box-type culvert, with a span of 15 feet 9 inches and a rise of 5 feet, was instrumented to obtain deflections and strains occurring during backfill and live load. Instrumentation consisted of mounting strain gages (130 strain measurements) on the culvert in seven critical sections and anchoring eye bolts in eleven locations for deflection measurements. In addition, horizontal displacements of the backfill mass were monitored with rod extensometers placed at four locations, two level with the end of the exterior rib and the remaining two 6-8" above the culvert crown. Two bolts (one at the interior rib end and the other at the exterior rib end) were instrumented with rosettes to investigate shear transmission between the rib and plate and stress concentrations due to the bolt.

Data acquisition began with the placement of backfill material around the culvert. At the completion of the construction sequence, which ended with the placement of 12"
of asphalt, live load was applied to the pavement surface at five locations for 50, 100 and 130 percent of the standard AASHTO HS_20 live load.

Strain data acquired in the field from gages mounted on the plate and ribs were converted to stresses using the appropriate form of the constitutive relation. The stresses were then used to calculate moments and thrusts in seven locations. Fully composite action between the plate and ribs was assumed. The instrumented cross section was analyzed as a plane stress problem. Due to the interior rib at the crown, crown sections were asymmetrical and the generalized flexural formula was used in moment and thrust calculations.

Strain data acquired from rosettes were used to determine principal stresses and shear stresses occurring at prescribed locations around the two instrumented bolts (one at the haunch, the other at the north footer). The nature of the shear transmission between plate and rib, and the effect of stress concentrations near the bolt as a possible mode of failure was investigated.

Simulation of the construction sequence was performed with two finite element programs CANDE and SEQCON and the results were compared to experimental findings. Half meshes were used to model the field problem during backfill and
provisions were taken to follow the actual construction sequence occurring in the field as closely as possible. Laboratory testing of the #603 backfill was not yet completed at the beginning of the finite element analyses and the hyperbolic parameters were thus not available for incorporation into CANDE or SEQCON. Instead, material parameters were obtained from another source.

Live load was also simulated with CANDE and SEQCON for the case of symmetrical loading. Half meshes were used with the load applied on the asphalt surface (40" of cover) directly over the culvert crown. In SEQCON runs, the live load was applied in three increments. In CANDE runs, the load was applied all at once. Non-symmetrical loading was modeled with CANDE using both front and back wheel loads to simulate loading at live load position #2.

6.2 Discussion Of Results

During the early stages of backfill placement, inward deflections were greatest at the north side of the culvert. The culvert crown exhibited virtually no upward movement as the backfill was placed at the culvert sides. The maximum inward deflection experienced by the north side was .169". Moments which resulted from 55" of backfill were symmetrical and reached maximum values at the sides and crown. The
haunch regions experienced very small moments. Maximum side moments were .714 Ft-Kips/Ft at the north footer and .657 Ft-Kips/Ft at the south. The crown experienced a negative moment of .906 Ft-Kips/Ft. The maximum thrust occurred at the north crown and was 15.9 Kips/Ft. Small tensile thrusts developed at both haunches.

As backfill material was placed over the culvert crown, all points in the crown region experienced similar downward deflections indicating that the culvert crown was deflecting downward as a rigid member. A maximum downward deflection of .730" occurred due to 32" of cover. Moments were maximum at the north haunch and the crown. A negative moment of 1.79 Ft-Kips/Ft occurred at the north haunch while the crown experienced a positive moment of 2.81 Ft-Kips/Ft. Thrusts were maximum at the north foundation and north crown 8.14 Kips/Ft and 12.1 Kips/Ft, respectively.

Rod extensometers placed in the backfill mass indicated that negative arching was occurring during the later stages of backfill and during live load. Axial thrusts that occurred during backfill and live load at the footers also indicated negative arching as verified by their larger magnitudes when compared to the weight of the soil column above the culvert crown and equivalent line loads.
Deflections occurring due to live load were much smaller than deflections due to backfill indicating that backfill material became stiffer with load application. Deflections increased in proportion to live load. The maximum deflection was caused by a 42 Kip live load at load position #3 and was .149" downward at the crown. The minimum crown deflection due to a 42 Kip live load at load position #2 was .074 " downward.

Moments were maximum for live load position #3. The moment distribution was unsymmetrical due to the load imposed by both the front and back wheels. This caused moments to be largest at the northern crown region. The maximum crown moment was 2.06 Ft-Kips/Ft due to a 42 Kip live load at position #3. The maximum haunch moment was 1.33 Ft-Kips/Ft.

Thrust distributions became much more uniform during live load. In addition, there were very few cases where tensile thrusts developed. Thrusts were maximum at the north crown region. The maximum thrust that occurred was 17.5 Kips/Ft due to a live load of 42 Kips at load position #4. Thrust distributions did not vary much with live load position.

Composite action was verified by plotting strains
versus distance from the neutral axis for the north crown region which experienced the largest moments and thrusts during backfill and live load. The more linear variation of strain with less live load, and the almost zero strain near the neutral axis was indicative of composite behavior. With increasing live load, the strain distribution became more non-linear and the neutral axis shifted downward slightly. In addition, most of the stress was carried by the outside rib.

Large lateral strains occurring in the outside rib at the crown were evidence of rib flattening. This has the effect of reducing the moment carrying capacity of the section and may contribute to local buckling as a possible mode of failure.

Stress concentrations due to the bolt at the connection of the exterior rib to the plate at the footer were compressively large just above the bolt. This is evidence that the bending of the rib was forcing the bolt to bear upward against the plate. Beneath the bolt the plate experienced large tensile stresses in the lateral direction indicating that the rib was forcing the plate to bend around it. Large tensile stresses also developed on the plate beneath the end of the interior rib at the haunch. This was due to the rib pulling the bolt away from the plate material.
below the interior rib end as it bent concave upward. The maximum combined stresses (42 Kip live load plus a 40" of cover dead load) were 7850 psi at the haunch and -7997 psi at the footer.

Results from the finite element analyses during the early stages of backfill placement underestimated crown moments and deflections. Good agreement, however, between footer thrusts and haunch moments was obtained. During the later stages of backfill placement, SEQCON came closest to predicting crown moments, footer thrusts, and crown deflections. CANDE and SEQCON predicted nearly identical haunch moments which were close to field values.

Moment and deflection distributions along the unfolded length of the culvert during backfill were overestimated for the most part. Predicted thrust distributions were not close to field values.

During live load application, CANDE and SEQCON overestimated crown moments and deflections with SEQCON to a lesser degree. Thrust distributions were underestimated by both CANDE and SEQCON at the culvert crown with SEQCON to a lesser degree. Good agreement was obtained for haunch moments. The non-symmetrical load condition modeled with CANDE predicted haunch moments but overestimated crown
moments. Predicted deflections were much larger than field values.

6.3 Conclusions

Results from field testing and the finite element analyses performed with CANDE and SEQCON indicate that:

1) The interior rib minimizes upward deflections of the crown during the early stages of backfill and helps to retain the original shape of the culvert.
2) Thrust distributions become more uniform with increasing backfill and compaction.
3) Uneven backfill placement at the culvert sides produces tensile thrusts at the foundation. In addition, it alters the symmetrical geometry of the culvert and moment and thrust distributions.
4) Shallow cover and the box shape geometry of the culvert influence negative arching.
5) When the backfill level is below the culvert crown, moments at the culvert foundation become appreciably as large or larger than crown and haunch moments.
6) Shallow cover limits longitudinal spreading of live load and as a result, the distribution of thrusts with respect to load position are similar. As far as thrusts are concerned, the effect of moving the live load around is to increase or
decrease the magnitude of thrust developing at all sections. The same applies for moment distributions and magnitudes with the exception of the crown.

7) Live load positions #1, 3 are the critical positions with respect to crown and haunch moments.

8) Moments and thrusts due to live load were smaller than those due to backfill indicating that the backfill was of good quality, was properly compacted and stiffened with repeating load application.

9) Due to the negative arching effect, soil debonding at the culvert crown and sides is not likely if a cohesive soil is used for backfill. Slip at the sides may occur but its effect would be more prevalent during backfill. Slipping of the backfill material would increase footer thrusts. Since good agreement between thrusts during backfill was obtained with SEQCON, it is apparent that the construction sequence can be modeled effectively without interface elements and that eight node soil elements yield more accurate stress distributions within the element.

10) Discrepancies between predicted moments and deflections and field values during the early stages of backfill placement are most likely the result of not including the effects compaction stresses in the analysis. Since the moments and deflections that would result if compaction forces were included would be in the opposite sense, they would subtract from moments and deflections that developed
in the latter stages of backfill placement thus reducing the totals at the end of the construction sequence.

11) Large stresses develop at the ends of the corrugated rib-stiffeners is due to the "pinching" type action that occurs as the plate bends around the rib. This effect manifests itself at the ending of the reinforcing ribs.

12) Rib and plate flattening reduce the moment carrying capacity of the structure and may lead to the development of local buckling and/or the formation of a plastic hinge. 13) The outside rib experiences the majority stress and strain in the direction of its longitudinal axis, thus the assumption of a one dimensional stress-strain field is valid.

6.4 Recommendations

To improve the structural performance of similar culvert structures, and to arrive at proper design guidelines, the following recommendations are listed below:

1) Maintain even levels of backfill material on either side of the culvert during backfill and compact the lifts simultaneously.

2) Extend the exterior rib-stiffener to the foundation to eliminate the "pinching" effect.

3) Conduct rib-plate load tests to investigate the effects of rib-flattening and longitudinal spreading of the plate.
4) Include hyperbolic soil parameters obtained from multiaxial tests performed on the #603 backfill in the finite element analysis to improve the legitimacy of the finite element findings.

5) Modify CANDE to accommodate eight node quadrilateral soil elements and three node beam elements.

6) Include compaction forces, both vertical and lateral, during the construction sequence.
REFERENCES


8. Ohio Department Of Transportation Division Of Highways, Testing Division. : Report No. Physical Characteristics of In-Situ Soil: Lane Culvert construction site, adjacent Duck Creek in Noble County, Ohio, on State Route 260, 1985.


A) Moment and thrust calculations:

\[ \sigma_1 = \pm \frac{M y_1}{I} + \frac{P}{A} \]

\[ \sigma_2 = \pm \frac{M y_2}{I} + \frac{P}{A} \]

two equations, two unknowns - solve for M and T.

\[ M = (\sigma_1 - \sigma_2) I / (y_1 + y_2) \]
B) Normal deflection calculation:

\[ y = y (-\sin \alpha) + X \cos \alpha \]
Fig. 4.18p  Transverse Stress Variation With Distance From Bolt

42 Kip live load at Pos. #1 / bolt at footer

Legend

- ARM 1
- ARM 3
- ARM 4
- ARM 5

Fig. 4.18q  Lateral Stress Variation With Distance From Bolt

42 Kip live load / bolt at haunch

Legend

- ARM 1
- ARM 2
- ARM 3
- ARM 4
Fig. 4.18r Transverse Stress Variation With Distance From Bolt

Legend
- ARM 1
- ARM 2
- ARM 3
- ARM 4

42 Kip live load / bolt at haunch
without bolt

LATERAL STRESS (psi)
DISTANCE FROM BOLT (inches)