Improved Load Distribution for Load Rating of Low-Fill Box Structures

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The University of Kansas

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| 15 | Supplementary Notes | Reinforced concrete box culverts are mostly used at shallow depths. Periodic evaluation of their load carrying capacities is required for load rating of the culvert by determining a rating factor (RF) or truck tonnage of an HS truck. The rating factor is defined as the capacity of the structure minus the dead load demand, and then divided by the live load demand. All the state DOTs are required to inspect and assess culvert conditions and capacities by load rating in every two years.

The distribution of live loads on the top slab of a box culvert plays a major role in determining the rating factor of the culvert. The current AASHTO guidelines do not consider the effects of pavements present above the fill while determining the load distribution. The distribution of the wheel load through a pavement may be different from that suggested by the current AASHTO guidelines. In addition to the pavement effect, the fill conditions (i.e., fill thickness and fill modulus) may affect the load distribution. Currently, there is lack of a design method to address the load distribution when a pavement is present above the fill.

In this research, two field tests were carried out on the concrete box culverts under rigid and flexible pavements respectively. The finite difference numerical models of the test culverts were created in the Fast Lagrangian Analysis of Continua in three dimensions (FLAC3D) software and were verified against the field test results. The verified finite difference models of the culverts were used for a parametric study to analyze the effects of pavement type (i.e., flexible and rigid pavement), pavement thickness, fill depth, and culvert span on the pressure distribution. The material properties and boundary conditions used in the models for the parametric study were similar to those used in the verified models.

The parametric study demonstrated that the intensity of the distributed vertical pressure on the top slab of the culvert gradually decreased as the pavement thickness increased. The vertical pressure under a rigid pavement was lower than that under a flexible pavement at the same pavement thickness. Within the range of the fill depth covered in this study, the intensity of vertical pressure decreased gradually with an increase of the fill depth over the culvert. The effect of the traffic load on the vertical pressure on the culvert was more significant at the lower fill depth and gradually decreased with the increase of the fill depth. The calculated vertical pressure decreased when the culvert span was increased from 1.8 to 5.4 m for a constant top slab thickness of the culvert. However, when the top slab thickness of the culvert increased, the vertical pressure at the larger span was close to that at the small span. The effect of the culvert span on the vertical pressure was negligible if the thickness of the top slab was properly designed.

The maximum vertical pressures obtained from the numerical analyses were compared with those calculated using the distribution formulae in the AASHTO guidelines. The comparisons showed that the current AASHTO guidelines over-estimated the pressure for low-fill culverts under a pavement. Simplified methods were developed in this study to estimate the vertical pressures under rigid and flexible pavements that closely match the experimental and numerical results. Proposed revisions to the current AASHTO LRFD Bridge Design Specifications are suggested and included in the appendix of this report. |
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Final Report

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PREFACE

The Kansas Department of Transportation’s (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

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Abstract

Reinforced concrete box culverts are mostly used at shallow depths. Periodic evaluation of their load carrying capacities is required for load rating of the culvert by determining a rating factor (RF) or truck tonnage of an HS truck. The rating factor is defined as the capacity of the structure minus the dead load demand, and then divided by the live load demand. All the state DOTs are required to inspect and assess culvert conditions and capacities by load rating in every two years.

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Chapter 1: Introduction

1.1 Background

Culverts are structures constructed below highways and railways to provide water drainage. They can sometimes provide access for animals to cross roads. The opening size of a culvert is determined based on a design flood, whereas the thickness of the culvert section is designed based on vertical loads applied on the culvert. A variety of materials have been used in the construction of culverts. Stone culverts have been used from ancient times. Rigid concrete to flexible thermal plastic culverts came into use more recently as buried or underground drainage structures in civil engineering. At present reinforced concrete culverts, steel culverts, and thermal plastic culverts are popular in practice. Circular, rectangular, and arch are the most commonly used culvert shapes and culverts with these shapes are called pipe culverts, box culverts, and arch culverts respectively. Culverts and bridges often serve the same purpose with the primary practical difference being the span length. The Federal Highway Administration (FHWA) considers a drainage structure constructed across a road having a total span less than 6 m as a culvert but above 6 m as a bridge.

Culverts are classified as rigid, semi-rigid or flexible based on material type, how they carry load, and to what degree they rely on the surrounding soil. The capacity of a culvert to carry imposed loads depends on many factors including the type and age of the material, the size and shape of the culvert, and the supporting materials around the culvert. Its capacity gradually decreases due to aging and degradation of the material after repeated loading by heavy trucks. The rate of the capacity reduction can be more significant if the depth of fill over the culvert is small and/or the culvert is frequently subjected to heavy truck loading. Reinforced concrete box culverts are the most common type used at shallow depths. Periodic assessment of their capacities is important for maintaining a state of good repair. The live load carrying capacity of a culvert is determined by load rating of the culvert. Load rating of a culvert is often done by evaluating a rating factor (RF) or truck tonnage of an HS truck. The HS truck is the design truck specified by the 1992 American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges and the 2007 AASHTO LRFD Bridge Design Specifications. The rating factor is defined as the capacity of the structure minus the dead
load demand, and then divided by the live load demand. In other words, the structure must have enough capacity after the dead load is subtracted to support the live load. If the rating factor falls below 1, the live load on the culvert should be reduced to maintain the culvert in a serviceable condition. The process of establishing the reduced live load on the culvert is called posting. All the state DOTs are required to inspect and assess culvert conditions and capacities every two years by load rating.

AASHTO provides three methods of load rating, namely Allowable Stress Rating (ASR), Load Factor Rating (LFR), and Load and Resistance Factor Rating (LRFR) (AASHTO, 2011). Regardless of the rating method employed in load rating, the distribution of live loads on the top slab of the box culvert plays a major role in determining the rating factor of the culvert. The 1992 AASHTO Standard Specifications for Highway Bridges and the 2007 AASHTO LRFD Bridge Design Specifications provided the guidelines for determining the live load distribution on the culvert. The existing guidelines have evolved through time with the contributions of many researchers. Recently, more research has been carried out to modify the existing guidelines to provide more accurate ways of modeling load distribution over culverts.

1.2 Problem Statement

Culverts are installed at grade to different depths. The effect of the live load becomes more significant when the culverts are installed at shallow depths. As load rating of culverts depends on how the live load is distributed on the culvert, a more rational approach to the modeling of the live load distribution over a shallow fill culvert is important in determining its live load capacity. The 1992 AASHTO Standard Specifications for Highway Bridges considered the wheel load as a point load on the surface with the load distributed onto a square area having a width of 1.75 times the fill depth, H, above the culvert as shown in Figure 1.1. The 2007 AASHTO LRFD Bridge Design Specifications suggested the wheel load is applied on a rectangular area (t_l x t_w) of 0.5 m x 0.25 m on the surface as a tire footprint, which is distributed onto the culvert by increasing the tire footprint by H or 1.15 H depending on the type of fill as shown in Figure 1.2.
FIGURE 1.1
Wheel Load Distribution per 1992 AASHTO Standard Specifications for Highway Bridges
FIGURE 1.2
Wheel Load Distribution per the 2007 AASHTO LRFD Bridge Design Specifications

(a) For select granular backfill

(b) For soils other than select granular backfill
However, the 1992 AASHTO Standard Specifications for Highway Bridges and the 2007 AASHTO LRFD Bridge Design Specifications do not consider the effects of pavements present above the fill when determining the load distribution. Their distributions are valid for the design of the culvert when there is no pavement present above the fill, which often occurs during construction. But for the case of load rating of existing culverts, pavements (rigid or flexible pavements) often exist above the fill. The distribution of the wheel load through a pavement may be different from those suggested by the AASHTO guidelines. In addition to the pavement effect, the fill conditions (i.e., fill depth and fill modulus) may affect the load distribution. Currently, there is lack of a design method to address the load distribution when a pavement is present above the fill.

1.3 Research Objective

The objective of this research was to study the distribution of live load on the top of a low-fill concrete box culvert by considering the effects of pavement type (i.e., flexible and rigid pavements), pavement thickness, fill depth, and span of the culvert. The test data and numerical results provided a basis for the development of simplified methods for load distribution through rigid or flexible pavements for load rating of low-fill concrete box culverts.

1.4 Research Methodology

The methodology adopted in this research included a comprehensive literature review of the research in this application, two field tests on low-fill box structures (one under a rigid pavement and another under a flexible pavement) using loaded trucks, verification of numerical models in Fast Lagrangian Analysis of Continua in three dimensions (FLAC3D) based on the field test results, a 3D numerical parametric study, and development of simplified design methods for estimating the load distribution considering the existence of a pavement. The following influence factors were investigated within typical design ranges in the parametric study: (1) flexible and rigid pavements, (2) pavement thickness, (3) fill depth, and (4) culvert span. In the numerical analysis, pavements, box culverts, and soils were modeled as elastic materials.
1.5 Organization of Report

This report is divided into six chapters and one appendix. Chapter 1 presents the background, problem statement, research objective, and research methodology. Chapter 2 describes the present state of knowledge in load distribution and load rating of box culverts and a literature review on wheel load distribution on low-fill box structures. The field testing of culverts and lab testing of the pavement materials and fill materials to determine their properties are presented in Chapter 3. Chapter 4 presents the verification of the FLAC3D numerical models of the tested culverts. The parametric study carried out to determine the effects of different influence factors in load distribution is presented in Chapter 5. Proposed simplified methods for pressure distribution are presented in Chapter 6. Conclusions and recommendations for future work are provided in Chapter 7. Proposed revisions to the current AASHTO LRFD Bridge Design Specifications are included in the appendix of this report.
Chapter 2: Literature Review

This chapter presents a literature review on relevant topics of this research. It begins with the classification of box culverts as outlined in the literature, which is followed by descriptions on load rating, different levels and methods of load rating, and posting policy. The AASHTO guidelines for load distribution for culverts are discussed in conjunction with the previous studies. Influence factors in load rating of box culverts are discussed. Finally, constitutive models for soils are discussed.

2.1 Classification of Box Culverts

The Kansas Department of Transportation classifies box culverts in three categories based on the span: bridge boxes, 3 m to 6 m structures, and road culverts (KDOT, 2011). The 2011 KDOT Bridge Design Manual defines a bridge box as a structure having a width greater than 6 m measured along the centerline of the roadway from the inside faces of both exterior walls (including all span widths and the thicknesses of all interior walls). A box culvert with a total width of 3 m or greater (measured perpendicular to the centerline of box) but less than or equal to 6 m (measured along the centerline of the roadway) is considered as a 3 m to 6 m structure. A road culvert is defined as a structure having a width of less than 3 m from the inside faces of both exterior walls measured perpendicular to the centerline of the box. The manual further classifies the box culverts as reinforced concrete boxes (pinned) and rigid frame boxes (fixed) based on wall to slab connection. A box designed with walls and slabs that are assumed to be simple spans (independent of one another) is a pinned box. A box designed with walls and slabs that are assumed to be continuous or connected to each other is called a fixed box. According to the type of installation, box culverts can also be classified in three categories: embankment culvert, trench culvert, and imperfect trench culvert (Yoo et al. 2005, Lawson et al. 2010, and Kang et al. 2008). In addition to these methods, tunneling is recognized as a method of installation of buried culverts (Sandford 2010). Culverts installed on existing soil or fill and then covered by backfill as shown in Figure 2.1 are referred as embankment culverts. In such culverts even the well-compacted surrounding soil mass is less stiff than the combined culvert and soil column. Therefore, the backfill material around the culvert has tendency to settle more than the
soil directly above the culvert. The relative stiffness of the combined culvert and soil column to the surrounding soil controls the magnitude and distribution of vertical pressures on structures (Lawson et al. 2010 and Sun et al. 2011).

(Source: Lawson et al. 2010)

**FIGURE 2.1**
*Embankment Culvert*

Figure 2.2 illustrates the trench installation. Trench installation culvert is mostly adopted in actual construction in field. Here the backfill soil is less stiff than the surrounding in-situ soil and undergoes more settlement relative to the in-situ soil.
Figure 2.3 shows an imperfect trench culvert, which is still a subject of research. Therefore it is not as popular as embankment and trench installation culverts. This kind of culvert is installed in a similar way as an embankment culvert with the exception that a layer of compressible material, such as geofoam, straw or compressive soil, is placed directly above the culvert. The remaining part of embankment is backfilled to the final level and compacted with a typical procedure. The compressible layer reduces the stiffness of the soil column to a lower level than that of the surrounding soil. Consequently, the settlement becomes comparable to the trench culvert and an imperfect trench culvert has the same load reduction behavior as the trench culvert (Kim and Yoo 2005, Kang et al. 2008, and Lawson et al. 2010). An instrumented field test carried out by Sun et al. (2011) found that the vertical stresses were greatly reduced by using the imperfect trench technique. The vertical stresses acting on the culvert were only about 9% to 11% of those occurring on the portion of the culvert where an imperfect trench was not used.
2.2 Load Rating

On December 15, 1967, the Silver Bridge on the U.S. 35 Highway between Point Pleasant in West Virginia and Gallipolis in Ohio collapsed, killed 46 people, and injured nine. The investigation revealed that a cracked eyebar created extra stress on other members of the bridge, which ultimately caused the collapse of the bridge. The incident raised national concern on the condition of bridges, which led to the establishment of the National Bridge Inspection Standards (NBIS) in early 1970s (Jaramilla and Huo 2005). Since then FHWA, AASHTO, and other agencies have developed guidelines for inspecting and maintaining existing bridges. The idea of load rating of bridges was eventually developed.

Load rating, in general, involves the determination of live load capacities of culverts or bridges. AASHTO (2011) defined load rating as the maximum truck tonnage, expressed in terms of HS load designation, permitted across a culvert. The 2011 KDOT Bridge Design Manual described load rating as the analysis of culverts and bridges performed to determine the live load that structures can safely carry.

The culvert load rating process is part of the regular inspection process. It involves the process of determining the safe load-carrying capacity of the culvert structure, and finding
whether design, legal or permit vehicles can safely cross the culvert, thus determining if the culvert needs to be restricted and if so, what level of load posting is required. Load rating is carried out based on the current culvert condition and needs analysis and engineering judgment by comparing the culvert structural capacity and dead load demand to live load demand (Lawson et al. 2009). A complete description of an as-built bridge, modifications since it was built, and knowledge of its present condition are required to load rate a bridge (KDOT 2011). The load rating engineer performs a detailed inspection of the culvert beforehand. If the plan is not available, a set of measurements become necessary to determine the dimensions of the culvert that are needed for capacity and demand calculations. Actual conditions of the culvert are represented in the calculations by considering a reduction in the section, if any, and reduced resistance of the materials. Areas of deterioration become a special concern during field inspection because a reduced section of a member may control the capacity of the structure.

Jaramilla and Huo (2005) pointed out three major requirements for a bridge owner to carry out load rating. Firstly, the culverts deteriorate gradually during their long service life, thus assessment of their reduced capacities becomes important to ensure that the culverts can perform safely under current traffic loads. Secondly, existing culverts were constructed using different design loads, material strengths, and design methods at different times based on the design standards evolved at that time. So it becomes necessary to load rate them for the current traffic condition using current specifications. Lastly, permit rating is required for the culvert carrying a load more than the legal load. Heavy or frequent permit vehicles can cause permanent damage to the structure if its capacity is not determined appropriately.

Numerical analysis can be carried out to find the structural response at critical sections based on loading. The outcome of the analysis is used as input for the load rating equation to determine the appropriate load rating. Results obtained from load rating are used to determine if the culvert needs to be restricted to reduced loads, to help identify culvert components that require rehabilitation, or to help devise retrofitting measures to avoid the posting of the culvert (KDOT 2011). If a culvert is not safe enough to carry the loads allowed by state statute, it is posted at lower capacity. Kansas State Statute permits a gross vehicle weight of 355 kN on the Interstate and 380 kN on other highways without a special permit.
2.2.1 Levels of Load Rating

Bridges are load-rated at two different levels, referred to as "Inventory Rating" and "Operating Rating" (KDOT 2011). Inventory Rating is the load level a structure can safely withstand for an indefinite period of time. Operating Rating is the absolute maximum permissible truck load that may be on the culvert. Thus, the inventory load level is approximately comparable to the design load level under normal service conditions and the Operating Rating is indicative of the capacity of the structure for occasional use. If an unlimited number of vehicles are allowed on the structure at the operating level, it will reduce its service life. This value is typically used when evaluating overweight permit.

The 2011 AASHTO Manual for Bridge Evaluation suggested the following equation for the rating factor:

\[
RF = \frac{C - A_1 D}{A_2 L(1 + I)} \tag{Equation 2.1}
\]

where
- \( RF \) = the rating factor,
- \( C \) = the structural capacity of the member,
- \( D \) = the dead load effect on the member,
- \( L \) = the live load effect on the member,
- \( I \) = the impact factor,
- \( A_1 \) = the factor for dead loads,
- \( A_2 \) = the factor for live loads

Equation 2.1 clearly indicates that the main factors for load rating are the culvert capacity, the dead load demand, and the live load demand. The 2011 AASHTO Manual for Bridge Evaluation provided the guidelines for calculating the section capacity considering the reduction in the material capacity based on age and condition of deterioration of culvert components while the dead load and live load demands are determined by computer analyses. This simple load rating equation offers a real challenge to obtain reliable values for each of these governing factors. This is because the 2011 AASHTO Manual for Bridge Evaluation did not
suggest any particular tool for analyzing the dead load and live load effects. The output of the analysis varies with the simulation technique used. It is necessary to determine the rating factors using Equation 2.1 for each critical section of the culvert, such as corners, mid-spans, top and bottom slabs, and interior and exterior walls, for each demand type (i.e., moment, shear, or thrust), and for maximum and minimum load envelopes at both inventory and operating rating levels. Critical sections of a culvert for the moment are shown in Figure 2.4. The lowest inventory rating factor and the lowest operating rating factor control the load rating for the culvert (Lawson et al. 2009).

Abbreviations for the typical critical sections shown in Figure 2.4, listed clockwise, are: top exterior corner (TEC), top exterior mid-span (TEM), top interior corner (TIC), top interior mid-span (TIM), wall top interior corner (WTIC), wall interior mid-span (WIM), wall bottom interior corner (WBIC), bottom interior mid-span (BIM), bottom interior corner (BIC), bottom exterior mid-span (BEM), bottom exterior corner (BEC), wall bottom exterior corner (WBEC),

(Source: Lawson et al. 2010)

FIGURE 2.4
Moment Critical Sections for a Culvert without Any Haunch
wall exterior mid-span (WEM), and wall top exterior corner (WTEC). For multiple-span box culverts, the sections are identified by culvert span, e.g., TIC1, TIC2, BIC1, BIC2, etc.

Variability in the methods for assessing the culvert capacity, dead load demand, and live load demand leads to varying rating factors. Original construction documents and material property assumptions used in the design provide bases for calculating the capacity; however, they need to be used in conjunction with visual inspection of culvert conditions. The demand calculation process is carried out through analytical modeling. The 1992 AASHTO Standard Specifications for Highway Bridges and the 2007 AASHTO LRFD Bridge Design Specifications provided the values for all input parameters, such as soil unit weight, equivalent fluid weight for lateral load, and live load distribution through soil; however, these AASHTO guidelines do not specify the analytical model to be used or and the method of applying the load to the model. Therefore, load rating engineers must make their decisions about modeling practices and procedures. In addition, the assumptions, simplifications and mathematical structures of demand modeling tools can have a significant effect on the culvert load rating analysis (Lawson et al. 2009).

The response of a bridge depends not only upon the total weight from a vehicle, but also upon the axle configuration and the distribution of loads between the axles (KDOT 2011). Since it is not pragmatic to rate a bridge for a countless number of possible axle configurations, KDOT has a policy to load-rate highway bridges for eight standard vehicles which closely represent the actual vehicles on highways. These are the rating trucks, which are divided into five categories as shown in Table 2.1. The 2011 KDOT Bridge Design Manual also requires all bridges to be rated for the same truck to achieve consistency on the local and state system bridges. The standard truck is the "H" truck, which is a design truck. "T-3", "T3S2", and "T3-3" are recommended by AASHTO. FHWA requires the "HS". The "T130" and "T170" are used for special permits on state highways. In addition, the heavy equipment transport (HET) truck is required by KDOT (KDOT 2011).
TABLE 2.1  
Trucks Being Used for Load-Rating KDOT Bridges  

<table>
<thead>
<tr>
<th></th>
<th>Max gross weight (tons)</th>
<th>Posting weight (tons)</th>
<th>Location on posting sign</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Truck:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H Unit</td>
<td>20.0</td>
<td>12.5</td>
<td>Top</td>
</tr>
<tr>
<td>Type 3 Unit:</td>
<td>27</td>
<td>25</td>
<td></td>
</tr>
<tr>
<td>Truck-tractor Semi-Trailer</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HS Unit</td>
<td>36.0</td>
<td>22.5</td>
<td>Middle</td>
</tr>
<tr>
<td>Type 3S2 Unit</td>
<td>36.0</td>
<td>36</td>
<td></td>
</tr>
<tr>
<td>Truck Trailer and LCV's:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type 3-3 Unit</td>
<td>40</td>
<td>40</td>
<td>Bottom</td>
</tr>
<tr>
<td>Permit:</td>
<td></td>
<td></td>
<td>N/A</td>
</tr>
<tr>
<td>Type T130 Unit</td>
<td>65.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type 170 Unit</td>
<td>85.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Special Kansas:</td>
<td></td>
<td></td>
<td>N/A</td>
</tr>
<tr>
<td>Heavy Equipment Transport:</td>
<td></td>
<td>109.9</td>
<td></td>
</tr>
</tbody>
</table>

(Source: KDOT 2011)

The choices of KDOT for Standard Load Rating Trucks is based on several years of truck weight data collected in Kansas and also based on the recommended AASHTO Rating Trucks (KDOT, 2011). The KDOT Bridge Design Manual (2011) provided two reasons for using H-Trucks and HS-Trucks for load rating: (1) their familiarity in design and (2) they are conservative. The maximum axle loads allowed on the Kansas highway system are 10 tons for a single axle and 17 tons for a dual axle, with a maximum weight not to exceed 42.75 tons (40 tons on the Interstate), without special permits. The weight on a group of axles is limited by Kansas Statutes K.S.A. 8-1908 and 8-1909.

2.2.2 Load Rating Methods

Load rating methods include load testing and analysis (KDOT 2011). Load testing of a culvert for load rating is not economic. Therefore load testing is carried out only under special conditions where an analytical method cannot be used due to some specific difficulties. For the analysis method of load rating, commercially available software is used to determine the live load and the dead load demands for each mode of culvert response (i.e., moment, shear, or thrust). Culverts can also be load-rated by load testing and calibrating the finite
element/difference model and subsequently carrying out an analysis for load demands (Yost et al. 2005 and Chajes and Shenton 2005). The load testing method helps to eliminate the unreliable conditions. Approximately 95% of bridges out of more than 200 bridges analyzed by Yost et al. (2005) obtained higher load ratings using the load testing method. Schulz et al. (1995) also stated that this method could increase the rating of the bridges but they also presented an example showing where the rating decreased. The load testing method is more convenient for steel structures than concrete structures. Chajes and Shenton (2005) outlined four important factors for the load testing method that affect load rating, including (1) lateral load distribution, (2) support fixity, (3) composite action, and (4) effects of secondary members.

The 2011 AASHTO Manual for Bridge Evaluation included three methods of analysis to load-rate a structure, i.e., load and resistance factor rating (LRFR), allowable stress rating, and load factor rating (LFR). The 2011 KDOT Bridge Design Manual included two methods of analysis used to load-rate structures: the Load Factor Rating (LFR) method and the Load and Resistance Factor Rating (LRFR) method. The working stress method is also used by some DOTs to load-rate their bridges (e.g., the 2005 DelDOT Bridge Design Manual). KDOT has used the Load Factor Rating Method to load-rate bridges since 1988. The philosophy of the LFR method is to use smaller factors of safety for more predictable loads (such as dead loads) and higher factors of safety for less predictable loads (such as live loads). The introduction of new criteria, such as load modifiers, multiple presence factors, change in distribution of live loads and dynamic load allowance, in the LRFR method, sometimes produces a rating factor less than 1.0 for the bridges which pass the criteria in the LFR method (NCHRP 2011).

The introduction of the LRFD specification for highway bridges by AASHTO in 1994 made it necessary to develop a new bridge evaluation manual to be consistent with the new specification. NCHRP Project 12-46 was initiated in March 1997 to develop a new AASHTO LRFR manual for bridge evaluation (Shivakumar 1999). The 2011 AASHTO Manual for Bridge Evaluation incorporated the load and resistance factor methodology for load rating of bridges. This method needs the use of site specific information to support an Engineer’s judgment on the safe rating level for a particular bridge. Under this specification, a bridge’s rating may be
improved by making use of options related to thorough inspection and maintenance or control of heavy overloads (KDOT 2011).

The current policy at KDOT is to load rate all structures using the LFR method. However, a structure designed or rehabilitated with the Load and Resistance Factor Design (LRFD) is also rated using the LRFR Specification. For purposes of reporting to FHWA, the HS-type truck is used for LFR rating and HL-93 loading is used for LRFR (KDOT 2011).

Rund and McGrath (2000) compared all the provisions from the 2002 AASHTO Standard Specifications for Highway Bridges (17th edition) and the 1998 AASHTO LRFD Bridge Design Specifications (2nd edition) for precast concrete box culverts. The analysis conducted on several combinations of box culvert sizes and fill depths under both specifications revealed that the provisions from the LRFD specifications yielded higher design loads and therefore required more area of steel reinforcement. More steel was particularly required for the low-fill culverts having a fill of 0.6 m or less. Therefore, the low-fill culverts constructed according to the LRFD specifications yield higher capacities and hence have a higher load rating than those constructed according to the LFD method.

### 2.2.3 Analytical Steps for Load Rating

The analytical steps that are followed to load-rate each member are similar and independent of the type of a member and the role played by the member in the overall structure. The method of analysis varies with any of the steps for each member, depending on the member and the choice of LFR or LRFR, but the function of the calculations is the same. The 2011 KDOT Bridge Design Manual summarized the following analytical steps:

1. Determination of section properties.
2. Determination of allowable and/or yield stresses.
3. Calculation of section capacity.
4. Determination of dead load effect.
5. Calculation of dead load portion of section capacity.
7. Calculation of live load impact and distribution.
8. Calculation of allowable live load.

2.2.4 Posting Policy

The 1995 FHWA publication *Recording and Coding Guide of the Structure Inventory and Appraisal of the Nation's Bridges* states: “Although posting a bridge for load-carrying capacity is required only when the maximum legal load exceeds the operating rating, highway agencies may choose to post at a lower level.” Posting of a culvert at a particular level should not shorten the life of the culvert and the level of posting must be less than or equal to the operating rating. KDOT commonly employs a level of posting that is approximately midway between inventory and operating rating. The public authority responsible for inspection and maintenance of a structure has the authority to post anywhere within this range. It may not be wise or advisable to commonly post the culvert near the operating rating (KDOT, 2011).

The 2011 KDOT Bridge Design manual also provides the option of posting a speed limit. A reduced speed reduces the impact and increases the posted load or eliminates the need for posting altogether. This alternative is not generally considered feasible and has not been used by KDOT. If a bridge is not capable of carrying three tons of load at the operating level, KDOT closes that bridge. The manual directs that if a structural rating is low, the local authority should consider using the load factor method and post the culvert at the operating rating as the maximum posting value. However, posting the culvert at operating rating should be employed only with more frequent inspections, for short periods, or until repair or rehabilitation can be done.

If the inventory and operating rating factors are greater than 1.0, the culvert will be unrestricted (Lawson et al. 2010). The culvert load rating is obtained by multiplying the rating factors by the tractor tonnage (i.e., 20 tons for HS-20 trucks) to determine the operating (OR) and inventory (IR) load ratings. If either of the inventory rating factor or operating rating factor is less than 1.0, the culvert may be subjected to load posting. In some cases the load rater may prefer to perform the analysis again using a higher level of model sophistication to avoid posting.
2.3 AASHTO Guidelines for Load Distribution

The AASHTO Standard Specifications for Highway Bridges and AASHTO LRFD Bridge Design Specifications are two primary documents which provide the guidelines about the live loads in the design of the box culverts. The same guidelines are also used for the load rating application. AASHTO introduced the Load and Resistance Factor Design (LRFD) Bridge Design Specification methodology in 1994. Before the introduction of the LRFD specifications, the Standard Specifications was the only guideline for determining the live loads on the culverts. The aim of the LRFD specifications is to provide a reliability-based code which can offer a more consistent level of safety than the existing Standard Specifications. Both guidelines used load factors and strength reduction factors. The LRFD specifications not only included load factors and strength reduction factors but also added provisions for load modifiers, multiple presence factors, design vehicle loads, distribution of live loads through fill, and dynamic load allowance.

A load factor accounts for the uncertainty inherent in the specific load or combination of loads whereas the load modifiers relate mainly to ductility, redundancy and importance of a particular component of the structure or the structure as a whole. The 1992 AASHTO Standard Specification for Highway Bridges did not use the concept of the load modifiers, even though it included the provision of load factors. The 2007 AASHTO LRFD Bridge Design Specifications used a load factor of 1.75 as compared with the load factor of 2.17 used by the 1992 AASHTO Standard Specification for Highway Bridges. However, this change in the load factor in the code is covered by the introduction of multiple presence factors. The values of the multiple presence factors depend on the numbers of loaded lanes. According to the 2007 AASHTO LRFD Bridge Design Specifications, the multiple presence factor is 1.2 for a single loaded lane, 1.0 for two loaded lanes, 0.85 for 3 loaded lanes, and 0.65 for 4 or more loaded lanes.

The design vehicle load was another important factor changed from the AASHTO standard specifications to the AASHTO LRFD specifications. The Design truck and design tandem are two design vehicles specified by the AASHTO LRFD specifications. The design truck is the same as the HS20 truck as specified in the AASHTO standard specification. The axle load for the design tandem is increased from 107 to 111 kN from the AASHTO standard specifications to the AASHTO LRFD specifications. Figures 2.5 and 2.6 show the design truck
and design tandem respectively. Furthermore, the AASHTO LRFD specifications specified that both the design truck and design tandems should be accompanied with the design lane load of intensity 8 kN/m uniformly distributed over a lane width of 3 m.

FIGURE 2.5
Design Truck HS20 in the 2007 AASHTO LRFD Bridge Design Specifications

FIGURE 2.6
Design Tandem in the 2007 AASHTO LRFD Bridge Design Specifications
The 2007 AASHTO LRFD Bridge Design Specifications defined the tire contact area of a wheel consisting of one or two tires to be a single rectangle, whose width and length are 0.5 m and 0.25 m, respectively. The tire pressure is assumed to be uniformly distributed over a contact area.

2.3.1 Dead Loads

The 1992 AASHTO Standard Specifications for Highway Bridges and the 2007 AASHTO LRFD Bridge Design Specifications provide the guidelines for the dead load to be used in load rating. AASHTO has modified its guidelines over the years to impose more load on culverts. In-service culverts, which were built many years ago, must be reanalyzed using the 2011 AASHTO Manual for Bridge Evaluation methods. Many researchers, for example, Tadros and Benak (1990) and McGrath et al. (2005), have contributed to the determination of the loads on culvert top, base and sides.

The development of the AASHTO provision for culvert design began with the American Association of State Highway Officials' (AASHO) standard specifications for highway bridges in 1949. The 1949 AASHO standard specifications adopted the unit weight of compacted sand, earth, gravel or ballast as 18.9 kN/m$^3$. According to the 1949 AASHO standard specifications, the earth load on a culvert could be computed ordinarily as the weight of earth directly above the slab. The 1949 AASHO standard specifications allowed the effective weight of the soil to be taken as 70% of its actual load to calculate the design load on the culvert.

In 1983, the AASHTO standard specifications defined the effective horizontal unit weight as 4.7 kN/m$^3$. All the AASHTO standard specifications before 1987 allowed the use of vertical soil pressure of 0.7 times the equivalent fluid pressure having a unit weight of 18.9 kN/m$^3$ and horizontal soil pressure equal to the equivalent fluid pressure having unit weight of 4.7 kN/m$^3$ (Abdel-Karim et al. 1993). Tadros et al. (1987) concluded that the earlier AASHTO specifications were un-conservative for the soil loads. The AASHTO interim report released in 1987 updated the 1983 standard specifications and revised the lateral soil pressure to the equivalent fluid pressure having a unit weight of 4.7 to 9.4 kN/m$^3$. This report also removed the vertical pressure reduction factor of 0.7 (Abdel-Karim et al. 1993, Kim and Yoo 2005). In 1990,
the AASHTO standard specifications was further updated by including an equation for calculating the earth pressure on the reinforced concrete box structures, which included a soil-structure interaction factor for embankment and trench installations. It was the first time when the soil-structure interaction of reinforced concrete box culverts was addressed by the AASHO standard specifications. This method for determining the soil pressure was continued in the later editions of the AASHTO standard specifications (Lawson et al. 2010). Culvert load rating parameters associated with these installation methods should be used as specified by the 2011 AASHTO Manual for Bridge Evaluation.

A lateral dead load is applied as a pressure from equivalent fluid on the side walls of the culvert. The AASHTO standard specifications do not include any provisions for the compaction requirements and geometry of the side fill for box culverts. However, the Yoo et al. (2005) study showed that compaction of the side fill had a significant effect on the behavior of the box culvert. According to Lawson et al. (2009), the 2002 AASHTO Standard Specifications for Highway Bridges required using a lateral pressure of equivalent fluid having a unit weight of 9.4 kN/m$^3$ for a total load case and 4.7 kN/m$^3$ for a reduced load case. The total load case generates the maximum axial and shear demands in all components of the culvert and creates the maximum moments in all critical sections except the top and bottom slab mid-spans. The reduced lateral load case is analyzed to determine the maximum moments at positive moment sections. This load creates a worst case of loading for the slabs by reducing the deflection of the walls caused by the lateral loads. Lateral live load surcharge is not considered in this case to further reduce the amount of the lateral pressure.

Yoo et al. (2005) stressed that the behavior of the box culvert was more dependent on the method of installation than the yielding or unyielding type of the foundation. Furthermore, Lawson et al. (2010) found that soil arching and culvert deformation were two primary factors that determined the magnitude and distribution of soil loads on a culvert. Soil arching, which is the result of the differential settlement in soil, has an indeterminate effect on soil load. When one section of soil settles more than its adjacent section, shear stresses develop to resist the settlement. Soil arching on the culvert depends primarily on the type of culvert installation.
A negative arching effect can develop for embankment installation culverts. For embankment installation culverts, the combined column of culvert and soil is stiffer than the surrounding soil. When the surrounding soil settles more than the soil above the culvert, shear planes develop along the interface between soil and culvert. These shear forces transfer some of the adjoining soil weight onto the culvert. As a result, the culvert carries the weight of the soil column directly above it as well as some of the surrounding soil weight. Figure 2.7 shows this negative soil arching effect. As the soil continues to settle over time, the load will continue to increase. Some studies suggested that the increased load might be as much as twice the weight of the in-situ soil column (Tadros 1986, Yang 1997, Yang et al. 1999, Kang et al. 2008, Sandford 2010).

(FIGURE 2.7) Embankment Culvert Installation

Positive arching occurs in trench installation culverts and imperfect trench culverts for which the combined culvert-soil column becomes less stiff and has a larger settlement than the adjoining soil. Therefore, the shear stress and load changes are in the opposite direction than those for the embankment installation culvert. The resulting load reduction can result in less than
half the weight of the soil column being carried by the culvert. Figure 2.8 shows this positive soil arching effect (Dasgupta and Sengupta 1991, Vaslestad et al. 1993).

To account for the soil structure interaction, the 2007 AASHTO LRFD Bridge Design Specifications allowed the use of the soil structure interaction factor, $F_e$. As shown in Equation 2.2, the soil structure interaction factor incorporates the increase in the soil load due to negative arching in embankment installation culverts. A lower value of $F_e$ is suggested if the soil is compacted. The soil structure interaction coefficient is 1.15 for installation with compacted backfill or 1.4 if the fill is uncompacted along the sides of the box section.

$$F_e = 1 + \frac{0.2 \cdot H}{B_c} \quad \text{Equation 2.2}$$

where $F_e$ = soil-structure interaction coefficient for an embankment installation culvert,

$H$ = fill depth above the culvert,

$B_c$ = outside width of the culvert.

The soil-structure interaction coefficient for a trench installation culvert, where positive arching occurs, is given by Equation 2.3:

$$F_t = \frac{C_d \cdot B_d^2}{H \cdot B_c} \leq F_e \quad \text{Equation 2.3}$$

where $F_t$ = soil-structure interaction coefficient for a trench installation culvert,

$B_d$ = width of the trench,

$C_d$ = load coefficient for trench installation culvert.
The nature of the deflection of the culvert also affects the load acting on the culvert. When the top and bottom slabs of the culvert deflect, the soil begins to transfer the load away from the center of the span to the outside of the culvert. The stress transfer causes a decreased load in the mid-span and an increased load near the supports. As a result, the moment at the mid-span decreases. As shown in Figure 2.9, the actual pressure distribution was found to be parabolic instead of uniform (Katona and Vittes 1982, Dasgupta and Sengupta 1991, Lawson et al. 2010). Oswald (1996) indicated the possibility of load redistribution resulting from creep of concrete.
Lateral loads also produce deflections of the culvert members, which affect the loads around the culvert. The nature of such deflections is opposite to that induced by the vertical pressure as shown in Figure 2.10. This effect causes the decrease in moments in the top and bottom slabs and increases in the walls (Awwad 2000). Yang et al. (1999) carried out two field tests on culverts to study the earth pressures on the culvert roofs and walls and carried out a parametric analysis using the finite element method. This study revealed that the prevailing 1996 AASHTO load distribution factors for the soil load were un-conservative. The lateral soil pressure was found to be dependent on backfill modulus with a higher modulus yielding a higher pressure. Relatively higher pressures were observed near the base of the culvert, which induced a high shear stress on the bottom of the wall.
From the literature, it becomes apparent that the AASHTO guidelines for the soil loads in embankment culverts are un-conservative, and those for the trench and imperfect trench culverts are over-conservative. For trench and imperfect trench culverts, Lawson et al. (2010) indicated the possibility that a more refined analysis could minimize the over-conservatism in load ratings and still maintain an acceptable factor of safety. However, this approach may not be applicable for very old culverts because the soil overburden stresses may become stable over time and no installation effect on soil pressure remains. As far as the load rating is concerned, the effect of the culvert deflection on the soil pressure should be considered.

2.3.3 Impact Factors

The 1992 AASHTO Standard Specification for Highway Bridges directed the live loads to be increased by a factor to consider the dynamic, vibratory and impact effects. This specification limited the impact factor only to culverts with a fill depth of 3 ft or less and allowed the impact to be reduced by steps as shown in Table 2.2. It is recommended that the impact factor be 20% for a fill depth ranging from 0.3 m to 0.6 m, 10% for a fill depth from 0.6 m to 0.9 m,
and 0% for a fill depth of 0.9 m or greater. On the other hand, Article 3.6.6.2 of the 2007 AASHTO LRFD Bridge Design Specifications suggested increasing loads for impact effects by 33% for a zero fill depth. The effect does not drop to 0% until the fill depth is 2.4 m. Live load estimates for fill depths of 0.6 to 1.2 m developed using the 2007 AASHTO LRFD Bridge Design Specifications are 15 to 20% higher than those developed using the AASHTO standard specifications (NCHRP, 2010).

<table>
<thead>
<tr>
<th>Fill depth (m)</th>
<th>Impact factor (I)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 0.3</td>
<td>30%</td>
</tr>
<tr>
<td>0.3 to 0.6</td>
<td>20%</td>
</tr>
<tr>
<td>0.6 to 0.9 m</td>
<td>10%</td>
</tr>
<tr>
<td>≥ 0.9 m</td>
<td>0</td>
</tr>
</tbody>
</table>

KDOT uses a straight line interpolation for calculating the impact factor, as shown in Equation 2.4, to avoid the sudden jumps in the live load moments caused by the steps shown in the AASHTO standard specifications.

\[
\text{\% Impact: } 0 < 30 - [(F - 1) \times 10] < 30
\]

Equation 2.4

where \( F \) = fill depth in feet

2.3.2 Live Loads

Both the 1992 AASHTO Standard Specifications for Highway Bridges and the 2007 AASHTO LRFD Bridge Design Specifications indicated that for culverts with less than 0.6 m of fill, the soil does not distribute the wheel load considerably and suggested that the culvert should be designed as a direct slab for fill depths of less than 0.6 m. Several researchers have expressed concerns about the inconsistencies of this assumption and indicated that the AASHTO guidelines greatly underestimate actual soil pressures (Tadros et al. 1989, Abdel-Karim et al. 1990, Yang et
al. 1999). The provision for the distribution of the live load was also changed from the 1992 AASHTO Standard specifications for Highway Bridges to the 2007 AASHTO LRFD Bridge Design Specifications. Both AASHTO guidelines use the equivalent strip method for a fill depth of less than 0.6 m. Article 3.24.3.2 of the 1992 AASHTO standard specifications provided a single equation (Equation 2.5) for the distribution width, B, for a single wheel load on the top slab of a box culvert for a fill depth less than 0.6 m. A distribution width of 2B is used for the calculation of pressure for the axle load (McGrath et al., 2005).

\[
B = 1.2m + 0.018S \leq 2.1m \quad \text{for } H < 0.6 \text{ m} \quad \text{Equation 2.5}
\]

where B is the distribution width,
S is the effective span length in meters.

The 2007 AASHTO LRFD Bridge Design Specifications on the other hand provided a separate distribution width for the positive moment and negative moment as shown in Equations 2.6 and 2.7 respectively. Table 4.6.2.1.3-1 of the 2007 AASHTO LRFD Bridge Design Specifications provides distribution widths for an axle load, These widths are dependent on the span of the culvert:

\[
B = 0.65m + 1.98S \quad \text{for positive moment} \quad \text{Equation 2.6}
\]

\[
B = 1.2m + 0.9S \text{ for negative moment} \quad \text{Equation 2.7}
\]

The above equations are valid only for a span less than or equal to 4.5 m. For box sections with spans greater than 4.5 m, Article 4.6.2.3 of the 2007 AASHTO LRFD Bridge Design Specifications provides the distribution width which depends on the span of the box and the total length of the bridge as shown in Equation 2.8:
where $W_f$ = the modified edge-to-edge width of the bridge in meter, which is limited to a maximum value of 18 m for multiple-lane loading and 9 m for single-lane loading.

Calculations using the above equations for a fill depth less than 0.6 m show that the 2007 AASHTO LRFD Bridge Design Specifications for spans greater than 4.5 m gave similar distribution widths to the 1992 AASHTO Standard Specifications for Highway Bridges. However, for span lengths less than 4.5 m the AASHTO LRFD Bridge Design Specifications gave smaller distribution widths. Therefore, the AASHTO LRFD Bridge Design Specifications are more conservative than the AASHTO Standard Specifications for Highway Bridges for spans less than 0.3 m (McGrath et al. 2005).

According to the 2007 AASHTO LRFD Bridge Design Specifications, for fill depths of 0.6 m and greater, a wheel load acts over the tire footprint area of 0.5 m x 0.25 m. The wheel load distribution on the culvert is calculated by increasing the tire footprint dimensions by 1.15 times the depth of fill for a select granular backfill and by 1.0 times the depth of fill for all other soil types. On the other hand, the 1992 AASHTO Standard Specifications for Highway Bridges considered the wheel load acting as a point load and distributing it over a square area with a side dimension of 1.75 times the depth of fill. When the fill depth increases to the depth that the distribution areas from two or more tires overlap, the interacting tire loads are added together to calculate the total load, which is considered to be uniformly distributed over the area defined by the outside limits of the individual areas. Estimated pressures on the culvert will be higher when using the 2007 AASHTO LRFD Bridge Design Specifications than when using the 1992 AASHTO Standard Specifications for Highway Bridges and the difference in pressures is greater for shallow depths.

The 2011 KDOT Bridge Design Manual recommended the design wheel load distribution width on an under-fill structure with a fill depth of less than 0.6 m be the greater of Equation 2.5 and 1.75 times the fill depth.

Seed and Raines (1988) provided an equivalent line load equation to determine the axle load for a two-dimensional finite element analysis. Tadros and Benak (1989), Abdel-Karim et al.
(1990), and Awwad (2000) believed the AASHTO square area distribution to be conservative. They also agreed that beyond a fill depth of 3 m the truck load became negligible when compared to the earth pressure loads. Abdel-Karim et al. (1990) suggested including the distributive effect of the road bed. They recommend considering flexible pavements as just additional fill depth. For rigid pavement structures the load can be distributed through the pavement according to Boussinesq’s equations. Another option was to develop an equivalent depth for rigid pavements (Abdel-Karim et al. 1990).

Lawson et al. (2010) pointed out that possible methods to minimize over-conservatism in load ratings include more accurate modeling of the distribution of applied loads through finite element analysis or Boussinesq’s equations by considering the load-distribution effects of pavement stiffness.

Awwad et al. (2000) carried out a finite element analysis of culverts with different sizes under fill depths ranging from 0 to 3 m at an interval of 0.6 m. They concluded that the wheel load effect was dominant within the fill depth of 0.9 m. Both the soil weight and wheel load affected the culvert response when the fill depth was between 0.9 to 2.1 m. They further indicated that the effect of the wheel load was negligible for fill depths of more than 2.1 m and found that the effect of lateral pressure contributing to a reduction in the maximum positive moment on the top slab decreased with an increase in the span of the culvert.

Bloomquist and Gutz (2002) carried out research in relation to the distribution of live load through earth fill and reported the development of equations for calculating the distribution of wheel loads through fill above the precast concrete box culvert. They aimed at devising a new distribution method and developing a single design equation for the live load distribution on the top slab of the precast concrete box culvert.

Article 3.20.3 in the 1992 AASHTO Standard Specifications for Highway Bridges required inclusion of a surcharge equivalent to an additional 0.6 m of fill when estimating the lateral load on the culvert to account for the near-structure lateral live load due to approaching vehicles. This load is constant regardless of the number of trucks that pass over the culvert. However, this load is not considered in the reduced load case while the maximum positive moments on the top and bottom slabs are calculated. The 2011 KDOT Bridge Design Manual
also uses a lateral live load surcharge pressure equal to 0.6 m of earth fill for all culvert structures.

Although AASHTO has provided the guidelines for load rating, some state DOTs still have a tendency to use their own methods and policies for load rating of culverts. Lawson et al. (2010) carried out a survey in 2009 to find the policies used by different states for load rating of culverts. The survey revealed that 21 out of 32 states who completed the survey calculated loads following the AASTHO guidelines. Table 2.3 shows the distribution of the load application policies used by the responding states to load-rate concrete box culverts. Among these 32 states, 15 states incorporate soil-structure interaction and only 7 states consider the effects of varying soil conditions. Two of the responding states that claimed to use the load application on culverts per the AASHTO specifications had different answers to the questions about whether their procedure accounted for soil-structure interaction and varying soil conditions. This fact implies the existence of some level of confusion regarding how to follow the AASHTO specifications. However, this problem is not a new one. A similar survey carried out by Tadros et al. (1987) found the variations in the loading policy from state to state. They also raised the question about the adequacy of the AASHTO provisions. Because of the confusion created by the existence of the AASHTO standard specifications and the LRFD specifications some state DOTs had two different loading guidelines (Iowa DOT 2005).

<table>
<thead>
<tr>
<th>TABLE 2.3</th>
<th>Load Application Policies Used by Different State DOTs</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO</td>
<td>Custom</td>
</tr>
<tr>
<td>21</td>
<td>9</td>
</tr>
</tbody>
</table>

(Source: Lawson et al. 2010)

2.4 Influence Factors for Load Rating

Multiple independent variables affect the load rating of culverts. Some of them have significant effects and some do not cause any considerable change on the overall Inventory Rating and Operating Rating. Soil modulus, depth of fill, modulus of subgrade reaction, and Poisson's ratio are among the important parameters that should be supplied as inputs to the
numerical models used to analyze the culvert to determine live load and dead load demands. Therefore, knowledge about the sensitivity of different input parameters would be beneficial to load rating engineers. Variability in the load rating is not limited to the input parameters only. There are many analytical tools available in practice with different levels of model sophistication. Different analytical tools yield different rating values for same parameters. The numerical models with a lower level of model sophistication are more conservative than those with a higher level of sophistication. Thus the rating of a culvert is also dependent on the level of effort expended during the load rating calculation. Lawson et al. (2010) carried out a regression analysis between six independent variables (modulus of subgrade reaction, Poisson's ratio, multi-barrel effects, lateral earth pressure, modulus of elasticity, and depth of fill) and the actual inventory ratings for seven selected culverts in Texas. These culverts had been designed and constructed at different times using the design philosophy prevailing at that particular design era. Their analysis showed that the depth of the fill was the most significant parameter in the load rating calculation and found that there was no significant relationship between the number of spans, the barrel height or the span length and the load rating. However, the relationship between the load rating and the depth of fill showed higher load ratings for greater fill depths. Some of the findings from their work are discussed below.

2.4.1 Modulus of Subgrade Reaction

Modulus of subgrade reaction, k, is the measure of the soil support to the bottom slab of a culvert. The analysis using k values of 20.4, 40.7, and 67.9 MN/m$^3$ showed that the change in the k value had little effect on the inventory rating.

2.4.2 Poisson's Ratio

Poisson's ratio is the ratio between the transverse strain and longitudinal strain. The analysis using Poisson's ratios of 0.5, 0.3 and 0.1 for a soil modulus of 138 MPa showed that the slope of the change in the load rating with respect to Poisson's ratio was small (i.e., less than 10% change across the range of Poisson's ratio). This result showed that the inventory rating was not sensitive to Poisson's ratio. For most of the cases, a typical Poisson’s ratio of 0.3 provided
suitable results. However, if tall culverts were backfilled with poor materials like highly plastic clays, the sensitivity of Poisson's ratio increased.

### 2.4.3 Multi-barrel Effects

Lawson et al. (2010) compared the inventory ratings of five, six, and seven barrel culverts with those of four barrel culverts. The results showed a small change in the load rating with respect to the number of barrels. The percent difference between the inventory ratings was less than 10%. This result also showed that the inventory rating for four barrel culverts was the lowest and most conservative.

### 2.4.4 Lateral Earth Pressure

Lateral earth pressures acted on the end walls of the culverts. The soil was modeled as an equivalent fluid and the distribution of the lateral earth pressure was triangular. The equivalent fluid weight depended on the soil properties and the stress history of the soil. The inventory ratings were determined using lateral earth pressure values ranging from 6.3 kN/m$^3$ to 18.9 kN/m$^3$ at 3.15 kN/m$^3$ increments. It was shown that the inventory rating was not very sensitive to the lateral earth pressure for typical culverts. However, it was sensitive to the lateral earth pressure for tall culverts because the critical section moved to the mid-spans of the exterior wall. Lawson et al. (2010) considered the 2007 AASHTO LRFD Bridge Design Specifications requirement for the lateral earth pressure to be logical and reasonable.

### 2.4.5 Modulus of Elasticity

The analysis carried out using a modulus of elasticity ranging from 27.5 to 275 MPa at increments of 27.5 MPa showed that the culverts with a greater depth of fill were more sensitive to the elastic modulus. This finding suggested the importance of accurately determining the elasticity of the soil for deeper culverts. They recommended measuring the elastic modulus of the soil with a precision of ±1.4 MPa for fill depths greater than 1.8 m and ±6.9 MPa for smaller depths. The modulus of elasticity significantly affected the inventory rating, especially at higher fill depths. Since soil is highly variable and its strength is dependent on the stress level and time,
the selection of soil modulus for culvert load-rating purposes can induce higher uncertainty into the load rating process.

2.4.6 Depth of Fill

The depth of fill represents the overburden pressure acting on the top of the culvert. Culverts are considered being subjected to direct traffic when the depth of fill is less than 0.6 m. Inventory ratings are higher at minimum and maximum depths. For intermediate depths the rating is between the minimum and maximum ratings. Lawson et al. (2010) revealed that the highest rating occurred at the maximum design depth. Although a culvert's rating factor was found to be greater than one for deep fills, it was not necessarily greater than one for low fills. However for shallow fills the dead load became relatively small and the traffic load became dominant as a result of the minimum distribution of live loads.

Lawson et al. (2010) regarded the depth of fill and the elastic modulus of the soil as "very sensitive" to the inventory rating and other parameters as “not sensitive”.

2.5 Constitutive Models of Soil

A constitutive model, also referred to as constitutive equation, is a mathematical approximation of the stress-strain behavior of a material. A constitutive model is an essential and important part of finite element and finite difference analyses. Since the stress-strain behavior of soil is dependent on many factors such as stress level, soil type, saturation condition, level of compaction, and others, a number of soil constitutive models have been developed and are available for finite element/difference analyses. Lade (2005) summarized a number of available constitutive models. One constitutive model cannot represent all soil behavior; however each model captures part of important behavior for a particular type of soil. Simulating the response of a buried structure to live loads acting on the surface in a finite element/difference analysis requires a soil constitutive model that best captures the soil-culvert interaction. Linearly elastic soil models have been used by many researchers in their studies (for example, Moore and Brachman 1994, Fernando and Carter 1998, NCHRP 2010). Nonlinear models, such as elastic-plastic soil modes, have also been used by researchers (e.g., Pang 1999). Stress-dependent
stiffness and shear failure have been found to be important characteristics for an analysis using soil models. The Duncan-Chang hyperbolic model has such features (Selig 1988), and has been implemented in the finite element programs CANDE and SPIDA to analyze soil-structure interaction problems for culverts.

For computationally intensive 3D models it becomes important to select the simplest soil model that is suitable for the soil-structure interaction problem being analyzed. The linearly elastic model and the Mohr-Coulomb model have been used in soil-structure interaction analyses of buried culverts (NCHRP 2010, Lawson et al. 2010). The linearly elastic model provides a reasonable model of basic soil behavior; however, it does not consider nonlinear stress-strain behavior or plasticity at failure. For linearly elastic isotropic soil behavior, there are four elastic constants (modulus of elasticity, Poisson’s ratio, bulk modulus, and shear modulus); however, only two (it can be any two) of the four constants are independent. In reality, these elastic constants vary with stress level. Some analyses use elastic properties that vary with depth (for example, NCHRP 2010). The stress-strain relationship of a linearly elastic isotropic model can be expressed as Equation 2.9. An elastic model simulates recoverable deformation of soil; however, complicated soil behavior cannot be captured by the elastic model in most cases.
\[ \varepsilon_{11} = \frac{1}{E} [\sigma_{11} - \nu (\sigma_{22} + \sigma_{33})] \]

\[ \varepsilon_{22} = \frac{1}{E} [\sigma_{22} - \nu (\sigma_{11} + \sigma_{33})] \]

\[ \varepsilon_{33} = \frac{1}{E} [\sigma_{33} - \nu (\sigma_{11} + \sigma_{22})] \]

Equation 2.9

\[ \varepsilon_{12} = \frac{\sigma_{12}}{2G} \]

\[ \varepsilon_{13} = \frac{\sigma_{13}}{2G} \]

\[ \varepsilon_{23} = \frac{\sigma_{23}}{2G} \]

where \( \sigma_{11}, \sigma_{22}, \) and \( \sigma_{33} \) = the principal stresses in three directions,

\( \varepsilon_{11}, \varepsilon_{22}, \) and \( \varepsilon_{33} \) = the principal strains in three directions,

\( \varepsilon_{12}, \varepsilon_{13}, \) and \( \varepsilon_{23} \) = the shear strains in three directions,

\( G \) = the shear modulus,

\( \nu \) = Poisson’s ratio.

The Mohr-Coulomb model (also known as the linearly elastic perfectly plastic model) is one of the simplest elastoplastic models. In this model, Mohr-Coulomb's yield criterion and a non-associated flow rule for shear failure are used. Equation 2.10 shows the simple form of Mohr-Coulomb's yield criterion:

\[ \tau = c + \sigma \tan(\phi) \]

Equation 2.10

where \( \tau \) and \( \sigma \) = the shear stress and normal stress on the plane, on which a slip is initiated,
c and $\varnothing = \text{respectively the cohesion and the internal friction of the soil.}$

In terms of maximum and minimum principal stresses, the Mohr-Coulomb yield criterion can be expressed as follows:

$$\frac{\sigma_1 - \sigma_3}{2} = \frac{\sigma_1 + \sigma_3}{2} \sin(\varnothing) + c \cos(\varnothing)$$

Equation 2.11

The Mohr-Coulomb model is effective in modeling shear strengths of soils and rocks. However, for this model the soil elasticity constants are independent of the stress level, which can lead to significant inaccuracies in modeling results.

NCHRP (2010) carried out a detailed study on live load distribution for buried structures using the most frequently adopted software used in practice for culvert modeling. Soil-structure interaction, sequential model development, structure/soil interface modeling, 3D analysis, structural analysis capabilities, and built-in soil models were the major criteria for selecting the software to use in their study. The Fast Lagrangian Analysis of Continua in 3 Dimensions (FLAC3D) was selected since it met most of their requirements. FLAC3D is a finite difference program, which was developed for simulating three-dimensional geotechnical engineering problems and is suitable for solving nonlinear and large displacement problems. This program has 11 built-in constitutive models for modeling various types of geomaterials and can model reinforcement and structural features.

The NCHRP (2010) study analyzed over 830 3D models of culverts including box culverts. One example of the model is shown in Figure 2.11. The Mohr-Coulomb model for soils was used. However, the elastic model was used for the top thin layer to prevent the failure of the soil under the wheel load and it was also used for the pavement.
2.6 Summary

This chapter reviewed culvert classification, load rating of culverts, AASHTO guidelines for load distribution, influence factors in load rating, and constitutive models of soils. Conclusions from this literature review can be summarized below:

1. Vertical stresses due to soil weight on the top slab are higher for embankment installation culverts and lower in trench installation and imperfect trench installation culverts than the overburden stress of the soil would be if no culvert was present. This increase/decrease in vertical stresses due to the soil loads are significant for newly installed culverts. This effect gradually decreases with the age of the culvert and can be ignored in old culverts.

2. Live load and dead load demands, and hence the load rating, of the culvert are dependent on the level of model sophistication used in analyzing these demands. A higher level of model sophistication yields a higher load rating. Therefore a
higher level of model sophistication can capture the soil-structure interaction behavior more precisely than the lower one.

3. AASHTO Standard Specifications for Highway Bridges and AASHTO LRFD Bridge Design Specifications provided the guidelines for load distribution over culverts under dead and live loads. These guidelines do not consider the effect of a pavement present over the fill, which is a controlling case in design considering the loads during construction. However, the actual stress distribution on the culvert under the pavement, which is the case in load rating, may not be truly represented by the current AASHTO distribution.

4. Load rating is particularly sensitive to depth of fill and soil modulus.

5. FLAC3D finite difference modeling technique can simulate the three-dimensional load distribution in culverts. The elastic model and Mohr-Coulomb model can be used for modeling soils around culverts.
Chapter 3: Field and Laboratory Tests

This chapter presents the results from two field tests carried out on concrete box culverts under rigid and flexible pavements. Field and laboratory tests were conducted to characterize the pavement layers and the natural subgrade soils in the field. The field tests were conducted on two low-fill box culverts using a loaded truck consisting of a truck with a low boy trailer carrying a backhoe. The soil, concrete, and hot mix asphalt (HMA) samples taken from the field were tested in the laboratory at the University of Kansas to determine their physical and mechanical properties.

3.1 Statistical Study of Culverts

A statistical study of box culverts in Kansas having a span greater than 3.6 m and a fill depth up to 0.6 m was carried out before the selection of the test culverts. Three hundred and five culverts met these criteria. Figure 3.1 shows that box culverts used by KDOT had two common spans of 3.6 and 4.2 m. Figure 3.2 shows the number of individual boxes with different spans. Since some of the box culverts had more than one box, the numbers of boxes and culverts are different. Although the number of box culverts with 4.2 m span boxes was slightly more than the number with 3.6 m span boxes in Figure 3.1, 3.6 m span boxes outnumbered 4.2 m span boxes in Figure 3.2. Figure 3.3 shows that the two most common fill depths above these culverts were 0.3 and 0.6 m. For simplicity, single span culverts with simple geometry were selected for testing. After field visits to several possible culverts in Kansas, the culverts on the US50 Highway and the KS148 (All American Drive) were selected. Both culverts had the fill depth of nearly 0.6 m. The culvert on the US50 was under a rigid pavement while that on KS148 was under a flexible pavement. Their box spans are 3.6 and 5.4 m, respectively.
FIGURE 3.1
Span Distribution of Low-Fill Culverts in Kansas

FIGURE 3.2
Span Distribution of Low-Fill Boxes in Kansas
3.2 Field Test on Box Culvert under Rigid Pavement

3.2.1 Site Condition

The culvert selected for the field test was a single span reinforced concrete box culvert, which is located at milepost 399 on US 50 Highway, west of Emporia, Kansas. The culvert was a rigid frame box (RFB). The cross section of the culvert and the interior of the culvert are shown in Figures 3.4 and 3.5 respectively. The inside dimensions of the culvert were 3.6 m wide and 3 m high. The culvert was aligned perpendicularly to the highway. The fill height up to the riding surface of the concrete pavement was 0.65 m from the top of the culvert roof. The overall length of the culvert was 27.6 m, of which 13.2 was under the concrete pavement and the concrete shoulders. The width of the concrete pavement in each lane was 3.6 m while the width of each shoulder was 3 m. The culvert extended under an unsurfaced embankment area on each side of the road. The pavement, the shoulder, and the unsurfaced area over the culvert are shown in Figure 3.6. The embankment fill was high-plasticity clay. The concrete pavement was 0.25 m thick, which was placed over a 0.1 m thick cement-treated aggregate base course. The base course was underlain by a 0.15 m thick lime-treated subgrade, which was underlain by a 0.15 m thick soil layer. The soil layer had a liquid limit of 59, plastic limit of 30, plasticity index of 29,
and specific gravity of 2.68 and was placed immediately above the culvert. The shoulder portion consisted of similar layers as the concrete pavement except the concrete shoulder thickness was 0.2 m thick. The culvert had 0.3 m wide haunches at the corners.

FIGURE 3.4
Cross Section of the Culvert and Pavement Layers
FIGURE 3.5
Interior of the Test Culvert

FIGURE 3.6
Concrete Pavement, Concrete Shoulder, and Unsurfaced Sections Over the Culvert
3.2.2 Test Devices and Instrumentations

A series of test devices and instruments were used during the field test to evaluate the performance of the culvert under different loading conditions. One-half of the culvert was instrumented with displacement transducers and earth pressure cells under the eastbound portion of the highway. Displacement transducers were used to measure the vertical deflections of the culvert roof slab while the pressure cells were used to measure the vertical pressures on the culvert.

3.2.2.1 Displacement Transducers

The displacement transducers used in this research were strain gauge-type sensors manufactured by Tokyo Sokki Kenkyujo, Co., Ltd., Japan. They had two displacement ranges: 0 to 100 mm (Model: CDP-100) and 0 to 50 mm (Model: CDP-50). The accuracy of the transducers was 0.01 mm. The locations of the displacement transducers are shown in Figure 3.7. Three displacement transducers with a 100 mm range, labeled as L1, L2 and L3, were installed under the pavement section. Two displacement transducers with a 50 mm range, labeled as L4 and L5, were installed under the shoulder and the unsurfaced section respectively. Displacement transducers L2, L4 and L5 were positioned below the center of the pavement, the shoulder, and the unsurfaced sections respectively along the culvert axis. Displacement transducer L3 was also installed along the same axis but was directly below the outer wheel of the test truck during loading. Displacement transducer L1 was the only transducer placed at the quarter span to monitor the deflection along the transverse direction. More displacement transducers were installed under the pavement section because it was the main focus of this study. Metal frames of approximately 2.8 m height were used to support and fix the displacement transducers in position as shown in Figure 3.8. The frames were stabilized by sand bags at the base.
3.2.2.2 Earth Pressure Cells

The earth pressure cells used in this research were strain gauge-type soil pressure gauges and were manufactured by Tokyo Sokki Kenkyujo Co., Ltd. in Japan. They had two capacity ranges: 200 kPa (Model: KDE-200KPA) and 500 kPa (Model: KDE-500KPA). The pressure cells were made of stainless steel and can work at a temperature range from -20°C to 60°C. Each
cell had a thickness of 11.3 mm, an outer diameter of 50 mm with a sensing area diameter of 46 mm, and a total weight of 160 g. It had minute displacement of pressure-sensitive area due to a double diaphragm structure and nonlinearity of 1% RO (random occurrence). These pressure cells are suitable for measuring earth pressure under dynamic loading.

Four earth pressure cells were installed within the fill above the unsurfaced portion of the culvert to measure the vertical stresses at the interface between soil and the culvert top slab. Four holes were dug at the locations shown in Figure 3.9 for the placement of the pressure cells. The pressure cells, labeled as E1, E2, E3, and E4, were placed at depths of 0.45, 0.40, 0.38, and 0.35 m respectively, due to the slope of the ground surface. Pavement and shoulder sections were not instrumented with pressure cells due to the difficulty of installation. Pressure cells E1 and E4 were placed 1.8 m apart and were intended to be below the wheels during loading. However, the actual distance between the wheels of the truck from center to center was 2 m as shown in Figure 3.9. As a result, only pressure cell E4 was directly below the wheel while E1, though it was below the wheel, was 0.15 m off the center of the wheel during loading. E2 was in the middle between E1 and E4 while E3 was in the middle between E2 and E4. Holes were filled and compacted with the same soil excavated after the placement of the pressure cells. The same amount of the soil was excavated and compacted back to the same hole to the same elevation to ensure the same density before and after the installation of pressure cells.

**FIGURE 3.9**
Schematic of the Pressure Cell Locations
3.2.2.3 Data Acquisition

A Smart Dynamic Strain Recorder DC-204R, manufactured by Tokyo Sokki Kenkyujo, Co., Ltd., Japan, was used to record the data from displacement transducers and earth pressure cells. There were four recorders used during the test. One data recorder and a computer were used to record the earth pressure cell data. The remaining three recorders were used to obtain the data from displacement transducers using the second computer. Among these three recorders one data recorder served as a master recorder and the remaining two served as slaves, which were synchronized with the master recorder during connections. Each recorder had four connection ports to strain gauge sensors. The power to the recorders and computers were supplied using batteries and inverters.

3.2.2.4 Load Scheme

A truck pulling a low-boy loaded with a backhoe was used as the test truck. The truck consisted of six physical axles: a front steering axle, middle tandem axles, and triple axles at the end. However, seven axle positions were adopted in this study. The axle configuration and the load on each axle are shown in Figure 3.10 while Figure 3.11 shows a picture of the truck. Table 3.1 shows the calculated contact area for each wheel load. The load of the front steering axle was 49 kN. The second axle from the front of the truck shown in the photo of Figure 3.11 did not touch the ground; therefore, it was not counted. The center of the tandem axles was 4.8 m from the front axle and had a 105 kN load on each axle. The center of the triple axles was 12.3 m behind the center of the tandem axles. Each axle of the triple provided 80.5 kN load. The center to center distance between wheels on opposite sides of the same axle was 2 m.
FIGURE 3.10
Axle Loads and Configuration

FIGURE 3.11
Test Truck

TABLE 3.1
Calculated Contact Area for Each Axle Load

<table>
<thead>
<tr>
<th>Axle no.</th>
<th>Axle load (kN)</th>
<th>Tire pressure (kPa)</th>
<th>Calculated wheel contact area (m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>49</td>
<td>760</td>
<td>0.064</td>
</tr>
<tr>
<td>2</td>
<td>105</td>
<td>760</td>
<td>0.138</td>
</tr>
<tr>
<td>3</td>
<td>0</td>
<td>760</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>105</td>
<td>760</td>
<td>0.138</td>
</tr>
<tr>
<td>5</td>
<td>80.5</td>
<td>760</td>
<td>0.105</td>
</tr>
<tr>
<td>6</td>
<td>80.5</td>
<td>760</td>
<td>0.105</td>
</tr>
<tr>
<td>7</td>
<td>80.5</td>
<td>760</td>
<td>0.105</td>
</tr>
</tbody>
</table>
The culvert was tested under static loading and traffic loading. The axis of the culvert was marked with color spray paint on the surface to determine the position of each axle during static loading. Also the intended lateral positions of the wheels were marked along the same line in each of the three sections as shown in Figure 3.6. Seven load combinations were obtained through applying static loading at each section by placing the six axles of the truck over the marked line in turn. One more combination was obtained by assuming one dummy axle in the middle of the tandem axle. This dummy axle also provided one more symmetric load, which will be analyzed in the numerical study to be discussed later. The numbering of each axle load combination is shown in Figure 3.10.

Static loading was first applied at each section beginning with the unsurfaced section. Desired positions of axles were achieved by guiding the truck on the points previously marked. All seven axles, including the dummy axle, were placed on the marks in turn to create seven loading positions (referred as Loads 1 to 7) as shown in Figure 3.12. Pressure and deflection readings were recorded using the data acquisition system. A similar procedure was repeated on the shoulder and the pavement for static loading. Traffic loading was applied only on the pavement section by moving the truck at three predetermined speeds: 25, 45, and 65 mph.

FIGURE 3.12
Seven Load Combinations
3.2.3 Field Test Results

The test data collected through the displacement transducers and earth pressure cells were analyzed and presented in tabular forms and graphically in this section.

3.2.3.1 Displacement Results

The measured deflections of the culvert at each displacement transducer location are shown in Figure 3.13 and Table 3.2. The deflections at L1, L2, and L3 were measured when the concrete pavement was loaded. The deflections at L4 were measured when the concrete shoulder was loaded. The deflections at L5 were measured when the unsurfaced section was loaded. Figure 3.13 shows that the deflections under the unsurfaced section (i.e., at L5) were much larger than those under the pavement and shoulder sections (i.e., at L1, L2, L3, and L4). The deflections at L2 under the pavement and L4 under the shoulder were almost the same under each axle load. This comparison implies that the concrete pavement and the concrete shoulder had similar performance. However, displacement transducer L1, placed at the quarter span of the culvert, measured the least deflections. The deflections of the culvert under the unsurfaced section were 2 to 3 times larger than those under the concrete pavement and the shoulder sections because the pavement and shoulder sections consisted of pavement layers with much higher stiffness than the natural soil in the unsurfaced section.
Figure 3.14 shows the profiles of the vertical deflections under the culvert from the centerline of the two-lane highway when the loads were applied on the unsurfaced, the shoulder, and the pavement sections, respectively. The maximum deflection for each test section occurred at the point of the wheel load. The arrows represent the locations of the axle loads. The vertical deflections under the load on the shoulder were nearly symmetric along the center of the load.

The front steering axle, the tandem axles, and the triple axles shown in the inset in Figure 3.14 had gross loads of 49, 219, and 241.5kN, respectively. These axle loads became symmetric with regard to the culvert axis at Loads 1, 3, and 6, respectively. Figure 3.15 also shows that the vertical deflection generally increased with an increase of the gross load of the axle; however, there was a slight reduction when the triple axles were applied instead of the tandem axles.

![Figure 3.14](image_url)

**FIGURE 3.14**
Deflection of Culvert Along Culvert Axis for Load on Different Section
The culvert was also tested under a moving load by driving the test truck at approximately 25, 45 and 65 mph on the pavement section. Figure 3.16 shows the maximum deflections of the culvert at three locations. The general trend of the plot shows that the deflection at location L1 decreased by a small amount from static loading to traffic loading at the speed of 25 mph. Then it increased with the increasing speed. The general trend of the deflection at Location L2 was also increasing with an increase of the speed. The larger deflection might be the accumulated deflection that is a combination of the deflection induced by the front axle followed by the rear axle. The vertical deflections at L3 decreased with increasing speed.
Figure 3.16 shows the variation of the measured vertical pressure on the top of the culvert with the axle load applied on the unsurfaced area. The measured vertical pressures at different

<table>
<thead>
<tr>
<th>Axle Load</th>
<th>Deflection (mm)</th>
<th>Pressure (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pavement</td>
<td>Shoulder</td>
</tr>
<tr>
<td></td>
<td>L1</td>
<td>L2</td>
</tr>
<tr>
<td>1</td>
<td>-0.04</td>
<td>-0.06</td>
</tr>
<tr>
<td>2</td>
<td>-0.11</td>
<td>-0.15</td>
</tr>
<tr>
<td>3</td>
<td>-0.12</td>
<td>-0.16</td>
</tr>
<tr>
<td>4</td>
<td>-0.1</td>
<td>-0.15</td>
</tr>
<tr>
<td>5</td>
<td>-0.12</td>
<td>-0.14</td>
</tr>
<tr>
<td>6</td>
<td>-0.13</td>
<td>-0.17</td>
</tr>
<tr>
<td>7</td>
<td>-0.1</td>
<td>-0.15</td>
</tr>
</tbody>
</table>

3.2.3.2 Pressure Results

Figure 3.17 shows the variation of the measured vertical pressure on the top of the culvert with the axle load applied on the unsurfaced area. The measured vertical pressures at different
locations are also summarized in Table 3.2. The maximum pressure was measured by pressure cell E4 when it was under Axle Load 4, which was approximately 29% of the tire pressure. The measured pressure was zero at Axle 3. Pressure cell E2 did not measure any pressure during loading. This result implies that the distribution of the wheel load through the soil did not reach this point, which was located at a distance of 0.9 m from the center of the wheels. Pressure cell E3 located at 0.45 m from the center of the wheels recorded lower pressures than those recorded by Pressure cells E4 and E1. Since pressure cell E4 was right below the wheels, it measured the highest pressure as compared with pressure cells E1, E2, and E3 under the same load except for Axle 5. The reason for the lower pressure measured by cell E4 at Axle 5 is not clear. The measured pressures at Axle 6 were higher than those at Axles 5 and 7 because of the influence by Axles 5 and 7.

Figure 3.18 shows the distribution of the measured pressure with distance from the wheel for different axle loads. The highest pressure developed directly beneath the wheels and decreased with an increase of the distance. The rates of pressure reduction for Loads 1 and 4 were higher than those at other axle loads. The measured pressure by cell E1 was lower than that by cell E4 because the pressure cell E1 was 0.15 m away from the center of the wheel and the fill depth at E1 was approximately 0.1 m more than that at E4 due to the sloping ground.
3.3 Laboratory Tests

After the loading test on the culvert was finished, samples of the pavement layers along with the natural backfill soil around the culvert were obtained by KDOT as shown in Figure 3.19 from three boreholes: one borehole was drilled above the culvert and two boreholes were drilled in the unsurfaced section. The first borehole was located at 0.27 m west of the axis of the box and 2.19 m south of the centerline of the highway. The second borehole was located at 3.42 m west of the culvert axis and 9.93 m south of the centerline of the highway. The third borehole was located at 1.02 m south of the second borehole. The truck used in the drilling operation is shown in Figure 3.19. The borehole above the culvert confirmed the pavement layers were constructed as presented in Figure 3.4. They consisted of 0.25 m thick concrete at the top surface, 0.1 m thick cement-treated base course, 0.15 m thick lime-treated subgrade, and 0.15 m thick natural backfill soil on the top of the culvert. Five Shelby tube samples were obtained from the boreholes. The Shelby tube obtained over the culvert recovered the lime-treated subgrade and the natural backfill soil. The remaining four Shelby tube samples, which were taken from the borehole in the unsurfaced section, recovered the natural soil samples at the depths of 1.8 m and 3.9 m. Both boreholes in the unsurfaced section were advanced to a depth of 4.5 m. Grayish brown, moist, firm silty clay fill existed within the top 1.5 m. The soil at depths from 1.5 m to
3.7 m was dark gray, moist and firm clay fill with small trace roots. The soil at depths from 3.7 m to 4.5 m was moist, firm gray clay with trace brown mottling. The Shelby tubes were capped, sealed, and labeled with borehole and sample numbers after they were taken out.

![Figure 3.19](image)

**FIGURE 3.19**
Core Drilling on the Pavement and the Truck Used in Drilling

### 3.3.1 Compressive Strength Test of Concrete Sample

The core obtained from the pavement consisted of 250 mm thick concrete and 113 mm thick cement-treated base. The cement-treated base was separated from the sample by sawing. The diameter of the concrete cylinder was measured to be 101.2 mm with the help of a vernier caliper. The concrete sample was sawed into a specimen with a length of 200 mm to maintain the ratio of sample height to diameter to approximately 2:1. The density of the concrete sample was measured to be 2243 kg/m³. This sample was tested in a compressive testing machine. The load was applied at the rate of 245 kPa per second as specified in ASTM C39/C39M-11a until the
failure of the sample. The compressive strength of the pavement concrete was found to be 12.44 MPa.

3.3.2 Compressive Strength Test of Base Material

The cement-treated base sample was separated from the pavement concrete sample by sawing. The base sample had the diameter and length of 101.2 and 113 mm respectively and a density of 1986 kg/m$^3$. The top and bottom ends of the sample were smoothened out by sulfur capping. This sample was placed in the curing room overnight and then tested in a compressive testing machine. The load was applied at the rate of 245 kPa per second as specified in ASTM C39/C39M-11a until the failure of the sample. Dial gage readings along with the corresponding load readings were taken manually during the test. The stress-strain curve of the base sample is shown in Figure 3.20. The compressive strength of the cement-treated base sample was 15.5 MPa and the elastic modulus was 180 MPa.

![Stress-Strain Curve of the Cement-Treated Base](image)

**FIGURE 3.20**
Stress-Strain Curve of the Cement-Treated Base

3.3.3 Laboratory Tests on the Backfill Soil

The undisturbed soil samples in the Shelby tubes obtained from the field were extruded using a Shelby tube sample extractor. The extrusion was carried out at a very slow rate so as to
minimize disturbance to the soil sample. The soil sample was then trimmed to a required diameter of 71 mm and a length of 142 mm for the triaxial test. The soil obtained during trimming of the sample was used to measure moisture content and carry out Atterberg limit and specific gravity tests. The moisture contents of the soil samples from different tubes were between 28 to 30%.

The Atterberg limit tests were carried out in accordance with ASTM D4318-05 on the soil obtained from trimming of the undisturbed soil sample. To determine the liquid limit of the backfill soil, a flow curve was developed by plotting the data from four liquid limit tests using the Casagrande apparatus at different moisture contents in Figure 3.21. From the flow curve the liquid limit was found to be 59. The plastic limit test determined the plastic limit of 30. Therefore, the plasticity index of the backfill soil was 29. Based on the unified soil classification system (USCS), the soil was classified as CH (high plasticity clay).

A specific gravity test was conducted in accordance with ASTM D854-10. The specific gravity of the soil was obtained to be 2.67. Specific gravity is useful in determining the soil parameters such as degree of saturation and void ratio.
Triaxial tests of the soil samples were carried out at the in-situ moisture content to determine the elastic modulus, cohesion, and friction angle of the soil. The soil samples extruded from the Shelby tubes had a diameter of 100 mm. The samples were trimmed to 71 mm in diameter and 142 mm in height. The triaxial tests were conducted on three specimens at confining pressures of 10, 45, and 80 kPa respectively. Figure 3.22 shows the sample after being sheared, and clearly shows the development of a shear plane at failure. Figure 3.23 shows the stress-strain plots obtained from these three tests. The elastic moduli of the soil samples were calculated as the secant moduli at 50% peak strength as 10.1, 9.8, and 15 MPa at the confining stresses of 10, 45, and 80 kPa respectively, with an average modulus of 12.9 MPa. The total stress envelope was drawn based on the test results as shown in Figure 3.24 and resulted in a cohesion of 29.6 kPa and a friction angle of 21°.
3.3.4 Summary of Experimental Study on Culvert under Rigid Pavement

This section summarizes the field test results on a low fill box culvert under a rigid pavement under static and moving traffic loads and the laboratory tests on the samples obtained
from the field. The data obtained from these tests will be used to verify the numerical models created in this study. The following conclusions can be drawn from the test results:

1. The deflections of the culvert under static loading varied with the magnitude and position of the axle load and the type of the test section (concrete pavement, shoulder, or unsurfaced area). The higher axle load resulted in a larger deflection of the culvert. The culvert under the unsurfaced area deformed the most while that under the pavement deformed the least. This result implies that the distributed pressure of the wheel loads through the pavement onto the culvert was lower than that through the unsurfaced area.

2. The maximum deflection happened in the mid-span of the culvert when the load was applied at that location. The deflection decreased longitudinally and transversely with a distance. This result implies a two-way slab action.

3. In general, the observed deflections were higher for moving loads than static loads.

4. The pressure cell results showed that the wheel load was distributed onto the culvert within an area. The maximum pressure occurred beneath the point of wheel loading.

5. Compressive strength tests were carried out on the samples cored from the concrete pavement and cement treated base courses. The compressive strength of the pavement concrete was 12.44 MPa, which was lower than the typical concrete compressive strength used for pavements. The cement stabilized base course had a compressive strength of 16.03 MPa and elastic modulus of 138 MPa. Both values were less than the typical values for the cement treated base course.

6. The backfill soil around the culvert was high-plasticity clay (CH), which had a liquid limit of 59, a plastic limit of 30, and a plasticity index of 29. The specific gravity of the soil was 2.67. The soil had an average elastic modulus of 12.9 MPa, cohesion of 29.6 kPa, and a friction angle of 21° as determined from the triaxial tests.
3.4 Field Test on Culvert Under Flexible Pavement

3.4.1 Site Condition

The culvert selected for the field test was a single span reinforced concrete box culvert located at milepost 68.7 on the K-148 highway over Mercer creek drainage near Barnes, Kansas. The culvert was a rigid frame box. A cross section of the culvert and a picture of the culvert are shown in Figures 3.25 and 3.26 respectively. The inside dimensions of the culvert were 5.4 m wide and 3 m high. The culvert was aligned perpendicularly to the highway. The fill depth from the riding surface of the asphalt concrete pavement to the top of the culvert roof was 600 mm. The overall length of the culvert was 10.35 m. There were two lanes of 3.3 m wide each. The culvert backfill was composed of dark brown low plasticity clay. The soil layer had liquid limit of 43, plastic limit of 20, plasticity index of 23, and specific gravity of 2.71. The hot mix asphalt (HMA) layer at the top surface was 475 mm thick, which was placed over a 125 mm thick lime-stabilized subgrade. The culvert had 300 mm wide haunches at the corners.
3.4.2 Test Devices and Instrumentations

To evaluate the performance of the culvert under the variable loadings, a series of test devices and instruments were used during the test. The culvert was instrumented with displacement transducers under the southbound lane of the highway. Only one transducer was used under the northbound lane. Displacement transducers were used to measure the vertical deflections of the culvert roof slab.

3.4.2.1 Displacement Transducers

The displacement transducers used in this test were similar to those described in Section 3.2.2. The layout of the displacement transducers is shown in Figure 3.27. Six transducers were used to capture the deflection response of the culvert under loading. Four displacement transducers labeled as L1, L2, L3, and L4 were installed along the box axis to obtain the deflection profile along the centerline of the culvert whereas transducers L2, L5, and L6 were installed along the culvert span. Five out of six transducers were installed under the southbound lane of the highway and the remaining one was placed under the northbound lane. The layout of
the displacement transducers were planned to utilize the symmetry of the culvert about the centerline of the road so that the deflection profile could be drawn for a longer length of the culvert. Also the three transducers located along a line perpendicular to the culvert axis were positioned to give a deflection profile along the span under a symmetric load about the culvert axis. Transducers L1 and L3 were located right below the wheels and L2 was located under the middle of the axle. Load was applied on both the northbound and southbound lanes. When the load was applied in the northbound lane, transducer L4 was serving as transducer L3 when the southbound lane was loaded. Similarly, the deflections measured at transducers L1 and L2 during the northbound lane loading could be considered as the deflections at the respective locations under the northbound lane while the southbound lane was loaded. Metal frames of approximately 2.8 min height were used to support and fix the displacement transducers in position as shown in Figure 3.28. The frames were stabilized by sand bags at the base.

FIGURE 3.27
Layout of Displacement Transducers
The data acquisition system used in the previous test was also used in this test.

3.4.2.2 Load Scheme

A low-boy loaded with a backhoe was used as the test truck. The truck consisted of six physical axles: a front steering axle, middle tandem axles, and triple axles at the end. However, seven load positions were adopted in this study. The axle configuration and the load on each axle are shown on Figure 3.29 while Figure 3.30 shows a picture of the truck used in this test. Table 3.3 shows the calculated contact area for each wheel load. The load of the front steering axle load was 57 kN. The center of the tandem axles was 4.8 m from the front axle and had a 98 kN load on each axle. The center of the triple axles was 12.3 m behind the center of the tandem axles. Each axle of the triple provided an 80.5 kN load. The center to center distance between wheels on two sides of the same axle was 2 m.
FIGURE 3.29
Axle Load and Configuration of the Test Truck

FIGURE 3.30
Test Truck Used in Loading the Culvert
TABLE 3.3
Calculated Contact Area for Each Axle Load

<table>
<thead>
<tr>
<th>Axle no.</th>
<th>Axle load (kN)</th>
<th>Tire Pressure (kPa)</th>
<th>Calculated wheel contact area (m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>57</td>
<td>760</td>
<td>0.075</td>
</tr>
<tr>
<td>2</td>
<td>98</td>
<td>760</td>
<td>0.129</td>
</tr>
<tr>
<td>3</td>
<td>0</td>
<td>760</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>98</td>
<td>760</td>
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</tr>
<tr>
<td>5</td>
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<td>6</td>
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<td>760</td>
<td>0.106</td>
</tr>
<tr>
<td>7</td>
<td>80.5</td>
<td>760</td>
<td>0.106</td>
</tr>
</tbody>
</table>

The response of the culvert was monitored under static and traffic loadings. Static and traffic loadings were applied on both the southbound and northbound lanes. The axis of the culvert was marked with color spray paint on the surface to facilitate the positioning of the truck for each axle loading during static loading as shown in Figure 3.31. Also the intended lateral positions of the wheels were marked along the same line in both lanes. Seven load combinations were obtained through applying static loading at each section by placing six axles of the truck over the marked line in turn. One more combination was obtained by assuming one dummy axle in the middle of the tandem axle. This dummy axle provided one more symmetric load. The numbering of each axle load combination is shown in Figure 3.29.
Static loading was first applied on the southbound lane and then on the northbound lane. Desired positions of axles were achieved by guiding the truck onto the points previously marked on the pavement. All seven axles, including the dummy axle, were placed on the marks in turn to create seven loading positions (referred as Loads 1 to 7) as shown in Figure 3.32. Pressure and deflection readings were recorded using the data acquisition systems. Later, a similar procedure was followed on the northbound lane for static loading. Traffic loading was applied on both lanes by moving the truck at six predetermined speeds: 10 to 60 mph at an increment of 10 mph.
3.4.3 Field Test Results

The test data collected through displacement transducers were analyzed and are presented below in tabular forms and graphically.

The measured deflections of the culvert at the displacement transducer locations when the load was applied on the southbound lane are shown in Figure 3.33 and Table 3.4. The deflections observed at transducers L1, L2, L3, and L5 were almost equal. However, transducer L2, which was at the middle of the axle, recorded the maximum deflection. Displacement transducer L4 installed below the northbound lane recorded the minimum deflection during the southbound lane loading. The deflection at the quarter span of the culvert was also considerably lower than the deflections at other locations.

<table>
<thead>
<tr>
<th>Axle</th>
<th>L1</th>
<th>L2</th>
<th>L3</th>
<th>L4</th>
<th>L5</th>
<th>L6</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>-0.09</td>
<td>0.04</td>
<td>-0.07</td>
<td>-0.05</td>
<td>-0.09</td>
<td>-0.08</td>
</tr>
<tr>
<td>2</td>
<td>-0.47</td>
<td>-0.41</td>
<td>-0.43</td>
<td>-0.27</td>
<td>-0.48</td>
<td>-0.41</td>
</tr>
<tr>
<td>3</td>
<td>-0.53</td>
<td>-0.53</td>
<td>-0.49</td>
<td>-0.31</td>
<td>-0.52</td>
<td>-0.41</td>
</tr>
<tr>
<td>4</td>
<td>-0.51</td>
<td>-0.51</td>
<td>-0.49</td>
<td>-0.31</td>
<td>-0.47</td>
<td>-0.35</td>
</tr>
<tr>
<td>5</td>
<td>-0.46</td>
<td>-0.44</td>
<td>-0.43</td>
<td>-0.27</td>
<td>-0.47</td>
<td>-0.4</td>
</tr>
<tr>
<td>6</td>
<td>-0.58</td>
<td>-0.58</td>
<td>-0.55</td>
<td>-0.36</td>
<td>-0.57</td>
<td>-0.45</td>
</tr>
<tr>
<td>7</td>
<td>-0.47</td>
<td>-0.47</td>
<td>-0.46</td>
<td>-0.3</td>
<td>-0.43</td>
<td>-0.31</td>
</tr>
</tbody>
</table>
The measured deflections of the culvert at the displacement transducer locations when the load was applied on the northbound lane are shown in Figure 3.34 and Table 3.5. The maximum deflection was observed at transducer L4 and the minimum deflections were observed at L6 and L1. The deflections observed at transducers L2 and L5 were almost equal.

**TABLE 3.5**
Deflections During Northbound Lane Loading

<table>
<thead>
<tr>
<th>Axle</th>
<th>L1</th>
<th>L2</th>
<th>L3</th>
<th>L4</th>
<th>L5</th>
<th>L6</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
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<td>0</td>
<td>0</td>
<td>0</td>
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</tr>
<tr>
<td>1</td>
<td>-0.01</td>
<td>-0.03</td>
<td>-0.06</td>
<td>-0.13</td>
<td>-0.02</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>-0.09</td>
<td>-0.17</td>
<td>-0.28</td>
<td>-0.46</td>
<td>-0.14</td>
<td>-0.09</td>
</tr>
<tr>
<td>3</td>
<td>-0.11</td>
<td>-0.2</td>
<td>-0.32</td>
<td>-0.54</td>
<td>-0.17</td>
<td>-0.11</td>
</tr>
<tr>
<td>4</td>
<td>-0.1</td>
<td>-0.18</td>
<td>-0.52</td>
<td>-0.46</td>
<td>-0.17</td>
<td>-0.11</td>
</tr>
<tr>
<td>5</td>
<td>-0.09</td>
<td>-0.16</td>
<td>-0.27</td>
<td>-0.6</td>
<td>-0.14</td>
<td>-0.08</td>
</tr>
<tr>
<td>6</td>
<td>-0.12</td>
<td>-0.22</td>
<td>-0.38</td>
<td>-0.6</td>
<td>-0.2</td>
<td>-0.13</td>
</tr>
<tr>
<td>7</td>
<td>-0.11</td>
<td>-0.19</td>
<td>-0.33</td>
<td>-0.53</td>
<td>-0.18</td>
<td>-0.13</td>
</tr>
</tbody>
</table>
Deflection transducer L4 was located at a distance of 1.2 m from the inner wheel of the truck when the load was applied at the southbound lane. Similarly, deflection transducer L3 was at a distance of 1.2 m from the inner wheel of the truck when the load was applied at the northbound lane. Therefore the observed deflections at those locations during southbound and northbound lane loadings were nearly interchangeable. Because of the relative locations of the displacement transducers during southbound and northbound lane loadings, it was possible to plot the deflection profile along the culvert axis even under the northbound lane. The resulting deflection profiles under each axle load during southbound lane loading are shown in Figure 3.35. While these deflection profiles were drawn, the deflections recorded at L1 and L2 were assumed to be equal to the deflections at the corresponding locations under the northbound lane for northbound loading. Axle 6 produced the maximum deflections at all locations whereas Axle 1 produced the least deflections.
The loading was symmetric about the culvert axis when Axles 1, 3, and 6 were at the marked locations. Therefore the deflections observed at L5 and L6 can be assumed to be equal to those at the corresponding locations of the symmetric half of the culvert. Under this assumption the deflection curve can be plotted as shown in Figure 3.36. However, the recorded deflections under Axle 1 loading show a curvature in the opposite direction from what would be expected.
The front steering axle, the tandem axles, and the triple axles as shown in Figure 3.37 had gross loads of 57, 196, and 241.5 kN, respectively. The axle loads were symmetric with regard to the culvert axis for Axles 1, 3, and 6, respectively. Figure 3.37 also shows that the vertical deflection generally increased with an increase of the gross load of the axle. Here the deflections were taken from southbound lane loading at L1, L2 and L3 and from northbound lane loading at L4. The deflections at L1 and L2 were equal for all the loads; therefore they are overlapping.

The culvert was also tested for a moving load by driving the test truck at speeds varying from approximately 10 mph to 60 mph with an increment of 10 mph. The moving load was applied to both lanes. The resulting maximum deflections at all the locations are shown in Tables 3.5 and 3.6 and Figures 3.38 and 3.39 for southbound lane loading and northbound lane loading respectively. The general trend of the plot shows that the deflections decreased gradually with an increase in speed from 10 to 40 mph. Little change in deflection was observed for speeds greater than 40 mph.
### TABLE 3.6
Maximum Deflections Due to Moving Load for Southbound Lane Loading

<table>
<thead>
<tr>
<th>Speed (mph)</th>
<th>L1</th>
<th>L2</th>
<th>L3</th>
<th>L4</th>
<th>L5</th>
<th>L6</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>-0.41</td>
<td>-0.41</td>
<td>-0.33</td>
<td>-0.22</td>
<td>-0.37</td>
<td>-0.28</td>
</tr>
<tr>
<td>20</td>
<td>-0.41</td>
<td>-0.36</td>
<td>-0.3</td>
<td>-0.21</td>
<td>-0.33</td>
<td>-0.25</td>
</tr>
<tr>
<td>30</td>
<td>-0.37</td>
<td>-0.34</td>
<td>-0.32</td>
<td>-0.18</td>
<td>-0.31</td>
<td>-0.24</td>
</tr>
<tr>
<td>40</td>
<td>-0.35</td>
<td>-0.37</td>
<td>-0.26</td>
<td>-0.23</td>
<td>-0.31</td>
<td>-0.25</td>
</tr>
<tr>
<td>50</td>
<td>-0.36</td>
<td>-0.36</td>
<td>-0.31</td>
<td>-0.22</td>
<td>-0.33</td>
<td>-0.25</td>
</tr>
<tr>
<td>60</td>
<td>-0.39</td>
<td>-0.41</td>
<td>-0.33</td>
<td>-0.23</td>
<td>-0.35</td>
<td>-0.27</td>
</tr>
</tbody>
</table>

### TABLE 3.7
Maximum Deflections Due to Moving Load for Northbound Lane Loading

<table>
<thead>
<tr>
<th>Speed (mph)</th>
<th>L1</th>
<th>L2</th>
<th>L3</th>
<th>L4</th>
<th>L5</th>
<th>L6</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>-0.12</td>
<td>-0.22</td>
<td>-0.23</td>
<td>-0.33</td>
<td>-0.16</td>
<td>-0.12</td>
</tr>
<tr>
<td>20</td>
<td>-0.1</td>
<td>-0.2</td>
<td>-0.21</td>
<td>-0.3</td>
<td>-0.15</td>
<td>-0.11</td>
</tr>
<tr>
<td>30</td>
<td>-0.11</td>
<td>-0.18</td>
<td>-0.24</td>
<td>-0.29</td>
<td>-0.13</td>
<td>-0.11</td>
</tr>
<tr>
<td>40</td>
<td>-0.1</td>
<td>-0.15</td>
<td>-0.19</td>
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<td>-0.12</td>
<td>-0.09</td>
</tr>
<tr>
<td>50</td>
<td>-0.1</td>
<td>-0.15</td>
<td>-0.19</td>
<td>-0.3</td>
<td>-0.17</td>
<td>-0.11</td>
</tr>
<tr>
<td>60</td>
<td>-0.1</td>
<td>-0.13</td>
<td>-0.2</td>
<td>-0.3</td>
<td>-0.12</td>
<td>-0.12</td>
</tr>
</tbody>
</table>

### FIGURE 3.38
Maximum Deflections Due to Moving Load for Southbound Lane Loading
3.5 Laboratory Tests

After the field loading test on the culvert, the samples of the pavement layers along with the natural backfill soil were obtained from two boreholes: one borehole was drilled above the culvert and another borehole was drilled along the side of the culvert. Both boreholes were drilled on the southbound lane at 1.5 m west from the centerline of the road. All the sample tubes and cores taken from the field are shown in Figure 3.40. Two asphalt cores and four Shelby tube samples were obtained during the drilling operation. Both asphalt cores were 475 mm thick. The Shelby tubes, which were pushed inside the borehole along the side of the culvert, recovered soil samples at depths of 1, 2, 3, and 4 m respectively. The borehole located along the side of the culvert was advanced to a depth of 5.9 m. Below the asphalt layer there was a dark brown gray fill containing a silty clay with some sand, asphalt, and wood particles down to a depth of 4 m. The ground water table was at the depth of 3.8 m below the pavement surface. Weathered limestone was found from 4.3 m to 4.7 m deep. From the depths of 4.7 m to 5.9 m there was brown mottled dark gray shale which was firm and moist. Hard and dense limestone was found at a depth of 5.9 m.
3.5.1 Rebound Test of Asphalt Sample

The core obtained from the pavement consisted of 475 mm thick HMA concrete. The diameter of the HMA sample was measured with a Vernier caliper and found to be 98 mm. The 475 mm high samples were sawed into the samples with a height-to-diameter ratio of 2:1. The heights of the samples were approximately 200 mm after sawing. The density of the sample measured before testing was 2138 kg/m$^3$. A rebound test was conducted to estimate the elastic modulus of the asphalt concrete. Figure 3.41 shows the dial gage arrangement and test setup for a rebound test of the HMA sample. The gage length for the deformation measurement was 150 mm. The compressive load was applied up to 5.33 kN at the rate of 0.5% strain per second. The corresponding maximum compressive stress was 690 kPa, which was close to the tire contact pressure applied by the test truck. The dial gage reading was taken immediately after the applied pressure reached the maximum value and the maximum compression was 0.114 mm. Then the applied load was released and the specimen was allowed to rebound. The dial gage reading after the rebound was 0.038 mm; therefore, the total rebound was 0.076 mm. The elastic modulus of the asphalt concrete was determined to be 1,827 MPa.
3.5.2 Laboratory Tests of Backfill Soil

The undisturbed soil samples in the Shelby tubes obtained from the field were extruded using the Shelby tube sample extractor. The soil sample was then trimmed to the required size for the triaxial test. The soil obtained during trimming of the sample was used to measure moisture content and carry out Atterberg limit and specific gravity tests. The moisture contents of the soil samples from different tubes were between 18 and 20%.

The Atterberg limit tests were carried out in accordance with ASTM D4318-05 on the soil obtained from trimming of the soil sample to determine the liquid and plastic limits. To determine the liquid limit of the backfill soil, a flow curve was developed by conducting three liquid limit tests using the Casagrande apparatus at different moisture contents as shown in Figure 3.42. From the flow curve the liquid limit was found to be 43% and the plastic limit was determined to be 20%. Therefore, the plasticity index of the backfill soil was 23. According to the USCS the soil was classified as low plasticity clay (CL).
A specific gravity test was conducted in accordance with ASTM D854-10. The specific gravity of the soil was determined to be 2.71.

Triaxial tests of the soil samples were carried out in a natural condition to determine the elastic modulus, cohesion, and friction angle of the soil. The soil sample extruded from the Shelby tubes was 100 mm in diameter. The samples were trimmed to 71 mm in diameter and 142 mm in height. The triaxial tests were conducted on three samples at confining pressures of 10, 45, and 80 kPa respectively. Figure 3.43 shows the stress-strain curves obtained from the three tests. The elastic modulus of the soil was calculated as a secant modulus at 50% of the peak strength. The elastic moduli of the soil were found to be 10.8, 9.1, and 12.3 MPa at the confining stresses of 10, 45, and 80 kPa respectively with an average value of 10.7 MPa. The total stress envelope drawn using the test results is shown in Figure 3.44. It resulted in cohesion of 44 kPa and friction angle of 22°.
FIGURE 3.43
Stress-Strain Curves for Samples Tested at Different Confining Pressures

FIGURE 3.44
Total Stress Envelope
3.5.3 Summary of Experimental Study on Culvert under Flexible Pavement

This section summarizes the field test on a low fill box culvert under a flexible pavement under static and moving traffic loads and the laboratory tests on the samples obtained from the field. The following conclusions can be drawn from the test results:

1. The deflections of the culvert under static loading varied with the magnitude and position of axle load. The higher axle load resulted in a larger deflection of the culvert.
2. The maximum deflection happened in the mid-span of the culvert when the load was applied at that location. The deflection decreased longitudinally and transversely with the distance. This result implies a two-way slab action.
3. In general, the observed deflections were smaller under moving loads than under static loads.
4. The rebound test was carried out on the HMA sample to determine its elastic modulus of 1,827 MPa.
5. The backfill soil around the culvert was a natural clay soil having a liquid limit of 43, a plastic limit of 20, and a plasticity index of 23. The specific gravity of the soil was 2.71. The backfill soil had an average elastic modulus of 10.7 MPa, cohesion of 44 kPa, and a friction angle of 22° as determined from the triaxial tests.
Chapter 4 Verification of Numerical Model

4.1 Introduction

To study the load distribution on the culverts, a numerical study was carried out using the commercial software, Fast Lagrangian Analysis of Continua in 3 Dimensions (FLAC3D). A true representation of the culvert response under loading becomes important before carrying out the study on the effects of different influence factors on load distribution. In this chapter two culverts that were tested under truck loads as presented in Chapter 3 were modeled in FLAC3D for verification.

4.2 FLAC3D

FLAC3D is a finite difference program, which is specially designed for simulating three-dimensional geotechnical engineering problems. This program is suitable for solving nonlinear and large displacement problems and has 11 built-in constitutive models for various types of geomaterials. With this program it is possible to model reinforcement and structural features along with appropriate soil models. This software also provides users the facility to create their own constitutive model, which is also called user-defined model (UDM). However, the user defined model should be programmed with C++ and compiled to a DLL file to make it work with FLAC3D.

4.3 Culvert under Rigid Pavement

4.3.1 Material Models and Parameters

The pavement-culvert system involved different materials. The pavement consisted of plain cement concrete (PCC), under which there were cement-treated base course, lime-treated subgrade, and a natural subgrade above the top slab of the culvert. The culvert was made of reinforced concrete. The backfill around the culvert was a natural soil. The dimensions of the culvert and pavement layers were described in Chapter 3. All the layers were modeled as elastic materials with different properties.

The compressive strength of the concrete pavement as determined in the laboratory was 12.44 MPa. This strength value was low compared to a typical value of 15 to 40 MPa (Sidney et
al. 2003). The lower strength of the test sample might be attributed to possible sample damage during coring and boundary effects. The typical concrete pavement design strength used by KDOT is 26.9 MPa. The compressive strength of the concrete can be used to determine its elastic modulus using the correlation in Equation 4.1 as given by ACI 318-11 in Section 8.5:

$$E_c = 57000 \sqrt{f'_c}$$

where \(E_c\) = elastic modulus of the concrete in psi; 
\(f'_c\) = compressive strength of the concrete in psi.

The calculated elastic modulus of the concrete based on the typical concrete pavement design strength by KDOT was 24,545 MPa, which was adopted for the numerical analysis. In addition, Poisson's ratio of concrete of 0.15 was used in the model verification.

The elastic modulus of the cement-treated base course as described in Chapter 3 was 180 MPa, which was much lower than the typical value of 3,450 to 6,900 MPa for cement aggregate mixture provided by the 1993 AASHTO Guide for Design of Pavement Structures. The lower modulus of the test sample might also be attributed to possible sample damage during coring and boundary effects. Therefore, the average elastic modulus of 5171 MPa based on the typical value was used in the verification of the model with a Poisson's ratio of 0.3.

The 1993 AASHTO Guide for Design of Pavement Structures also suggested a typical value of elastic modulus for lime-treated subgrade ranging from 138 to 483MPa. Therefore, an average value of 310 MPa was adopted for the verification of the numerical model. Poisson's ratio of 0.3 was used for the lime-treated subgrade. The average elastic modulus of the soil obtained from the triaxial test was 12.9 MPa, which was used for the unsurfaced section including the natural subgrade and the backfill soil. A Poisson's ratio of 0.3 was used for soils.

It was assumed that the concrete for the box culvert had a typical compressive strength of 31 MPa. Considering 1% steel reinforcement with an elastic modulus of 29,000 MPa, the elastic modulus of the reinforced concrete was determined to be 27,580 MPa. Poisson's ratio of the
Concrete was assumed at 0.15. The summary of the material properties used in the numerical analysis is listed in Table 4.1.

Axles 1, 3 and 6 provided symmetric loads with respect to the culvert axis. Therefore, they were used to verify the numerical model and their computed deflections will be compared with those from the field study.

<table>
<thead>
<tr>
<th>TABLE 4.1</th>
<th>Summary of the Material Properties Used in the Model Verification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material</td>
<td>Elastic modulus (MPa)</td>
</tr>
<tr>
<td>Pavement concrete</td>
<td>24,545</td>
</tr>
<tr>
<td>Cement treated base course</td>
<td>5,171</td>
</tr>
<tr>
<td>Lime treated subgrade</td>
<td>310</td>
</tr>
<tr>
<td>Natural subgrade and backfill soil</td>
<td>12.9</td>
</tr>
<tr>
<td>Culvert concrete</td>
<td>27,580</td>
</tr>
</tbody>
</table>

4.3.2 Numerical Meshes and Boundary Conditions

Only half of the culvert was modeled to take advantage of the symmetrical condition as shown in Figure 4.1. Unyielding foundation conditions were assumed for the model; therefore the vertical movement at the bottom of the culvert was restricted. All vertical boundaries were restrained for horizontal movement except for the free boundary of the box culvert in the y direction. The horizontal displacement of the culvert was restrained at the symmetry plane (yz plane). Rollers were used at the symmetrical boundary, which allowed vertical movement but restrained horizontal movement. The boundary conditions of this model are shown in Figure 4.2.

The length, width, and height of the model were 28 m, 6.75 m, and 4.15 m respectively. The numbers of zones were 123,436, 117,088 and 119,986 when Axle loads 1, 3, and 6 were applied in the model respectively. The reason for the difference in the number of zones for different axles is that each axle had a different load contact area, which required changing the size of the mesh on the surface where the load was applied. Finer zones were used near the culvert and the zone density gradually decreased away from the culvert. The load was applied as a pressure on the surface of the pavement, the shoulder, and the unsurfaced area in turn. The number and size of zones required to apply the pressure were calculated as shown in Table 4.2 so
that the total wheel load was equal to that in the field test while the contact pressure was close to that in the field test.

FIGURE 4.1
FLAC3D Model of the Culvert
When Axle 1 was applied in the model, each wheel only had two zones due to the symmetry condition as shown in Figure 4.3. Axle 3 did not have its own load. However, when Axle 3 was in place, the wheel loads used for Axles 2 and 4 were applied on the culvert. Due to the symmetry, only one of the wheel loads was applied in the model as shown in Figure 4.4. Similarly, when Axle 6 was on place, the wheel loads for Axles 5 and 7 were also applied on the

**TABLE 4.2**
Calculation of Pressure and Number of Zones to Apply Pressure

<table>
<thead>
<tr>
<th>Axle No.</th>
<th>Single wheel load (kN)</th>
<th>Actual tire pressure in field (kPa)</th>
<th>Contact area (m²)</th>
<th>Area of each zone (m²)</th>
<th>Number of zones</th>
<th>Applied pressure in model (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>24.5</td>
<td>760</td>
<td>0.032</td>
<td>0.010</td>
<td>4</td>
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<td>760</td>
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<tr>
<td>4</td>
<td>52.5</td>
<td>760</td>
<td>0.069</td>
<td>0.016</td>
<td>4</td>
<td>840</td>
</tr>
<tr>
<td>5</td>
<td>40.25</td>
<td>760</td>
<td>0.052</td>
<td>0.013</td>
<td>4</td>
<td>805</td>
</tr>
<tr>
<td>6</td>
<td>40.25</td>
<td>760</td>
<td>0.052</td>
<td>0.013</td>
<td>4</td>
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<tr>
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<td>760</td>
<td>0.052</td>
<td>0.013</td>
<td>4</td>
<td>805</td>
</tr>
</tbody>
</table>
culvert. However, half of the load from Axle 6 and the full load from Axle 5 or 7 were applied to the model as shown in Figure 4.5.

**FIGURE 4.3**
Axle 1 Load Applied as Pressure on the Top Surface of the Pavement

**FIGURE 4.4**
Axle 3 Applied on the Top Surface of the Pavement
4.3.2 Deflections of Culvert Top Slab

The measured deflections at Locations L1 through L5 were compared with the computed ones from FLAC3D. Table 4.3 and Figure 4.6 show the measured deflections (a negative sign represents a downward deflection) compared with the computed ones from FLAC3D when Axle 1 was applied on different test sections. Figure 4.6 (a) shows that the measured deflections at all the locations reasonably matched the computed ones from FLAC3D when Axle 1 was applied on the unsurfaced section. The difference in their deflections was more significant near the point of the load application and gradually decreased with an increase of the distance. Similarly, Figure 4.6 (b) shows a reasonable similarity between measured and computed deflections when Axle 1 was applied on the shoulder. Figure 4.6 (c) shows even more similarity between the measured and computed deflections when Axle 1 was applied on the pavement section. Overall, the computed deflection profiles had similar shapes to the measured ones but the measured deflections were larger or smaller than the computed ones depending on the test sections. Their differences were most obvious when Axle 1 was applied on the shoulder and was smallest when
Axle 1 was applied on the concrete pavement. Both the measurement and the numerical methods had their maximum deflections when Axle 6 was applied on the unsurfaced section.

**TABLE 4.3**  
**Measured and Computed Deflections under Axle 1 on Different Test Sections**

<table>
<thead>
<tr>
<th>Transducer location</th>
<th>Load on unsurfaced section</th>
<th>Load on shoulder</th>
<th>Load on pavement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Measured (mm)</td>
<td>Computed (mm)</td>
<td>Measured (mm)</td>
</tr>
<tr>
<td>L1</td>
<td>0</td>
<td>-0.01</td>
<td>0</td>
</tr>
<tr>
<td>L2</td>
<td>0</td>
<td>0</td>
<td>-0.01</td>
</tr>
<tr>
<td>L3</td>
<td>-0.01</td>
<td>0</td>
<td>-0.03</td>
</tr>
<tr>
<td>L4</td>
<td>-0.04</td>
<td>-0.01</td>
<td>-0.06</td>
</tr>
<tr>
<td>L5</td>
<td>-0.25</td>
<td>-0.14</td>
<td>-0.01</td>
</tr>
</tbody>
</table>

(a) Unsurfaced section
Table 4.4 and Figure 4.7 show the measured deflections and the computed ones from FLAC3D when Axle 3 was applied on different test sections. Axle 3 resulted in higher deflections than Axle 1. Figures 4.7(a), (b), and (c) show the measured and computed deflections when Axle 3 was applied on the unsurfaced, shoulder, and pavement sections. The comparisons between the measured and computed deflections are similar to those in Figure 4.6.

<table>
<thead>
<tr>
<th>Transducer Location</th>
<th>Load on unsurfaced section</th>
<th>Load on shoulder</th>
<th>Load on pavement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Measured (mm)</td>
<td>Measured (mm)</td>
<td>Measured (mm)</td>
</tr>
<tr>
<td></td>
<td>FLAC3D (mm)</td>
<td>FLAC3D (mm)</td>
<td>FLAC3D (mm)</td>
</tr>
<tr>
<td>L1</td>
<td>0</td>
<td>-0.04</td>
<td>-0.06</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>-0.06</td>
<td>-0.12</td>
</tr>
<tr>
<td>L2</td>
<td>0</td>
<td>-0.05</td>
<td>-0.07</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>-0.13</td>
<td>-0.16</td>
</tr>
<tr>
<td>L3</td>
<td>-0.01</td>
<td>-0.08</td>
<td>-0.13</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>-0.25</td>
<td>-0.13</td>
</tr>
<tr>
<td>L4</td>
<td>-0.03</td>
<td>-0.18</td>
<td>-0.13</td>
</tr>
<tr>
<td></td>
<td>-0.02</td>
<td>-0.25</td>
<td>-0.05</td>
</tr>
<tr>
<td>L5</td>
<td>-0.47</td>
<td>-0.03</td>
<td>-0.01</td>
</tr>
<tr>
<td></td>
<td>-0.4</td>
<td>-0.02</td>
<td>0</td>
</tr>
</tbody>
</table>

The deflection caused by Axle 6 was found to be similar to the deflection caused by Axle 3 with Axle 3 producing little more deflection. Table 4.5 and Figure 4.8 show the measured and
computed deflections when Axle 6 was applied on different test sections. Their deflections were similar but slightly less than those when Axle 1 was applied on the same test section.

**FIGURE 4.7**
Measured and Computed Deflections under Axle 3 Applied at Different Test Sections
FIGURE 4.8
Measured and Computed Deflections under Axle 6 Applied on Different Test Sections
TABLE 4.5
Deflections Under Axle 6 Applied at Different Test Sections

<table>
<thead>
<tr>
<th>Transducer Location</th>
<th>Load on unsurfaced section</th>
<th>Load on shoulder</th>
<th>Load on pavement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Measured (mm)</td>
<td>FLAC3D (mm)</td>
<td>Measured (mm)</td>
</tr>
<tr>
<td>L1</td>
<td>0</td>
<td>0</td>
<td>-0.05</td>
</tr>
<tr>
<td>L2</td>
<td>0</td>
<td>0</td>
<td>-0.06</td>
</tr>
<tr>
<td>L3</td>
<td>-0.01</td>
<td>0</td>
<td>-0.09</td>
</tr>
<tr>
<td>L4</td>
<td>-0.06</td>
<td>-0.02</td>
<td>-0.17</td>
</tr>
<tr>
<td>L5</td>
<td>-0.4</td>
<td>-0.3</td>
<td>-0.03</td>
</tr>
</tbody>
</table>

The above comparisons demonstrate that the numerical model reasonably simulated the behavior of the box culvert when an axle load was applied on the unsurfaced, shoulder, and pavement areas. Therefore, similar material properties, boundary conditions, and mesh densities were adopted for the numerical analysis in the parameter study to be discussed in the following chapter.

4.3.3 Earth Pressures above Culvert

The measured and computed vertical earth pressures on the top of the culvert under different axle loads on the unsurfaced section are shown in Table 4.6. Table 4.6 shows reasonable agreement between the measured and computed values. Figures 4.9, 4.10, and 4.11 show the vertical earth pressure contours above the culvert under Axle 1, Axle 2 or 4, and Axle 6 and Axle 5 or 7, respectively. Clearly, the wheel loads were distributed to the top of the culvert. The vertical pressure on the top of the culvert found from FLAC3D due to outer wheel of axle 1 was 131 kPa and that for the inner wheel was 43 kPa. The vertical pressures measured at those locations during the field test were 175 and 32 kPa respectively. Figure 4.9 shows the contour of vertical pressure (a negative sign represents a compressive stress) on the top of the culvert due to Axle 1.
TABLE 4.6
Comparison of Measured and Computed Pressures

<table>
<thead>
<tr>
<th>Axle No.</th>
<th>Pressure (kPa)</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>E1 (inner wheel)</td>
<td>E4 (outer wheel)</td>
<td>Measured</td>
<td>Computed</td>
</tr>
<tr>
<td>1</td>
<td></td>
<td>32</td>
<td>43</td>
<td>175</td>
<td>131</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>118</td>
<td>185</td>
<td>180</td>
<td>235</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>122</td>
<td>185</td>
<td>220</td>
<td>235</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>86</td>
<td>113</td>
<td>65</td>
<td>198</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>118</td>
<td>148</td>
<td>165</td>
<td>198</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>68</td>
<td>113</td>
<td>140</td>
<td>198</td>
</tr>
</tbody>
</table>

FIGURE 4.9
Vertical Earth Pressure Contour on the Top of the Culvert Due to Axle 1 (Unit: kPa)

The vertical pressure on the top of the culvert calculated by FLAC3D due to outer wheel of Axle 2/4 was 235 kPa and that for the inner wheel was 185 kPa. The vertical pressures measured at these locations during the field test due to Axle 2 were 180 and 118 kPa respectively.
and that from Axle 4 were 220 and 122 kPa respectively. Figure 4.10 shows the contour of vertical pressure on the top of the culvert due to Axle 2/4.

The vertical pressure on the top of the culvert found from FLAC3D due to the outer wheel of axle 6 was 198 kPa and that for the inner wheel was 148 kPa. The vertical pressures measured at these locations during the field test due to Axle 6 were 165 and 118 kPa respectively. Also the vertical pressure on the top of the culvert found from FLAC3D due to outer wheel of Axle 5/7 was 198 kPa and that for the inner wheel was 113 kPa. The measured vertical pressures at those locations from Axle 5 were 65 and 86 kPa respectively and those for Axle 7 were 140 and 86 kPa respectively. Figure 4.11 shows the contour of vertical pressure on the top of the culvert due to Axles 6 and 5/7.

![FIGURE 4.10
Vertical Earth Pressure Contour on the Top of the Culvert Due to Axle 2 or 4 (Unit: kPa)](image-url)
4.4 Culvert under Flexible Pavement

4.4.1 Material Models and Parameters

The flexible pavement on the top of the culvert consisted of two layers. The surface course of the pavement consisted of a hot mixed asphalt (HMA) layer, under which there was lime-treated subgrade. The culvert was made of reinforced concrete. The backfill around the culvert was a natural soil. The dimensions of the culvert and pavement layers were described in Chapter 3. Elastic models were used for all the materials.

The elastic modulus of the asphalt layer as obtained from the rebound test was 1827 MPa, which was used along with a Poisson's ratio of 0.3 for the model verification. The 1993 AASHTO Guide for Design of Pavement Structures suggested a typical value of elastic modulus for lime-treated subgrade ranging from 138 to 483 MPa. Therefore, an average value of 310 MPa was adopted for the verification of the numerical model. Poisson's ratio of 0.3 was used for the
lime-treated subgrade. The average elastic modulus of the soil obtained from the triaxial test was 10.7 MPa, which was used for the natural subgrade including the backfill soil. A Poisson's ratio of 0.3 was used for soils.

It was assumed that the concrete for the box culvert had a typical compressive strength of 31 MPa. Considering 1% steel reinforcement with an elastic modulus of 29,000 MPa, the elastic modulus of the reinforced concrete was determined to be 27,580 MPa. Poisson's ratio of the concrete was assumed to be 0.15. A summary of the material properties used in the numerical analysis is listed in Table 4.7.

Axles 1, 3, and 6 provided symmetric loading with respect to the culvert axis. Therefore, they were used to verify the numerical models and their computed deflections were compared with those from the field study.

<table>
<thead>
<tr>
<th>Material</th>
<th>Elastic modulus (MPa)</th>
<th>Poisson's ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>HMA</td>
<td>1,827</td>
<td>0.3</td>
</tr>
<tr>
<td>Lime treated subgrade</td>
<td>310</td>
<td>0.3</td>
</tr>
<tr>
<td>Natural subgrade and backfill soil</td>
<td>10.73</td>
<td>0.3</td>
</tr>
<tr>
<td>Culvert concrete</td>
<td>27,580</td>
<td>0.15</td>
</tr>
</tbody>
</table>

4.3.2 Numerical Meshes and Boundary Conditions

Only half of the culvert was modeled to utilize the symmetry condition as shown in Figure 4.12. Unyielding foundation conditions were assumed for the model. Therefore the vertical movement at the bottom of the culvert was restricted. All vertical boundaries were restrained for horizontal movement except for the free boundary of the box culvert in the y direction. The horizontal displacement of the culvert was restrained at the plane of symmetry (yz plane). Rollers were used at the symmetrical boundary, which allowed vertical movement but restrained horizontal movement. The boundary conditions are shown in Figure 4.2.

The length, width, and height of the model were 10.5 m, 7.5 m, and 4.225 m respectively. The numbers of zones were 87,864, 80,304, and 84,924 when Axles 1, 3, and 6 were applied in the model respectively. The reason for the difference in the number of zones for different axles is that each axle had a different load contact area, which required changing the size of the mesh on
the surface where the load was applied. Finer zones were used near the culvert and the zone density gradually decreased away from the culvert. The load was applied as a pressure on the surface of the pavement. The number and size of zones required to apply the pressure were calculated as shown in Table 4.8 so that the total wheel load was equal to that in the field test while the contact pressure was close to that in the field test.

**FIGURE 4.12**
FLAC3D Model of the Culvert
When Axle 1 was applied in the model, each wheel only had two zones due to the symmetry condition as shown in Figure 4.3. Axle 3 did not have its own load. However, when Axle 3 was in place, the wheel loads used for Axles 2 and 4 were applied on the culvert. Due to symmetry, only one of the axle loads was applied in the model as shown in Figure 4.4. Similarly, when Axle 6 was on place, the wheel loads for Axles 5 and 7 were also applied on the culvert. However, half of the load from Axle 6 and a full load from Axle 5 or 7 were applied to the model as shown in Figure 4.5.

### 4.3.2 Deflections of Culvert Top Slab

The measured deflections at Locations L1 through L6 were compared with the computed ones from FLAC3D. Table 4.9 and Figures 4.13 and 4.14 show the measured deflections compared with the computed ones from FLAC3D when Axles 1, 3 and 6 were applied on the pavement. Figures 4.13 (a) and 4.14 (a) show that the measured deflections at all the locations were in reasonable agreement with the computed ones from FLAC3D when Axle 1 was applied on the pavement. The differences in their deflections were more significant near the point of the load application and gradually decreased with an increase of the distance. Similarly, Figures 4.13 (b) and 4.14 (b) show reasonable agreement between the measured and computed deflections when Axle 3 was applied on the pavement. Figure 4.13 (c) and 4.14 (c) show a better comparison of the measured and computed deflections when Axle 6 was applied on the pavement. Overall, the computed deflection profiles had similar shapes to the measured ones but the measured
deflections were larger or smaller than the computed ones depending on the axles. Their differences were most obvious when Axle 1 was applied on the pavement and were smallest when Axle 6 was applied.

**FIGURE 4.13**
Measured and Computed Deflections along Culvert Axis
FIGURE 4.14
Measured and Computed Deflections along Culvert Span

(a) Axle 1

(b) Axle 3

(c) Axle 6
TABLE 4.9
Comparison of Measured and Computed Deflections

<table>
<thead>
<tr>
<th>Transducer Location</th>
<th>Axle 1 Measured (mm)</th>
<th>Axle 1 Computed (mm)</th>
<th>Axle 3 Measured (mm)</th>
<th>Axle 3 FLAC3D (mm)</th>
<th>Axle 6 Measured (mm)</th>
<th>Axle 6 FLAC3D (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L1</td>
<td>-0.09</td>
<td>-0.13</td>
<td>-0.53</td>
<td>-0.47</td>
<td>-0.58</td>
<td>-0.52</td>
</tr>
<tr>
<td>L2</td>
<td>-0.09</td>
<td>-0.13</td>
<td>-0.53</td>
<td>-0.48</td>
<td>-0.58</td>
<td>-0.53</td>
</tr>
<tr>
<td>L3</td>
<td>-0.07</td>
<td>-0.12</td>
<td>-0.49</td>
<td>-0.44</td>
<td>-0.55</td>
<td>-0.47</td>
</tr>
<tr>
<td>L4</td>
<td>-0.05</td>
<td>-0.08</td>
<td>-0.31</td>
<td>-0.3</td>
<td>-0.36</td>
<td>-0.32</td>
</tr>
<tr>
<td>L5</td>
<td>-0.1</td>
<td>-0.12</td>
<td>-0.52</td>
<td>-0.45</td>
<td>-0.57</td>
<td>-0.49</td>
</tr>
<tr>
<td>L6</td>
<td>-0.13</td>
<td>-0.15</td>
<td>-0.41</td>
<td>-0.37</td>
<td>-0.45</td>
<td>-0.41</td>
</tr>
</tbody>
</table>

The above comparisons demonstrate that the numerical model reasonably simulated the behavior of the box culvert when an axle load was applied on the flexible pavement. Therefore, similar material properties, boundary conditions, and mesh densities were adopted for the numerical analysis in the parametric study to be discussed in the following chapter.

4.4 Summary

This chapter describes the development of the numerical models for culverts under rigid and flexible pavements subjected to static axle loading. All the materials modeled in this study were linearly elastic because the stress levels of load rating as compared with the strengths of materials are usually low. The numerical models created using the finite difference program FLAC3D were validated using the data obtained from the field tests. A few conclusions can be drawn from this numerical study:

1. Elastic models were used for all the materials. This study showed that the assumption of linear elastic models for all the materials is valid for culverts under pavements.

2. The elastic moduli of the reinforced concrete culvert, plain cement concrete pavement, cement-treated base, lime treated subgrade, and HMA pavement used in the numerical modeling were 27,580, 24,545, 5,171, 310, and 1,827 MPa, respectively. The analyses showed that the selected modulus values were appropriate for the respective materials.
3. The deflections computed by the numerical method were in good agreement with those observed in the field tests for both culverts.

4. Pressure applied on the specified contact area of the tire can simulate the wheel load well.
Chapter 5: Parametric Study

5.1 Introduction

Based on the verification of the numerical models with the experimental results of the two box culverts loaded in the field, a simplified model was created for a parametric study. The parametric study was performed to investigate the influence of pavement type, pavement thickness, fill depth, and culvert span on the load distribution over the culvert under wheel loads. The constitutive model, boundary conditions, and material properties were similar to those used in the model verification in the previous chapter except for the soil elastic moduli. Since the soil elastic moduli obtained from the triaxial tests on the samples obtained from two culvert test sites were different, 12 MPa was used for the elastic modulus of soil for simplicity in the parametric study. A total of ninety-six culvert models were analyzed for the parametric study, among which forty-eight culverts each were under rigid and flexible pavements. One additional model each was created to investigate the culvert top slab thickness effect at a larger span under rigid and flexible pavements.

Figure 5.1 shows the schematic plan of the culvert modeled for the parametric study. Each modeled case consisted of a two-lane road having a lane width of 3.75 m and a 3 m wide unsurfaced shoulder on each side. The culvert was symmetrical about both the road centerline and the culvert axis. Therefore only a quarter of the culvert was modeled to utilize the symmetry of the culvert and also to minimize the time required for the numerical analysis. Figure 5.1 shows the limit of the culvert actually modeled and the coordinate system. The intersection point between the road centerline and the culvert axis was considered as the origin. For all the subsequent discussion in this study, the distance is considered from the origin as shown in Figure 5.1.
Figures 5.2 and 5.3 show the typical cross sections of the culvert models under rigid and flexible pavements respectively. Similar to the culvert model under the rigid pavement used for the verification, the culvert models with rigid pavements for the parametric study also consisted of a top concrete layer, a cement-treated base layer, a lime-treated subgrade layer, and a natural subgrade layer. The thicknesses of the cement-treated base layer and the lime-treated subgrade layer were fixed at 100 mm and 150 mm respectively for all the models. However, the flexible pavement consisted of a hot mix asphalt layer, a lime-treated subgrade layer, and a natural
subgrade layer, in which the lime-treated subgrade was 150 mm thick. The thicknesses of the concrete layer, the asphalt layer, and the natural subgrade layer were varied during the parametric study. All the box culverts modeled had 250 mm thick walls and top and bottom slabs. The clear height of each culvert was fixed at 3 m. The span of the culvert was one of the variables for the parametric study. The width of the backfill beyond the culvert was fixed at 4.625 m for all the culverts analyzed in this study, which is far enough to avoid the boundary effect. Figures 5.4 and 5.5 show two typical numerical models for the culverts under rigid and flexible pavements respectively. Finer zones were used near the culvert and the zone density gradually decreased away from the culvert.

FIGURE 5.2
Typical Cross-Section of the Culvert Model under a Rigid Pavement
FIGURE 5.3
Typical Cross-Section of the Culvert Model under a Flexible Pavement

FIGURE 5.4
Typical Numerical Model for the Culvert under a Rigid Pavement
Due to the symmetry of the problem, one quarter of the model was created in FLAC3D as shown in Figure 5.4. The boundary conditions included the vertical and horizontal displacements fixed at the bottom boundary and the lateral displacements fixed at the four-side boundaries. The load was applied on the pavement as a pressure in an area equal to the tire footprint area of 0.5 m x 0.25 m as specified in the 2007 AASHTO LRFD Bridge Design Specifications. Figure 5.6 shows the application of the wheel loads on the culvert. Because of the symmetry of the model created for the parametric study only half of one wheel load was applied to the model in an area of 0.5 m x 0.125 m. A typical tire contact pressure of 550 kPa was used.
Each culvert model was solved and saved at two stages. The culvert model was first solved and saved when the initial equilibrium was reached at the maximum unbalanced force ratio of $10^{-5}$ under the self-weight. Then the wheel load was applied on the pavement as the pressure and the model was again stepped to the equilibrium and saved after the equilibrium. The additional pressure on the culvert due to the applied load was obtained by deducting the pressure at the final equilibrium condition from that at the initial equilibrium condition under the self-weight.

5.2 Influence Factors

Different factors play different roles in the distribution of the load through the pavement and earth fill onto the culvert. Three key influence factors were considered in the parametric study. These factors were varied within the practical ranges to evaluate the effects of (a)
pavement thickness, (b) fill depth, and (c) span for culverts under both rigid and flexible pavements.

5.3 Rigid Pavement

Forty-eight culvert models under rigid pavements were created to study the effects of the influence factors mentioned above. Analyses were carried out in three categories based on the spans of 1.8 m, 3.6 m, and 5.4 m. In each category, fill depths above the culverts at 0.6 m, 1.2 m, 1.8 m, and 2.4 m were considered. For each fill depth, the thickness of the concrete pavement layer was varied from 0.2 m to 0.35 m at an increment of 0.05 m. Because of this variation, the thickness of the natural subgrade was also changed to meet the total fill depth. The vertical pressure distribution on the top of the culvert was monitored along the axis and span of the culvert through the origin of the coordinate system (referred to as the center herein).

5.3.1 Effect of Concrete Pavement Thickness

Figures 5.7 and 5.8 show the variations of vertical pressure distributions on the culvert along and perpendicular to the culvert axis with the thickness of the concrete pavement thickness at different fill depths, respectively. All these distributions are presented at the culvert span of 1.8 m. These figures clearly indicate that the intensity of the vertical pressure on the culvert decreased gradually with the increase of the concrete pavement thickness. A similar trend was observed at the culvert spans of 3.6 m and 5.4 m. However, there was one exception where the intensity of the vertical pressure increased with the thickness of the concrete pavement. This case occurred when the fill depth was 0.6 m and the concrete pavement thickness was 0.35 m (i.e., there was no soft natural subgrade above the culvert). All the layers above the culvert had relatively higher elastic moduli; therefore, the applied load was distributed to a smaller area, which resulted in higher vertical pressure.
(a) Fill depth = 0.6 m

(b) Fill depth = 1.2 m

FIGURE 5.7
Vertical Pressure Distribution on the Culvert along the Culvert Axis at Different Concrete Pavement Thickness and Fill Depth (Culvert Span = 1.8 m)
FIGURE 5.7
Vertical Pressure Distribution on the Culvert Along the Culvert Axis at Different Concrete Pavement Thickness and Fill Depth (Culvert Span = 1.8 m) (continued)
(a) Fill depth = 0.6 m

(b) Fill depth = 1.2 m

FIGURE 5.8
Vertical Pressure Distribution on the Culvert Perpendicular to the Culvert Axis at Different Concrete Pavement Thickness and Fill Depth (Culvert Span = 1.8 m)
Figure 5.9 shows the variations of the maximum vertical pressure on the culvert with the concrete pavement thickness at different fill depths for the culvert spans of 1.8 m, 3.6 m, and 5.4 m. This figure also indicates that the maximum vertical pressure on the culvert decreased gradually with the increase in the concrete pavement thickness. This trend was valid for all cases except for the case with 0.6 m fill depth and 0.35 m pavement thickness because of the absence of the soft natural subgrade layer above the culvert.
FIGURE 5.9
Variation of the Maximum Vertical Pressure on the Culvert with the Concrete Pavement Thickness and Different Fill Depths and Culvert Spans
5.3.2 Effect of Fill Depth

Figures 5.10 and 5.11 show the variations of the vertical pressure distributions on the culvert along and perpendicular to the culvert axis with the fill depth at different concrete pavement thicknesses. These figures clearly indicate that the intensity of the vertical pressure on the culvert decreased gradually with the increase in the fill depth. A similar trend was observed for the culverts with spans of 1.8 m and 5.4 m. However, the locations of the maximum vertical pressures were not consistent for all fill depths. Along the culvert axis, the maximum vertical stress was located below the wheel load for the case with 0.6 m fill depth but at the middle of the axle load for the case with 1.2 m or more fill depth. Along the culvert span, the locations of the maximum vertical pressures were either at the middle of the axle load or at the edge of the culvert wall.

![Figure 5.10](image_url)

(a) Concrete pavement thickness = 0.2 m

(b) Concrete pavement thickness = 0.25 m

FIGURE 5.10
Vertical Pressure Distribution on the Culvert along the Culvert Axis at Different Fill Depths and Concrete Pavement Thicknesses (Culvert Span = 3.6 m)
(c) Concrete pavement thickness = 0.3 m

(d) Concrete pavement thickness = 0.35 m

FIGURE 5.10
Vertical Pressure Distribution on the Culvert Along the Culvert Axis for Different Fill Depths and Concrete Pavement Thicknesses (Culvert Span = 3.6 m) (Continued)
FIGURE 5.11
Vertical Pressure Distribution on the Culvert Perpendicular to the Culvert Axis at Different Fill Depths and Concrete Pavement Thicknesses (Culvert Span = 3.6 m)

(a) Concrete pavement thickness = 0.2 m

(b) Concrete pavement thickness = 0.25 m
(c) Concrete pavement thickness = 0.3 m

(d) Concrete pavement thickness = 0.35 m

FIGURE 5.11
Vertical Pressure Distribution on the Culvert Perpendicular to the Culvert Axis for Different Fill Depths and Concrete Pavement Thicknesses (Culvert Span = 3.6 m) (Continued)

Figure 5.12 shows the variations of the maximum vertical pressures on the culverts with different spans, fill depths, and concrete pavement thickness. It is shown that the change of the maximum vertical pressure with the fill depth was higher at a low fill depth and the rate of the change gradually decreased at a higher fill depth. This similar trend was observed for each pavement thickness.
FIGURE 5.12
Variation of the Maximum Vertical Pressure on the Culvert with Different Spans, Fill Depths, and Concrete Pavement Thickness
5.3.3 Effect of Span

Figure 5.13 shows the variation of the vertical pressure distribution on the culvert along the culvert axis with the span for a fill depth of 1.2 m and different concrete pavement thicknesses. Figure 5.14 shows the variation of the vertical pressure distribution on the culvert along the culvert axis at different fill depth and the pavement thickness of 0.25 m. These figures clearly indicate that the intensity of the vertical pressure on the culvert decreased gradually with the increase of the culvert span.

(a) Concrete pavement thickness = 0.2 m

(b) Concrete pavement thickness = 0.25 m

FIGURE 5.13
Vertical Pressure Distribution on the Culvert along the Culvert Axis for Different Spans and Concrete Pavement Thicknesses (Fill Depth = 1.2 m)
(c) Concrete pavement thickness = 0.3 m

(d) Concrete pavement thickness = 0.35 m

FIGURE 5.13
Vertical Pressure Distribution on the Culvert along the Culvert Axis at Different Spans and Concrete Pavement Thicknesses (Fill Depth = 1.2 m) (Continued)
FIGURE 5.14
Vertical Pressure Distribution on the Culvert along the Culvert Axis at Different Spans and Fill Depths (Concrete Pavement Thickness = 0.25 m)
Figure 5.15 shows the variation of the maximum vertical pressure on the culvert with the span for different fill depths and concrete pavement thicknesses. The figure shows that the change in the culvert span had more effect on the vertical pressure distribution for a low fill depth. The vertical pressure on the culvert increased with the decrease in the span. For the lower fill depth the change in the vertical pressure when the span was varied from 1.8 m to 3.6 m was more significant than that when the span was varied from 3.6 m to 5.4 m. For the higher fill
depth, however, the rate of change for the maximum vertical pressure with the span was uniform over all spans considered.

![Graph showing variation in maximum vertical pressure on culvert with span at different fill depths and concrete pavement thicknesses](image)

(a) Concrete pavement thickness = 0.2 m

(b) Concrete pavement thickness = 0.25 m

**FIGURE 5.15**
Variation in the Maximum Vertical Pressure on the Culvert with the Span at Different Fill Depths and Concrete Pavement Thicknesses
Since the thickness of the culvert top slab used in the numerical model for all three culvert spans was the same, the culverts with a longer span were more flexible than those with a smaller span, and thus had greater deflections as shown in Figure 5.16. The deflections might have affected the magnitudes of the vertical pressure on the culverts with a larger span. To investigate this effect, a separate model was created for a 5.4 m span culvert under a 1.2 m fill depth and a 0.2 m thick pavement by doubling the top slab thickness (new thickness = 0.50 m). Figure 5.17 shows the distributions of the vertical pressure along the culvert axis for the 1.8 m span culvert and the 5.4 m span culverts with 0.25 m and 0.50 m thick top slabs. It is clearly
shown that the increase of the slab thickness increased the vertical pressure on the culvert and the increased vertical pressure for the larger span was closer to that for the smaller span. In practice, thicker top slabs are always used to reduce deflections for larger span culverts. Therefore, it is conservative not to consider the vertical pressure reduction due to the increase of the culvert span.

**FIGURE 5.16**
Deflection of Culvert Top Slab at Different Span

**FIGURE 5.17**
Effect of the Culvert Top Slab Thickness on the Vertical Pressure Distribution
5.3.4 Summary

The distribution of the vertical pressure due to a wheel load on the top slab of the low-fill box culvert under a rigid pavement was investigated using the 3D finite difference method in FLAC3D. Three key influence factors including the pavement thickness, fill depth, and culvert span, were considered. Before this parametric study, the finite difference model was verified against the field test data. The following conclusions can be made based on the parametric study:

1. The intensity of the vertical pressure on the top slab of the culvert gradually decreased as the concrete pavement thickness was increased because the wheel load was distributed over a wider area.
2. The intensity of the vertical pressure gradually decreased with the increase of the fill depth over the culvert also because the wheel load was distributed over a wider area.
3. The location of the maximum vertical pressure was below the wheel load when the fill depth was 0.6 m. However, the maximum vertical pressure was located below the middle of the axle load at the fill depth of 1.2 m and larger.
4. The vertical pressure on the top slab of the culvert decreased with the increase of the culvert span. The influence of the span was greater for lower fill depths. The rate of change in the vertical pressure on the top slab of the culvert with the span decreased with the increase of the culvert span. The increase of the slab thickness reduced the slab deflection and increased the maximum vertical pressure on the culvert.

5.4 Flexible Pavement

Forty-eight culvert models under flexible pavements were created to study the effects of the influence factors mentioned above. Analyses were carried out in three categories based on the spans of 1.8, 3.6 and 5.4 m. In each category, fill depths above the culverts at 0.6, 1.2, 1.8, and 2.4 m were considered. For each fill depth, the thickness of the HMA pavement layer was set to values of 0.15, 0.23, 0.30, and 0.38 m. Because of this variation, the thickness of the natural
subgrade was also changed to match the total fill depth. The vertical pressure distribution on the top of the culvert was monitored along the axis and span of the culvert through the center.

5.4.1 Effects of HMA Pavement Thickness

Figures 5.18 and 5.19 show the variations of vertical pressure distributions on the culvert along and perpendicular to the culvert axis with the thickness of the HMA pavement for different fill depths, respectively. All these distributions are presented at the culvert span of 1.8 m. These figures clearly indicate that the intensity of the vertical pressure on the culvert decreased gradually with the increase of the HMA pavement thickness. A similar trend was observed at the culvert spans of 3.6 and 5.4 m.
FIGURE 5.18
Vertical Pressure Distribution on the Culvert along the Culvert Axis at Different HMA Pavement Thickness and Fill Depth (Culvert Span = 1.8 m)
FIGURE 5.18
Vertical Pressure Distribution on the Culvert along the Culvert Axis at Different HMA Pavement Thickness and Fill Depth (Culvert Span = 1.8 m) (Continued)

(c) Fill depth = 1.8 m

(d) Fill depth = 2.4 m
FIGURE 5.19
Vertical Pressure Distribution on the Culvert Perpendicular to the Culvert Axis at Different HMA Pavement Thickness and Fill Depth (Culvert Span = 1.8 m)
Figure 5.20 shows the variations of the maximum vertical pressure on the culvert with the HMA pavement thickness at different fill depth for the culvert spans of 1.8, 3.6, and 5.4 m. This figure also indicates that the maximum vertical pressure on the culvert decreased gradually with the increase in the HMA pavement thickness.
FIGURE 5.20
Variation of the Maximum Vertical Pressure on the Culvert with the HMA Pavement Thickness at Different Fill Depth and Culvert Span
### 5.4.2 Effect of Fill Depth

Figures 5.21 and 5.22 show the variations in the vertical pressure distributions on the culvert along and perpendicular to the culvert axis with the fill depth at different HMA pavement thickness. These figures clearly indicate that the intensity of the vertical pressure on the culvert decreased gradually with the increase in the fill depth. A similar trend was observed for the culverts with spans of 1.8 and 5.4 m. However, the locations of the maximum vertical pressures were not consistent for all fill depths. Along the culvert axis, the maximum vertical pressure was located below the wheel load for the case with 0.6 m fill depth but at the middle of the axle load for the case with 1.2 m or more fill depth. Along the culvert span, the locations of the maximum vertical pressures were either at the middle of the axle load or at the edge of the culvert wall.
FIGURE 5.21
Vertical Pressure Distribution on Culvert Slab along the Culvert Axis at Different Fill Depth and HMA Pavement Thickness (Culvert Span = 3.6 m)

(a) HMA pavement thickness = 0.15 m

(b) HMA pavement thickness = 0.23 m
FIGURE 5.21
Vertical Pressure Distribution on Culvert Slab along the Culvert Axis at Different Fill Depth and HMA Pavement Thickness (Culvert Span = 3.6 m) (Continued)
(a) HMA pavement thickness = 0.15 m

(b) HMA pavement thickness = 0.23 m

FIGURE 5.22
Vertical Pressure Distribution on Culvert Slab Perpendicular to the Culvert Axis at Different Fill Depth and HMA Pavement Thickness (Span = 3.6 m)
Figure 5.23 shows the variations of the maximum vertical pressures on the culverts for different spans, fill depths, and HMA pavement thickness. It is shown that the change of the maximum vertical pressure with the fill depth was higher for a low fill depth and the rate of the change gradually decreased with a greater fill depth. This similar trend was observed for each pavement thickness.
FIGURE 5.23
Variation of the Maximum Vertical Pressure on the Culvert with Different Span, Fill Depth, and Asphalt Pavement Thickness

(a) Span = 1.8 m

(b) Span = 3.6 m

(c) Span = 5.4 m
5.4.3 Effect of Span

Figure 5.24 shows the variation of the vertical pressure distribution on the culvert along the culvert axis with the span length for a fill depth of 1.2 m and different HMA pavement thicknesses. Figure 5.25 shows the variation of the vertical pressure distribution on the culvert along the culvert axis for different fill depths and a pavement thickness of 0.25 m. These figures clearly indicate that the intensity of the vertical pressure on the culvert decreased gradually with an increase of the culvert span.

FIGURE 5.24
Vertical Pressure Distribution on the Culvert along the Culvert Axis at Different Span and HMA Pavement Thickness (Fill Depth = 1.2 m)
FIGURE 5.24
Vertical Pressure Distribution on the Culvert along the Culvert Axis at Different Span and HMA Pavement Thickness (Fill Depth = 1.2 m) (Continued)
FIGURE 5.25
Vertical Pressure Distribution on the Culvert along the Culvert Axis at Different Span and Fill Depth (HMA Pavement Thickness = 0.30 m)
Figure 5.26 shows the variation of the maximum vertical pressure on the culvert with span length for different fill depths and HMA pavement thicknesses. The figure shows that the change in the culvert span had more effect on the vertical pressure distribution for low fill depths. The vertical pressure on the culvert increased with a decrease in the span. For lower fill depths the change in the vertical pressure when the span was varied from 1.8 m to 3.6 m was
more significant than that when the span was varied from 3.6 m to 5.4 m. For greater fill depths, however, the rate of change for the maximum vertical pressure with the span was uniform over all spans considered.

**FIGURE 5.26**
Variation in the Maximum Vertical Pressure on the Culvert with Span Length at Different Fill Depths and Asphalt Pavement Thicknesses

(a) HMA pavement thickness = 0.15 m

(b) HMA pavement thickness = 0.23 m
Since the thickness of the culvert top slab used in the numerical model for all the three culvert spans was the same, the culverts with a longer span were more flexible than those with a shorter span and thus had greater deflections as shown in Figure 5.27. The deflections might have affected the magnitudes of the vertical pressure on the culverts with a larger span. To investigate this effect, a separate model was created for a 5.4 m span culvert under a 1.2 m fill depth and a 0.15 m thick pavement by doubling the top slab thickness (new thickness = 0.50 m). Figure 5.28 shows the distributions of the vertical pressure along the culvert axis for the 1.8 m span culvert and the 5.4 m span culverts with 0.25 m and 0.50 m thick top slabs. It is clearly
shown that the increase of the slab thickness increased the vertical pressure on the culvert and the increased vertical pressure for the larger span was close to that for the smaller span. In practice, thicker top slabs are always used to reduce deflections for larger span culverts. Therefore, it is conservative not to consider the vertical pressure reduction due to the increase of the culvert span.

![Graph showing deflection of culvert top slab for different span](image1)

**FIGURE 5.27** Deflection of culvert top slab for different span

![Graph showing vertical pressure for different span](image2)

**FIGURE 5.28** Deflection of Slab for Different Span

### 5.4.4 Summary

The distribution of the vertical pressure due to a wheel load on the top slab of the low-fill box culvert under a flexible pavement was investigated using the 3D finite difference method in FLAC3D. Three key influence factors including the pavement thickness, fill depth, and culvert
span were considered. The finite difference model was verified against the field test data before this parametric study was conducted. The following conclusions can be made based on the parametric study:

1. The intensity of the vertical pressure on the top slab of the culvert gradually decreased as the increase of the HMA pavement thickness because the wheel load was distributed over a wider area.
2. The intensity of the vertical pressure gradually decreased with the increase of the fill depth over the culvert also because the wheel load was distributed over a wider area.
3. The location of the maximum vertical pressure was below the wheel load when the fill depth was 0.6 m. However, the maximum vertical pressure was located below the middle of the axle load at the fill depth of 1.2 m and larger.
4. The vertical pressure on the top slab of the culvert decreased with the increase of the culvert span. The influence of the span was more at a lower fill depth. The rate of change in the vertical pressure on the top slab of the culvert with the span length decreased with the increase of the culvert span. An increase of the slab thickness minimized the influence of the culvert span.

5.5 Comparison with AASHTO Pressure Distribution

5.5.1 Rigid Pavement

The maximum vertical pressure on the culvert obtained by the numerical method for each case can be compared with the average vertical pressures using the formulae in the 2007 AASHTO LRFD Bridge Design Specifications and the 1992 AASHTO Standard Specifications for Highway Bridges. The 2007 AASHTO LRFD Bridge Design Specifications suggested two methods for the estimating the pressure distribution depending on the type of fill material, i.e., the H distribution and the 1.15 H distribution as discussed in Section 2.3.2, in which H is the fill depth. Tables 5.1 to 5.3 show the comparison of the calculated vertical pressures with the AASHTO LRFD H distribution, the AASHTO LRFD 1.15 H distribution, and the distribution specified by the AASHTO Standard Specifications for Highway Bridges respectively. Figure
5.29 compares the calculated vertical pressures from the numerical method and the AASTHO distribution methods for the culverts with a span of 1.8 m. Tables 5.1 to 5.3 and Figure 5.29 all show that the AASTHO distribution methods over-predicted the maximum vertical pressure when compared with the numerical method. It is shown that the 2007 AASHTO LRFD Bridge Design Specifications calculated higher vertical pressures than the 1992 AASHTO Standard Specifications for Highway Bridges. The difference in the calculated vertical pressures between the numerical method and the AASHTO distribution methods decreased with an increase of the fill depth.

### TABLE 5.1
Comparison of the Calculated Vertical Pressures by the Numerical Method and the AASHTO LRFD H Distribution Method

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<th>Fill depth (m)</th>
<th>Pavement thickness (m)</th>
<th>Maximum pressure from numerical method (kPa)</th>
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### TABLE 5.2
Comparison of the Calculated Vertical Pressures by the Numerical Method and the AASHTO LRFD 1.15 H Distribution Method

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</table>
FIGURE 5.29
Comparison of the Calculated Pressures by the Numerical Method and the AASHTO Distribution Methods for the Culvert with 1.8 m Span

(a) Fill depth = 0.6 m

(b) Fill depth = 1.2 m
The maximum vertical pressure on the culvert obtained by the numerical method for each case can be compared with the average vertical pressures using the formulae in the 2007 AASHTO LRFD Bridge Design Specifications and the 1992 AASHTO Standard Specifications for Highway Bridges. Tables 5.4 to 5.6 show the comparison of the calculated vertical pressures...
with the AASHTO LRFD H distribution, the AASHTO LRFD 1.15 H distribution, and the
distribution specified by the AASHTO Standard Specification respectively. Figure 5.30
compares the calculated vertical pressures by the numerical method and the AASTHO
distribution methods for the culverts with a span of 1.8 m. Tables 5.4 to 5.6 and Figure 5.30 all
show that the AASTHO distribution methods over-predicted the maximum vertical pressure as
compared with the numerical method for fill depths up to 1.2 m. However, for greater fill depths,
the AASHTO distribution methods predicted a maximum vertical pressure similar to or less than
the pressure predicted by numerical method. It is also shown that the 2007 AASHTO LRFD
Bridge Design Specifications results in the calculation of a higher vertical pressure than the
AASHTO Standard Specifications for Highway Bridges. The difference in the calculated vertical
pressures between the numerical method and the AASHTO distribution methods decreased with
an increase of the fill depth.

<table>
<thead>
<tr>
<th>Fill depth (m)</th>
<th>Pavement thickness (m)</th>
<th>Max. pressure from numerical method (kPa)</th>
<th>Average pressure by AASHTO LRFD H distribution (kPa)</th>
<th>Pressure difference (%)</th>
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<tbody>
<tr>
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<td>Culvert span (m)</td>
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<td>Span (m)</td>
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### TABLE 5.5
Comparison of the Calculated Vertical Pressures by the Numerical Method and the AASHTO LRFD 1.15 H Distribution Method

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<th>Fill depth (m)</th>
<th>Pavement thickness (m)</th>
<th>Max. pressure from numerical method (kPa)</th>
<th>Average pressure by AASHTO LRFD 1.15 H distribution (kPa)</th>
<th>Pressure difference (%)</th>
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<td>Culvert span (m)</td>
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<td>Pressure difference (%)</td>
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</table>
FIGURE 5.30
Comparison of the Calculated Pressures by the Numerical Method and the AASHTO Distribution Methods for the Culvert with 1.8 m Span
FIGURE 5.30
Comparison of the Calculated Pressures by the Numerical Method and the AASHTO Distribution Methods for the Culvert with 1.8 m Span (Continued)
5.5.3 Summary

From the comparison of the calculated vertical pressures estimated by the numerical method and the AASHTO distribution methods, the following conclusions can be drawn:

1. The AASHTO pressure distribution methods are overly conservative for the wheel load distribution on a low-fill box culvert under a rigid pavement. The differences between the AASHTO and numerical estimates of the distributed vertical pressure decreased with the increase of the fill depth.

2. For the culvert under a flexible pavement, use of the AASHTO pressure distribution methods resulted in higher vertical pressure estimates than those generated by the numerical method for fill depths up to 1.2 m. The differences between the calculated vertical pressures predicted using the numerical method and the AASHTO distribution methods decreased with increasing the fill depth and became small for fill depths of 1.8 and 2.4 m.

3. For all the fill depths considered, the pressures calculated using the 2007 AASHTO LRFD Bridge Design Specifications were higher than those calculated using the 1992 AASHTO Standard Specification for Highway Bridges. For greater fill depths, the pressure calculated using the 1992 AASHTO Standard Specification for Highway Bridges closely matched the numerical result.
Chapter 6: Proposed Simplified Methods for Pressure Distribution

6.1 Introduction

From the parametric study presented in Chapter 5, it became clear that the magnitude of the vertical pressure on the top slab of a low-fill box culvert under pavements (rigid and flexible pavements) due to wheel loads depends on pavement type, pavement thickness, and fill depth. The 2007 AASHTO LRFD Bridge Design Specifications and the 1992 AASHTO Standard Specifications for Highway Bridges over-predict the vertical pressure on the culvert due to the fact that the effect of the pavement is not considered. Simplified methods were developed in this study to estimate the vertical pressures under rigid and flexible pavements. The development of the simplified methods is presented in this chapter.

6.2 Development of Simplified Methods

6.2.1 Culvert under Rigid Pavement

The fill above the culvert considered in this study consisted of concrete pavement, cement-treated base course, lime treated subgrade, and natural subgrade. Each layer distributes the vertical pressure at a different distribution angle. Giroud and Han (2004) suggested an approximate solution for the vertical pressure distribution angle from a base course to a subgrade based on Burmister’s theoretical solution (Burmister, 1958) as follows:

\[
\tan \alpha_1 = \tan \alpha_0 \left[ 1 + 0.204 \left( \frac{E_{bc}}{E_{sg}} - 1 \right) \right]
\]

Equation 6.1

where \( \alpha_1 \) = pressure distribution angle in the base course,
\( \alpha_0 \) = reference pressure distribution angle for a uniform medium defined by \( E_{bc} = E_{sg} \),
\( E_{bc} \) = modulus of elasticity of base course,
\( E_{sg} \) = modulus of elasticity of subgrade.
In this proposed simplified model the vertical pressure distribution angles from the concrete pavement to the cement-treated base course, from the cement-treated base course to the lime-treated subgrade, and the lime-treated subgrade to the natural subgrade were determined by Equation 6.2, which is a modification of Equation 6.1:

\[
\tan \alpha_1 = \tan \alpha_0 \left[ 1 + 0.204 \left( \frac{E_1}{E_2} - 1 \right) \right]
\]

Equation 6.2

where \( E_1 \) = elastic modulus of the pavement layer under consideration, \( E_2 \) = elastic modulus of the underlying layer.

The angle, \( \alpha_0 \), is the reference pressure distribution angle for a uniform medium. This angle can be taken as 30° for select granular fill (i.e., the 60-degree rule) or 27° for all other fills (i.e., the 2:1 distribution). In the calculation of the distribution angle between pavement layers, this angle can be taken as 27°. The vertical pressure distribution angle from the natural subgrade to the culvert was also taken as 27° in this study because the subgrade in the field tests was not select granular fill.

Figure 6.1 shows the schematic of the vertical pressure distribution on the culvert under a rigid pavement. The distributed area is determined by the distribution angle method as shown in Figure 6.1. In Figure 6.1, “a” represents the length of the rectangular distribution area on the top slab of the culvert along the culvert axis due to a single wheel load and can be determined as follows:

\[
a = t_1 + 2(h_1 \tan \alpha_1 + h_2 \tan \alpha_2 + h_3 \tan \alpha_3 + h_4 \tan \alpha_4)
\]

Equation 6.3

where \( a \) = length of the distributed area by one single wheel load, 
\( t_1 \) = length of the tire footprint, 
\( h_1, h_2, h_3, \) and \( h_4 \) = thicknesses of concrete pavement, cement-treated base, lime-treated subgrade, and natural subgrade, respectively.
\( \alpha_1, \alpha_2, \alpha_3, \) and \( \alpha_4 \) = distribution angles between concrete pavement, cement-treated base, lime-treated subgrade, and natural subgrade.

Since there are the same distributions in the direction perpendicular to the culvert axis, the width of the distributed area in the perpendicular direction is \( a - (t_l - t_w) \), where \( t_w \) is the width of the tire footprint. The length of the combined distribution area by the two wheel loads can be determined as follows:

\[
b = a + s \tag{Equation 6.4}
\]

where \( b \) = length of the combined distribution area,
\( s \) = spacing of the two wheel loads.
Considering the rigidity of the concrete pavement, the pavement is assumed to act as a rigid foundation to carry the applied loads and distribute a uniform vertical pressure onto the culvert. The magnitude of the pressure can be determined by dividing the total loads from both wheels by the total distribution area as follows:
\[ p_c = \frac{2P}{b(a - t_i + t_w)} \]  

**Equation 6.5**

where \( p_c \) = distributed pressure on the top of the culvert.

Therefore, the proposed distribution model distributes the vertical pressure wheel loads on the top slab of a low-fill box culvert under rigid pavement as shown in Figure 6.2.

**FIGURE 6.2**

**Distributed Pressure on the Top Slab of the Culvert under a Rigid Pavement**

### 6.2.2 Culvert under Flexible Pavement

The fill above the culvert considered in this study consisted of HMA pavement, lime-treated subgrade, and natural subgrade. Each layer distributes the vertical pressure at a different distribution angle. In this proposed simplified model the vertical pressure distribution angles from the asphalt pavement to the lime-treated subgrade, and the lime-treated subgrade to the natural subgrade were determined by Equation 6.2. The angle, \( \alpha_0 \), which is the reference pressure distribution angle for a uniform medium, was taken as 27\(^\circ\) (i.e., 2:1 distribution). The vertical pressure distribution angle from the natural subgrade to the culvert was also taken as 27\(^\circ\).

Figure 6.3 shows the schematic of the vertical pressure distribution on the culvert under a flexible pavement. The distributed area is determined by the distribution angle method as shown in Figure 6.3. In Figure 6.3, “a” represents the length of the rectangular distribution area on the top slab of the culvert along the culvert axis due to a single wheel load and can be determined as follows:
\[ a = t_l + 2(h_1 \tan \alpha_1 + h_2 \tan \alpha_2 + h_3 \tan \alpha_3) \]  

Equation 6.6

where \( a \) = length of the distributed area by one single wheel load,
\( t_l \) = length of the tire footprint,
\( h_1, h_2, \) and \( h_3 \) = thicknesses of asphalt pavement, lime-treated subgrade, and natural subgrade, respectively,
\( \alpha_1, \alpha_2, \) and \( \alpha_3 \) = distribution angles between asphalt pavement, lime-treated subgrade, and natural subgrade.
Considering the flexibility of the HMA surface layer, the surface layer is assumed to carry the applied wheel loads and distribute a uniform vertical pressure independently onto the culvert. The uniform vertical pressure is:
Equation 6.7

\[ p_{c1} = \frac{P}{a(a - t_l + t_w)} \]

where \( p_{c1} \) = distributed vertical pressure from an individual wheel load.

Within the overlapped area, the vertical pressure on the top slab of the culvert can be determined using the superposition of the pressures due to two individual wheels, i.e.,

\[ p_{c2} = 2p_{c1} \quad \text{Equation 6.8} \]

where \( p_{c2} \) = distributed vertical pressure within the overlapped area. The overlapped length is

\[ c = \frac{2a - b}{2} = \frac{a - s}{2} \quad \text{Equation 6.9} \]

where \( c \) = overlapped length of the distributed areas by two wheel loads.

Therefore, the proposed pressure distribution model for the two wheel loads on the flexible pavement over the top slab of a low-fill box culvert is shown in Figure 6.4.
FIGURE 6.4
Distributed Pressure on the Top Slab of the Culvert under a Flexible Pavement
6.3 Comparison of Calculated Vertical Pressures

6.3.1 Culvert under Rigid Pavement

The average vertical pressure on the culvert under a rigid pavement was calculated using the simplified distribution method described in Section 6.2.1. The elastic moduli used in the calculation of the pressure distribution angles were same as the elastic moduli used in the parametric study in Chapter 5. Table 6.1 shows the distribution angle calculated based on the simplified model and the elastic modulus of each layer.

<table>
<thead>
<tr>
<th>Pavement layer</th>
<th>$E_1$ (MPa)</th>
<th>$E_2$ (MPa)</th>
<th>Distribution angle, $\alpha_1$ (°)</th>
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<td>Concrete pavement/cement-treated base</td>
<td>24545</td>
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<tr>
<td>Cement-treated base/lime-treated subgrade</td>
<td>5171</td>
<td>310</td>
<td>64.5</td>
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<tr>
<td>Lime-treated subgrade/natural subgrade</td>
<td>310</td>
<td>12</td>
<td>71.7</td>
</tr>
<tr>
<td>Natural subgrade/culvert slab</td>
<td>12</td>
<td>-</td>
<td>27.0</td>
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</table>

In Table 6.1, the elastic modulus $E_2$ used for the calculation of the distribution angle in the lime-treated subgrade is the elastic modulus of the natural subgrade, which was 12 MPa. The calculated pressure can be compared with the maximum pressure computed by the numerical model as discussed in Chapter 5. As discussed in Chapter 5, the maximum vertical pressure on the culvert in the small span (1.8 m) was higher than those in the large span (3.6 and 5.4 m) with the same top culvert slab thickness. Within the increase of the top slab thickness, the maximum vertical pressure on the culvert in the large span approached to that of the small span. Therefore, it is conservative to compare the maximum vertical pressure calculated by the numerical method based on the culvert with a small span. Table 6.2 shows the vertical pressures calculated by the proposed simplified method and the numerical model for the 1.8 m span. Clearly, the vertical pressures calculated by the proposed simplified method match those from the numerical method.
well. The distribution methods in the 2007 AASHTO LRFD Bridge Design Specifications and the 1992 AASHTO Standard Specifications for Highway Bridges significantly overestimated the maximum pressures, especially on the culverts with fill depths less than 1.2 m. The difference became smaller when the fill depth increased.

### TABLE 6.2
Vertical Pressures from Simplified and Numerical Methods

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<th>Fill depth (m)</th>
<th>Pavement thickness (m)</th>
<th>Simplified method</th>
<th>Numerical method</th>
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<th>AASHTO LRFD H distribution</th>
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<td>5.2</td>
<td>5.4</td>
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</tr>
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<td></td>
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<td>5.8</td>
<td>4.7</td>
<td>5.4</td>
<td>11</td>
</tr>
<tr>
<td></td>
<td>0.3</td>
<td>5.7</td>
<td>4.4</td>
<td>5.4</td>
<td>11</td>
</tr>
<tr>
<td></td>
<td>0.35</td>
<td>5.6</td>
<td>3.9</td>
<td>5.4</td>
<td>11</td>
</tr>
</tbody>
</table>

### 6.3.2 Culvert under Flexible Pavement

Vertical pressure on the culvert under a flexible pavement was calculated using the simplified distribution method described in Section 6.2.2. The elastic moduli used in the calculation of the pressure distribution angles were same as the elastic moduli used in the parametric study in Chapter 5. Table 6.3 shows the distribution angle calculated based on the simplified model and the elastic modulus of each layer.
TABLE 6.3  
Distribution Angle between Pavement Layers

<table>
<thead>
<tr>
<th>Pavement layer</th>
<th>$E_1$ (MPa)</th>
<th>$E_2$ (MPa)</th>
<th>Distribution angle, $\alpha_1$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt concrete/lime-treated subgrade</td>
<td>1827</td>
<td>310</td>
<td>45.0</td>
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<tr>
<td>Lime-treated subgrade/natural subgrade</td>
<td>310</td>
<td>12</td>
<td>71.7</td>
</tr>
<tr>
<td>Natural subgrade/culvert slab</td>
<td>12</td>
<td>-</td>
<td>27.0</td>
</tr>
</tbody>
</table>

In Table 6.3, the elastic modulus $E_2$ used for the calculation of the distribution angle in the lime-treated subgrade is the elastic modulus of the natural subgrade, which was 12 MPa. The calculated pressure was then compared with the maximum pressure magnitude calculated from FLAC3D numerical model. Table 6.4 shows the pressure distribution from proposed simplified method closely matched the pressure calculated from FLAC3D numerical model.

TABLE 6.4  
Vertical Pressures from Simplified and Numerical Methods

<table>
<thead>
<tr>
<th>Fill depth (m)</th>
<th>Pavement thickness (m)</th>
<th>Vertical pressure (kPa)</th>
<th>Simplified method</th>
<th>Numerical method</th>
<th>AASHTO standard specification distribution</th>
<th>AASHTO LRFD H distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.6</td>
<td>0.15</td>
<td></td>
<td></td>
<td>38.8</td>
<td>38.1</td>
<td>62.4</td>
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<tr>
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<td>62.4</td>
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<tr>
<td></td>
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<td>30.9</td>
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<td>62.4</td>
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<tr>
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<tr>
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<tr>
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<td>8.8</td>
<td>7.5</td>
<td>5.4</td>
</tr>
</tbody>
</table>
6.5 Summary

Simplified methods were developed in this study to predict the vertical pressure on a culvert under a rigid or flexible pavement. Three conclusions can be drawn from this development:

1. In the case of the culvert under a rigid pavement, the total wheel load can be distributed uniformly over the combined area with the extremities of the distributed areas by individual wheels. The calculated vertical pressure by the simplified method was in good agreement with the maximum vertical pressure by the numerical method.

2. In the case of the culvert under a flexible pavement, the vertical pressure within the overlapped area can be obtained by the superposition of the pressures due to individual wheels. The calculated vertical pressure by the simplified method was in good agreement with the maximum vertical pressure by the numerical method.

3. The distribution methods in the 2007 AASHTO LRFD Bridge Design Specifications and the 1992 AASHTO Standard Specifications for Highway Bridges significantly overestimated the vertical pressures, especially on the culverts with fill depths less than 1.2 m if a rigid or flexible pavement existed.
Chapter 7: Conclusions and Recommendations

This chapter summarizes the research work carried out in this study, draws conclusions based on the experimental and numerical results, and makes recommendations for future research.

7.1 Summary of Research Work

This research addressed the improved load distribution for low-fill box structures under rigid and flexible pavements. The objective and scope of this study were outlined in Chapter 1. Chapter 2 included a comprehensive literature review of the research work relevant to this study along with the classification of box culverts, descriptions and requirements of load rating methods, and constitutive models for geomaterials.

The load distribution on the top slab of the culvert depends largely on the depth of fill above the culvert, the type and the thickness of the pavement (rigid or flexible) over the culvert, and the span of the culvert. Therefore, this study was focused on the effect of pavements on the distribution of the wheel load over the top slab of the culvert.

Chapter 3 reported two field tests carried out on the culverts under rigid and flexible pavements. The culverts were subjected to static and moving traffic loadings. The deflections of the top slabs of the culverts were monitored during both tests. Earth pressures were monitored in an unsurfaced section under static loading. Laboratory tests were conducted to determine the properties of the pavement layers including the concrete, HMA, base, subgrade, and backfill soil obtained from the field. The culvert responses and material properties were used to validate the numerical model created in FLAC3D in Chapter 4.

Chapter 5 presents a parametric study carried out to investigate the effects of different influence factors (i.e., type of pavement, pavement thickness, depth of fill, and culvert span) on the vertical pressure distribution. The material properties used in the parametric study were similar to those used in the validation of the models in Chapter 4. The vertical pressure distribution over the top slab of the culvert due to a wheel load was monitored along the culvert axis and perpendicular to the culvert axis. In addition, the vertical pressure distribution obtained from the numerical method was compared with that in the AASHTO guidelines. In Chapter 6,
simplified methods were developed to estimate the vertical pressures on a box culvert under rigid and flexible pavements.

7.2 Conclusions of Research

7.2.2 Culvert under Rigid Pavement

The following conclusions can be drawn from the field and parametric study carried out to investigate the distribution of the wheel load on the culvert under a rigid pavement considering the influence of pavement thickness, fill depth, and span of the culvert:

1. The magnitude of vertical pressure on the top slab of the culvert gradually decreased as the pavement thickness increased due to the distribution of the wheel load over a wider area.

2. The magnitude of vertical pressure decreased gradually with an increase of the depth of fill over the culvert. The rate of pressure reduction with the depth of fill was higher at a small fill depth. The effect of the traffic load on the culvert was higher at a low fill depth and gradually decreased with an increase of the depth of fill.

3. The distribution of the vertical pressure for a fill depth of 0.6 m was characterized by a peak pressure under the wheel load. The pressure in the middle of the axle was smaller than the peak pressure. Under such a fill depth, there was little interaction between the distributed pressures by the wheel loads. However, when the fill depth was increased to 1.2 m and greater, the peak pressure was located in the middle of the axle. The interaction between the distributed pressures by wheel loads was developed at fill depths of 1.2 m and greater.

4. The vertical pressure on the top slab of the culvert decreased with increasing culvert span length. The influence of the span was greater for a smaller fill depth. The rate of change in the vertical pressure on the top slab of the culvert with the span decreased with increasing span length. The increase of the culvert top slab thickness reduced the slab deflection and increased the maximum vertical pressure on the culvert.
5. The AASHTO guidelines for the vertical pressure distribution are overly conservative for a wheel load on a low-fill box culvert under a rigid pavement. Use of the AASHTO guidelines resulted in much higher estimated pressures than those obtained by the numerical method down to a fill depth of 1.2 m. The AASHTO guidelines are still conservative for a rigid pavement with fill depths of 1.8 and 2.4 m. However, their difference in the estimated pressures from the two methods decreased with an increase of the fill depth. Therefore, the wheel load distribution used by the AASHTO guidelines for load rating of a culvert under a rigid pavement would give a lower rating factor than the numerical method since the pressure distribution is conservative. The rating factor of a low-fill culvert can be significantly increased if the effect of the rigid pavement is considered in load rating.

6. At all fill depths considered, the distributed vertical pressure calculated by the 2007 AASHTO LRFD Bridge Design Specifications was higher than that by the 1992 AASHTO Standard Specifications for Highway Bridges. For greater fill depths, the vertical pressure distribution from the AASHTO Standard Specifications closely matched that from the numerical method.

7. For the case of the culvert under a rigid pavement, the total wheel load can be distributed uniformly over the combined area with the extremities of the distributed areas by individual wheels. The vertical pressures calculated using the simplified method were in good agreement with the maximum vertical pressures calculated using the numerical method.

7.2.3 Culvert under Flexible Pavement

The following conclusions can be drawn from the field and parametric analysis carried out to study the distribution of wheel loads on culverts under flexible pavements considering the influence of pavement thickness, fill depth, and span of the culvert.
1. The magnitude of vertical pressure on the top slab of the culvert gradually decreased as the pavement thickness increased due to the distribution of the wheel load over a wider area.

2. The magnitude of vertical pressure decreased gradually with an increase of the depth of fill over the culvert. The rate of pressure reduction with the fill depth was higher for a small fill depth. The effect of the traffic load on the culvert was greater for smaller fill depths and gradually decreased with increasing depth of fill.

3. The distribution of the vertical pressure for a fill depth of 0.6 m was characterized by a peak pressure under the wheel load and the pressure in the middle of the axle was smaller than the peak pressure. Under such a fill depth, there was little interaction between the distributed pressures from the wheel loads. However, when the fill depth was increased to 1.2 m and greater, the peak pressure was located in the middle of the axle. The interaction between the distributed pressures by wheel loads was developed at the fill depth of 1.2 m and greater.

4. The vertical pressure on the top slab of the culvert decreased with the increase of the culvert span length. The influence of the span length was greater for smaller fill depths. The rate of change in the vertical pressure on the top slab of the culvert with the span decreased with the increase of the culvert span length. The increase of the slab thickness reduced the slab deflection and increased the maximum vertical pressure on the culvert.

5. For the culvert under a flexible pavement and a fill depth up to 1.2 m, the AASHTO pressure distribution methods calculated higher vertical pressures than the numerical method. The difference in the calculated vertical pressures from the numerical method and the AASHTO distribution methods decreased with the increase of fill depth and became small at fill depths between 1.8 and 2.4 m. Therefore, the wheel load distribution used by the AASHTO guidelines for load rating of a culvert under a flexible pavement would give a lower rating factor since the pressure distribution is conservative. The rating factor of a low-fill
culvert can be increased if the effect of the flexible pavement is considered for load rating.

6. For all fill depths considered, the vertical pressure estimated by the 2007 AASHTO LRFD Bridge Design Specifications was higher than that by the 1992 AASHTO Standard Specifications for Highway Bridges. For greater fill depths, the vertical pressure distribution by the AASHTO Standard Specifications closely matched that by the numerical method.

7. For the case of a culvert under a flexible pavement, the vertical pressure within the overlapped area can be obtained by the superposition of the distributed vertical pressures due to individual wheels. The vertical pressures calculated using the simplified method were in good agreement with the maximum vertical pressures using the numerical method.

7.3 Future Research

This research investigated the effect of pavement type (rigid or flexible), pavement thickness, fill depth, and span on the load distribution for load rating of low-fill box structures. Future research is needed to address the following issues:

1. Field tests should be carried out by installing pressure cells under the pavement section to study the effect of the pavement on the pressure distribution on the top of the culvert. The field data will help further verify the numerical results obtained in this research.

2. The numerical analysis done in this study was only for single-cell culverts. The potential for interaction between multiple cells should be investigated in the future.

3. The elastic moduli of the pavement layers and the backfill soil were kept constant. A future study is needed to evaluate the effects of the elastic moduli of the pavement layers and the backfill soil.

4. In this study soil was modeled as an elastic material. An advanced soil model may be used to verify the numerical results obtained in this study.
5. This study was carried out by assuming the unyielding foundation condition. A future study is needed to investigate the effect of a yielding foundation on the pressure distribution on culverts.
References


American Concrete Institute (ACI) (2011). *Building Code Requirements for Structural Concrete, ACI 318*.


Delaware Department of Transportation (2005). *Bridge Design Manual*.


Iowa Department of Transportation, Office of Bridges and Structures (2005). *Bridge Design Manual*.


Appendix: Proposed Revisions to AASHTO LRFD Bridge Design Specifications

Based on the results and findings in this research, the following revisions to the AASHTO LRFD Bridge Design Specifications are suggested. The underlined words are the suggested changes or additions.

3.6.1.2.6 - Distribution of Wheel Loads through Pavement Layers and Earth Fills

Where the depth of fill is less than 2.0 ft, live loads shall be distributed to the top slabs of culverts as specified in Article 4.6.2.10.

For a design purpose, the effect of pavement layers on load distribution should be ignored to be conservative. In lieu of a more precise an analysis, or use of other acceptable approximate methods of load distribution permitted in Section 12, where the depth of fill is 2.0 ft or greater, wheel loads may be considered to be uniformly distributed over a rectangular area with sides equal to the dimension of the tire contact area, as specified in Article 3.6.1.2.5, and increased by either 1.15 times the depth of the fill in select granular backfill, or the depth of the fill in all other cases. The provisions of Articles 3.6.1.1.2 and 3.6.1.3 should apply. Where such areas from several wheels overlap, the total load within the overlapped area should be uniformly distributed over the area.

For a load rating purpose, the effect of pavement layers on load distribution through pavement layers and earth fills should be considered by a stress distribution method. The load distribution angle from the upper pavement layer to the underlying pavement layer or earth fill can be approximately calculated by

\[ \tan \alpha_1 = \tan \alpha_0 \left[ 1 + 0.204 \left( \frac{E_1}{E_2} - 1 \right) \right] \]

\( \alpha_1 \) = distribution angle from the upper to the underlying layer
\( \alpha_0 \) = distribution angle in a uniform medium including the earth fill (i.e., 30° in select granular backfill or 27° in all other fills including pavement layers)
\( E_1 \) = elastic modulus of the upper pavement layer under consideration.
\( E_2 \) = elastic modulus of the underlying layer

The distributed area of a wheel load on a buried structure is calculated by the distribution angles through different layers with different thicknesses.

Under a rigid pavement, the total distributed area from different wheels is that formed by the outer boundary of individual distributed areas and the total load should be uniformly distributed over the total area.

Under a flexible pavement, where such areas from wheels overlap, the total load within the overlapped area should be super-positioned and uniformly distributed over the overlapped area.

C3.6.1.2.6

For a design purpose, a worst loading condition should be considered to be conservative no matter whether a pavement will be placed over earth fills. Under such a condition, unpaved earth fills may be subjected to wheel loads during construction.

Elastic solutions for pressure within an infinite half-space by loads on the ground surface can be found in Poulos and Davis (1974), NAVFAC DM-7.1 (1982), and soil mechanics textbook.

This approximation is similar to the 60-degree rule for select granular back fill or the 2 (vertical) to 1 (horizontal) distribution method found in many texts on soil mechanics.

For a load rating purpose, pavement layers exist on earth fills. The effect of pavement layers on load distribution through pavement layers and earth fills should be considered. Elastic layered theory or Burmister’s solution (Burmister, 1958) found in pavement textbooks can be used to calculate the distributed pressure on buried structures.

The approximation proposed by Giroud and Han (2004) can be used to estimate the distributed pressure through pavement layers by the distribution angle method. Through the earth fills below pavement layers, this approximation is similar to the...
60-degree rule for select granular back fill or the 2 (vertical) to 1 (horizontal) distribution method for all other fills.

The modulus of concrete can be determined by unconfined compression test or American Concrete Institute ACI 318. The moduli of asphalt surface, base course, subbase, and subgrade can be determined experimentally or estimated using the 1993 AASHTO Guide for Design of Pavement Structures.

The distributed pressure of wheel loads on a buried structure through a rigid pavement can be estimated based on the following figure.

The distributed pressure of wheel loads on a buried structure through a flexible pavement can be estimated based on the following figure.

The dimensions of the tire contact area are determined at the surface based on the dynamic load allowance of 33 percent at depth = 0. They are projected through the soil as specified. The pressure intensity on the surface is based on the wheel load without dynamic load allowance. A dynamic load allowance is added to the pressure on the projected area. The dynamic load allowance also varies with depth as specified in Article 3.6.2.2. The design lane load is applied where appropriate and multiple presence factors apply.
K-Tran

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Ad Astra Per Aspera