Bedding and Fill Heights for Concrete Roadway Pipe and Box Culverts

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EXECUTIVE SUMMARY

The overall objective of the proposed research is to investigate the fundamental interactions that take place during the process of excavating a trench, preparing the bedding, installing the conduit, and then placing and compacting the backfill. The materials and procedures used will significantly affect the conduit performance. The imposed loading will be greatly affected by the relative settlement of the soil prism directly above the conduit. An improved understanding of these fundamentals will be essential in order to develop technically better and more economical specifications for both designers and contractors.

The specific objectives of this research were :1) to increase our understanding of the fundamental soil-structure interactions; 2) to compare the ALDOT Specifications, 2002 with current Standard Specifications of AASHTO and ASCE in three areas (material, design, and construction) and to update the ALDOT Specifications, 2002 where necessary; 3) to develop improved design guides which include design guides and examples, drawings, tables for Maximum Fill Heights, proposed equations and graphs for vertical earth load reduction rates, and instructions for recommended Finite Element Analysis programs (SPIDA, CANDE-89); and 4) to develop detailed construction guides, including imperfect trench installation.

This research focused on obtaining a comprehensive understanding of fundamental soil-structure interactions, especially the influence of bedding and fill heights for the behavior of rigid conduits (concrete roadway pipe and box culverts). To meet the objectives of this research project, the following main tasks were carried out: 1) Review ALDOT Specifications, 2002 and neighboring states' procedures for installing roadway conduits; 2) Review the literature, including technical papers, manufacturer and trade association recommendations, and FHWA(1999)/AASHTO(2002) Specifications/standard practices; 3) Compare existing ALDOT design procedures with AASHTO and ASCE design procedures; 4) Develop Standard Installations for concrete pipes and box culverts that take into account backfill material properties; 5) Develop improved and expanded design guides for imperfect trench installations for rigid conduits; 6) Conduct finite element analyses to identify and verify the proposed backfill procedures and backfill material requirements. One of the most commonly used programs for the analysis of roadway conduits, CANDE-89 and the comprehensive finite element computer program for the analysis and design

of buried concrete pipe, SPIDA were used. ABAQUS version 6.3.1 and MSC/NASTRAN (2004), general finite element programs were also used to simulate and analyze a finite element model of the soil-structure system; and 7) Develop a recommended set of rigid conduits installation procedures that include Special Provisions, Special Highway Drawings, Maximum Fill Height tables without imperfect trench installation, and Maximum Fill Height tables with imperfect trench installation, including computer codes.

Numerical analyses focused on the interaction of bedding and fill heights for deeply buried rigid conduits under embankment installation and imperfect trench installation. The magnitude of the earth load transmitted to conduit is largely affected by the installation method and bedding type. Bedding under a conduit has generally been compacted in order to control the conduit grade by minimizing settlement after construction. The results of computer modeling indicate that uncompacting the middle bedding reduces both the load on the conduit and the invert bending moments. However, the sidefill area should be compacted in order to provide support to the conduit and to provide an alternate vertical load path around the bottom of the conduit. The results from the field tests and the finite element analyses show the maximum bending moments are greater in an untreated sidefill than in a treated sidefill, which means that the sidefill area of the concrete pipe contributes significantly to supporting the earth load.

The analyses of the concrete pipe were based on four AASHTO Standard Installations developed by ACPA and adopted by AASHTO. A Type 4 Standard Installation has no construction or quality requirements. A Type 1 Standard Installation requires the highest construction quality. Required construction quality is reduced for a Type 2 Standard Installation, and reduced further for a Type 3 Standard Installation. The results from finite element analyses and ACPA show that the difference in the earth loads between Type 1, Type 2, and Type 3 is less than 5%. Proposed Installations used in this study consist of a combination of TREATED and UNTREATED installations, which conform to Type 3 and Type 4 AASHTO Standard Installations, respectively. Therefore, Proposed Installations are an effective installation method for maximizing bedding effect with the least effort for construction quality.

AASHTO specifications include no provisions for the requirements and geometry of sidefill for box culverts. However, this study showed that the characteristics of foundation and compaction of side fill have a significant influence on the behavior of box culverts. In this study, the effects of different foundations and compaction levels on the side fill of box culverts were evaluated and quantified using finite element analysis.

The primary objective of the numerical analysis was to evaluate the effect of the main variables and the effectiveness of imperfect trench installation. The width, height, location, and properties of soft materials were used as the parameters for imperfect trench installation in this study. The results of the analyses showed that reduction rates are highly affected by Modulus of Elasticity (E_s) of soft materials, which means the reduction rate is a function of E_s. Based on the results of numerous parameter studies with Finite Element programs, proposed equations for the reduction rates were derived for both concrete roadway pipe and box culverts by means of a linear regression method.

The primary results of this study are as follows: 1) The behavior of concrete roadway pipe and box culverts is more significantly affected by the installation practices for the bedding, haunch, and lower side than by foundation characteristics such as yielding or unyielding; 2) Optimum geometries for the soft zones of an imperfect trench installation were developed by numerous parameter studies for the height, the distance from the top of the conduit, and the width of the soft zone. Optimum geometries of the soft zone were proposed as Geometry I and Geometry II for concrete pipe and box culverts. The upper half of the pipe is surrounded by soft material in Geometry II for pipes and in Geometry II for box culverts, the whole sidewall is surrounded by soft material. Geometry II is more effective than Geometry I for earth load reduction. Therefore, only Geometry II was used in the Special Provisions and Special Highway Drawings of ALDOT Specifications, 2002; 3) Total vertical earth loads or bottom loads on the box culverts are composed of the top earth load, dead load, and shear force on the sidewall. Therefore, the design loading of box culverts should be based on the bottom pressure. AASHTO provisions for the design loading of box culverts are unconservative, as AASHTO provisions do not consider the shear force on the sidewall; 4) TREATED sidefill for the box culverts is effective in reducing the shear force occurring on the sidewall of box culverts. AASHTO has no provisions for the requirements and geometry of sidefill for box culverts. For convenience in the installation and design process, installations for box culverts are proposed similarly to those of pipes in this study; and 5) Imperfect Trench Installation for box culverts increases the shear force on the sidewall due to the reverse arching effect of imperfect trench installation as well as the reduction of top earth pressure and total vertical earth load. The preventive method for the increase of shear force, Geometry II for the soft zone, is designed to maximize the reduction effect of the vertical earth load due to the imperfect trench installation. Increase in the shear force due to the imperfect trench method can be prevented through the installation of soft material between the sidewall and soil. Using Geometry II for the soft zone of box culverts is thus highly effective in relieving any increase of the downward shear force on the sidewall.

Sections 524, 530, 850 and RPC-530 of the ALDOT Specifications, 2002 are revised in the form of Special Provisions and Special Highway Drawings. During the course of this study, it become clear that many provisions in current AASHTO and ASCE Specifications in the materials, design, and construction for concrete pipes and box culverts appear to be disjointed and confusing. Therefore, a summary review on current AASHTO and ASCE provisions on concrete pipes and box culverts for fast reference is provided in Appendix F.

Studies of the flexible pipes and experimental field tests are recommended for future research. Since a flexible pipe has very little inherent strength, the backfill plays a vital role in the overall response of the pipe-soil system. Research on flexible pipes should focus on investigating the overall interaction of pipe wall stress and strain, pipe deflections, and the buckling capacity of flexible pipes. In the next phase of this research program, a detailed examination of the behavior of these flexible pipes will be undertaken.

Experimental field studies of buried rigid conduits would also be desirable in order to validate the results of the Finite Element Analyses presented in this study. In addition, experiments on selected samples of soft material may be necessary to augment the properties obtained from the literature.

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Introduction

1.1 Background

The behavior of roadway pipes (concrete, metal and plastic) and concrete box culverts (hereafter referred to as roadway conduits) is significantly affected by installation practices. No rigid or flexible pipe products in use today can carry all superimposed loads without depending to at least some extent on the surrounding soil for support. In the case of round pipes, bedding must be uniform in order to prevent point loads, and the lateral pressure at the sides of the pipe must be strong enough to restrain displacements. The loads imposed on a roadway conduit are thus closely related to the installation practices. As the backfill conditions and the installation practices are important for the performance of roadway conduits, it becomes incumbent upon the designer and the contractor to ensure that the backfill assumptions and installation schemes specified in the design are strictly adhered to in the field during construction. Recent failures of concrete roadway pipe on Corridor X point to either design or construction problems. However, because of the small number of reported failures, the current design methods are likely to be conservative and, hence, result in installations that are more costly than necessary.

Installation standards for roadway conduits have not been thoroughly reviewed and updated significantly for many years since the work of Marston, Spangler, and others during the first half of the twentieth century [1, 2, 3, 47]. Some of the current installation standards use terminology that is outdated and unsuitable for current construction contracts. Bedding conditions presented in current references, such as the ASCE Manual of Practice No. 37 [40], Concrete Pipe Design Manual [38], Concrete Pipe Technology Handbook [39] and Concrete Pipe Handbook [41] continue to present installation details based on this early work. Outdated terminology includes such vague terms as “granular material,” “backfill,” “fine granular fill”, and “soil” for soils. The compaction requirements are expressed using vague terminology such as “densely compacted,” “carefully compacted,” “lightly compacted,” “compacted,” and “loose.” These terminologies for backfill materials and compaction levels are difficult to interpret and rely heavily on the expertise and experience of contractors and inspectors, which today in Alabama is at very low levels.

Advances in modern finite element methods have made it possible to quantify these complex interactions [62]. According to a recently complete Auburn University Highway Research Center project [42], it appears very possible to reduce the imposed loading on roadway conduits by implementing an imperfect trench installation. Consideration of improved analysis techniques and new materials, for example some form of expanded polystyrene, for the imperfect trench method could result in a reduced reinforcing steel areas for roadway conduits with high fills, which a correspondingly enormous impact on the overall safety and economic issues.

1.2 Objectives

The overall objective of this research was to investigate the fundamental interactions that take place during the process of excavating a trench, preparing the bedding, installing the conduit, and then placing and compacting the backfill. The materials and procedures used significantly affect the conduit performance. The imposed loading is greatly affected by the relative settlement of the soil prism directly above the conduit. An improved understanding of these fundamentals is essential to develop technically better and more economical specifications both the designers and contractors.

The specific objectives of this research were as follows:

1) Increase our understanding of the fundamental soil-structure interaction.
2) Update Sections 524, 530, 850, and RPC-530 of the ALDOT Specifications, 2002 in the form of Special Provisions and Special Highway Drawings.
3) Develop improved design guides which include the following items
    - Proposed Installations
    - Design guides for pipe selection for a range of soil-structure systems, including Special Highway Drawings and examples
    - Maximum Fill Height tables without imperfect trench installation
    - New equations and graphs for Maximum Fill Heights with imperfect trench installation
    - Simple pipe selection schematic diagrams (flowchart)
    - Instructions for using recommended Finite Element Analysis programs

(SPIDA, CANDE-89)

4) Develop detailed construction guides, including new imperfect trench installation.

1.3 Scope

This research focused on obtaining a comprehensive understanding of the fundamental soil-structure interaction, especially the influence of bedding and fill heights, for the behavior of rigid conduits (concrete roadway pipe and box culverts). This includes:

1) A review of ALDOT Specifications, 2002 and neighboring states' procedures for installing roadway conduits
2) A comprehensive Review of the literature, including technical papers, manufacturer and trade association recommendations, and FHWA (1999)/AASHTO (2002) Specifications/standard practices.
3) A comparison of existing ALDOT procedures with current AASHTO and ASCE procedures
4) Develop Proposed Installations for concrete pipes and box culverts that take into account backfill material properties
5) Develop improved and expanded design guides of imperfect trench installation for rigid conduits
6) Conduct finite element analyses to identify/verify the proposed backfill procedures and backfill material requirements. The most commonly used programs for the analysis of roadway conduits, CANDE-89 and SPIDA were used. ABAQUS and MSC/NASTRAN, general finite element programs were also used.
7) Develop a recommended set of rigid conduit installation procedures that include Special Provisions, Special Highway Drawings, Maximum Fill Height tables without imperfect trench installation, and Maximum Fill Height tables with imperfect trench installation, including computer codes

1.4 Summary of Research

Chapters 2, 3, and 4 present a review of the current AASHTO and ASCE procedures and an analysis of the recommended set of rigid conduit installation procedures that include consideration of new imperfect trench installations. Chapter 5 presents Special Provisions for Sections 524, 530 and 850 of ALDOT Specifications, 2002. Chapter 6 presents the improved design guides developed in this research, which includes the following items:

- Proposed Installations
- Design guides for pipe selection for a range of soil-structure systems, including Special Highway Drawings and examples
- Maximum Fill Height tables without imperfect trench installation
- Maximum Fill Height tables with imperfect trench installation
- Simple pipe selection schematic diagrams (flowchart)
- Instructions for using the recommended Finite Element Analysis programs (SPIDA, CANDE-89)

Chapter 7 presents detailed construction guides that include developed imperfect trench installations. Finally, Chapter 8 presents conclusions and recommendation for future study.

CHAPTER 2

Literature Review

2.1 General

The main role of roadway conduits is to transport water and other fluids and the design and construction of buried structures is one of the most important functions undertaken by public works engineer. The major engineering challenge for a buried structure is the mechanism with which the structure withstands the earth load imposed on it. The analysis, design, and installation of buried structures thus require an extensive understanding of soil-structure interactions.

This chapter presents the current buried structures installation practice based on a review of the technical literature, current standard specifications, and surveys of current users.

2.2 Background and Theory

2.2.1 History of Early Research and Design

American concrete pipe structural design practice was born near the beginning of the twentieth century. In 1910, Anson Marston began his experiments and theoretical studies at Iowa State College's Engineering Experiment Station in Ames, Iowa. With help from A.N.Talbot of the University of Illinois and a theory for pressures in grain bins published by Janssen, Marston proposed a set of the equations for calculating the earth load on a pipe in a narrow trench [43].

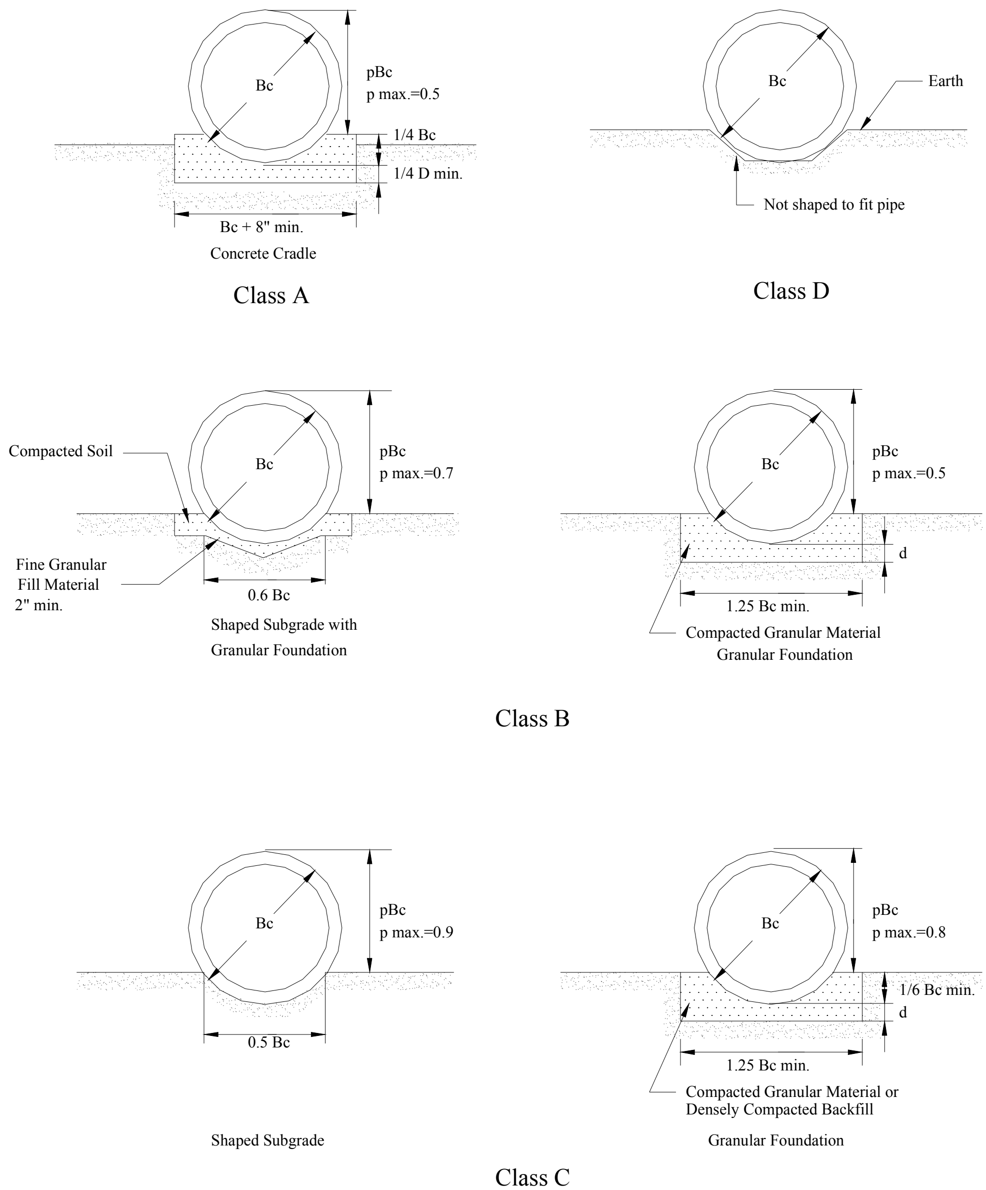
Marston and his co-workers at Iowa State then focused on the supporting strength of pipes in trenches, which is affected by bedding conditions, and developed methods for determining supporting strength using laboratory tests [44]. Between 1915 and 1917, they performed tests on pipes laid in trenches using several different types of bedding [45]. Marston defined the four types of bedding used as Class A, B, C, and D. The results of the research on "projecting" conduits at Iowa Experiment Station during the 1920s are presented in a comprehensive paper by Marston entitled " The Theory of External Loads on Closed Conduits in the Light of the Latest Experiments"[46]. This paper, which was published in 1929, summarizes Marston's theories for both trench conduits and embankment conduits at that time.

In the late 1920s and early 1930s, pipe research work at the Iowa Experiment Station was devoted to developing a method for defining the supporting strength of buried rigid culverts in embankment installations, termed projecting culverts. The results of this research were given in a comprehensive paper by M.G.Spangler, entitled “The Supporting Strength of Rigid Pipe Culverts” and published in 1933 as Bulletin 112 of the Iowa Experiment Station [47].

The objective of Spangler’s research was to determine the supporting strength of buried rigid pipe when subjected to the earth loads predicted by Marston’s theories of earth loads on projecting culverts. Supporting strength was defined as the load that caused cracking or, later, as the load that caused a specified width of crack such as 0.01in. Based on rational assumptions about pressure distribution and tests of pipe installations constructed with “ordinary bedding” and of pipes subjected to three-edge bearing loads, Spangler determined the ratio of the field earth load that cracks the pipe, to the three-edge bearing load that cracks the pipe [47].

Based on these studies, he related the ability of field installed pipe with various qualities of bedding to withstand earth loads determined using Marston’s theories to an equivalent smaller three-edge bearing load that produced the same cracking moment. He termed the ratio of the field load with a particular bedding and projection condition to the three-edge bearing load that produces the same invert moment, the “Load Factor” for that field condition. In current practice, this ratio is called the bedding factor, B_f.

As in the case of trench loads, Spangler realized that an installed pipe’s supporting strength in an embankment was greatly influenced by the quality of its bedding. Any method for determining supporting strength first required a definition of standard bedding types. For his research, Spangler defined the following four standard bedding types shown in Figure 2.1, which are similar to the beddings defined much earlier by Marston for trench conduits, except that for embankments the concept of projection ratio was introduced in addition to bedding type in order to reflect the effects of active lateral earth pressure in the embedment soil that is placed around the pipe above the natural subgrade.



Depth of Bedding
Material Below Pipe

D	d (min.)
27" & smaller	3"
30" to 60"	4"
66" & larger	6"

Legend

Bc = Outside Diameter of Pipe
H = Backfill cover above top of pipe
D = Inside diameter
d = Depth of bedding material below pipe

Figure 2.1 Four Types of Projection Bedding for Field Installation of Pipe

Spangler based his development of load factors relating the strength of a projecting conduit to the pipe's three-edge-bearing strength by making rational assumptions about the distribution of earth pressure around the circumference of a buried pipe with various bedding and projection conditions (based on limited field test data), and determining the moments at invert, crown, and springline by an elastic analysis of the pipe as a ring with uniform stiffness.

The design procedures developed between 1911 and 1932 cover pipe installed in trenches and pipe installed in embankments with positive projection ratios (i.e., where the top of the pipe projects above the natural subgrade of the embankment). Still missing was a procedure for determining earth loads and supporting strengths for pipes in a trench whose top was covered by an embankment(i.e., where the top of the pipe was below the natural subgrade), as shown in Figure 2.3(c). These installations are termed "negative projecting" installations. The supporting strength of negative projecting conduits is taken to be the same as the strength of trench conduits for the various trench bedding conditions described previously.

Spangler also applied the approach that he developed for negative projecting conduits to an installation type developed earlier by Marston to reduce the earth loads on embankments, which Marston called "the imperfect ditch" or "induced trench". In this installation, backfill is placed and thoroughly compacted on both sides and for some distance above a projecting embankment conduit. Then a trench is constructed in this compacted fill by removing a prism of soil having the same width as the conduit and refilling with very loose compressible material as shown in Figure 2.3(d). The embankment is then completed in the normal manner.

The loads on an imperfect ditch installation may be determined using the same procedure as that developed for negative projecting conduits. However, Spangler was not able to develop specific values for the settlement ratio, which is one of the parameters needed for determining loads [48]. The loads are minimized by maintaining a trench width equal to the pipe outside diameter and a trench fill that is much looser than the compacted fill of the sidewalls. The supporting strength is enhanced by the lateral pressure that acts on projecting embankment conduits.

Experiments demonstrated the validity of Spangler's prediction of greatly positive projecting culverts [48].

2.2.2 Classification of Buried Conduits

Loads on buried conduits have been shown to be dependent upon installation conditions. Because of the influence of these installation conditions and the importance of recognizing them when determining loads, installations of buried structures are classified into two broad categories, trench installations and embankment installations. Figure 2.2 shows the classification system for buried conduits.

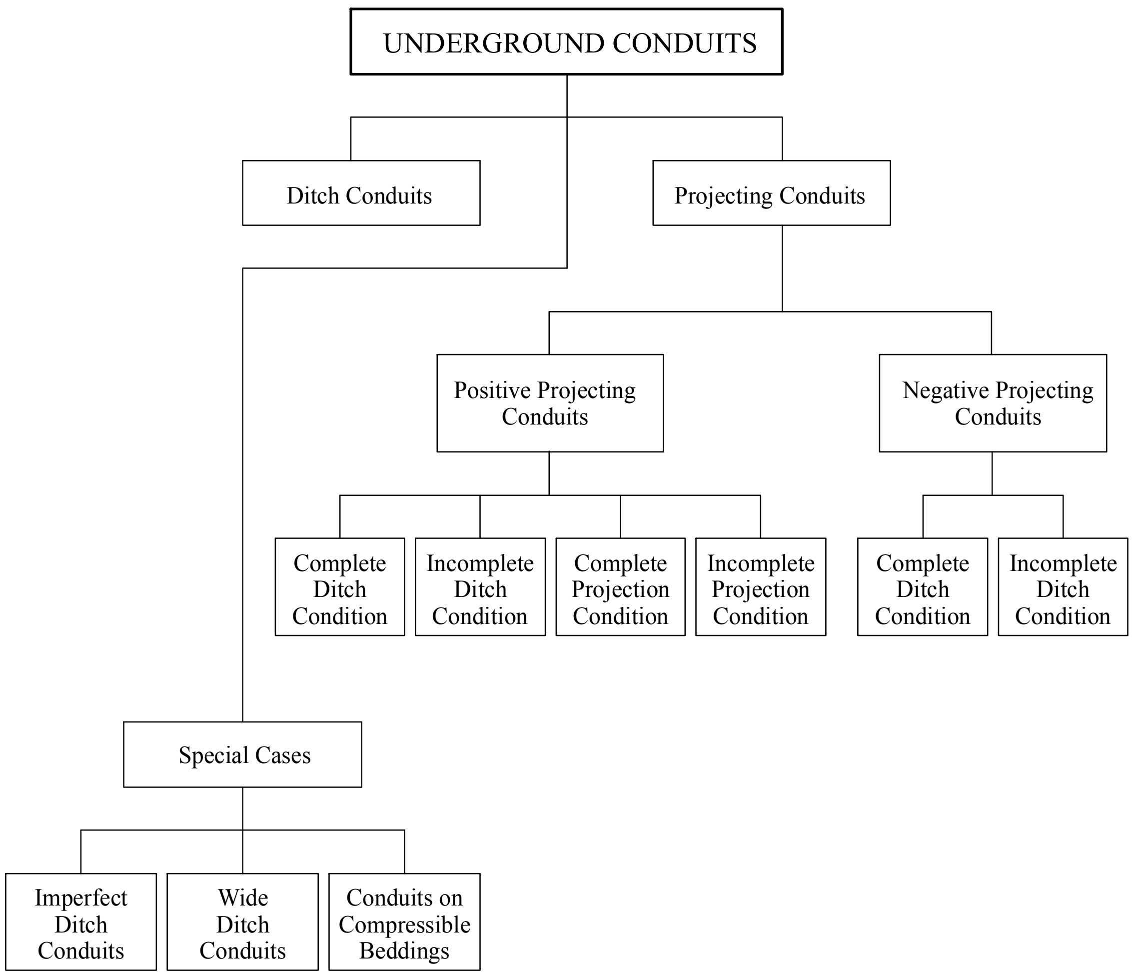
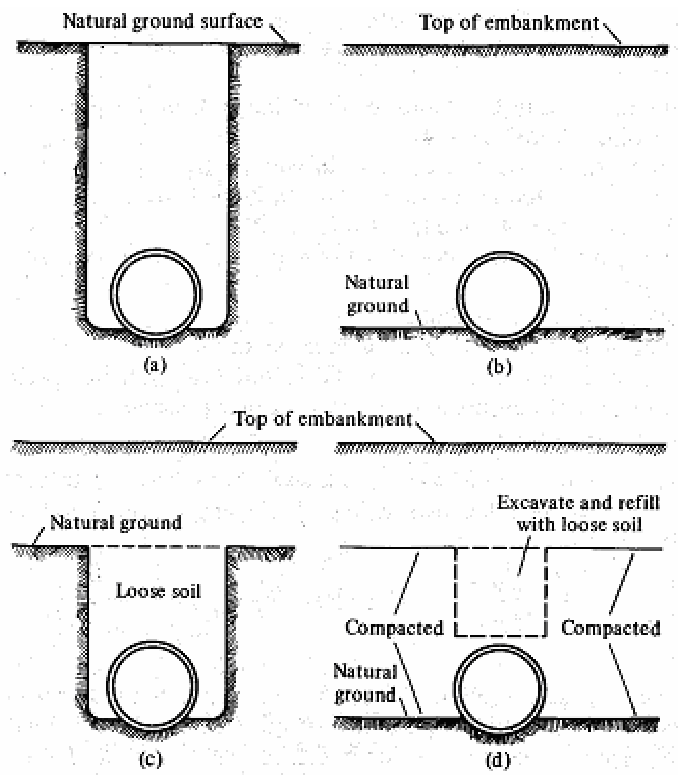


Figure 2.2 Construction Conditions for Underground Conduits [31]

Pipe installations are called trench installations when the pipe is located completely below the natural ground surface and the backfill over the pipe is placed between vertical or sloping walls of natural (in-situ) soil extending to the surface [39]. Frictional forces between the sides of the trench and the backfill material help to support the weight of the soil overlaying the pipe.

Pipe installations are called embankment installation when soil is placed in layers above the natural ground [39]. Embankment installations are further subdivided based on their location relative to the original ground level. Conduits founded partially or totally above the original ground level are classified as positive projecting conduits. Conduits founded in a trench excavated below the original ground level beneath the embankment are classified as negative projecting conduits. This is a very favorable method of installing a railway or highway conduit, since the load produced by a given height of fill is generally less than it would be in the case of a positive projecting conduit.



(a) Ditch Conduit (b) Positive Projecting Conduit

(c) Negative Projecting Conduit (d) Imperfect Ditch Conduit

Figure 2.3 Various Types of Conduit Installations

The imperfect trench conduit, sometimes called the induced trench conduit is installed with a compressible inclusion between the top of the conduits and the natural ground surface as shown in

Figure 2.3(d). The imperfect trench conduit is an important special case that is somewhat similar to the negative projecting conduit, but is even more favorable from the standpoint of reducing the load on the structure [41]. The four types of conduit installations are shown in Figure 2.3.

2.2.3 The Marston - Spangler Theory

2.2.3.1 General

The Marston theory of loads on buried conduits was developed near the beginning of the twentieth century. Earth load is the weight of the earth that must ultimately be carried by the pipe. This weight varies depending on the soil characteristics. More importantly, however, it varies with the installation conditions. The method for determining the earth loading is best approached by considering the two major classes of installation, trench and embankment.

M.G. Spangler presented three bedding configurations and used the concept of a bedding factor to relate the supporting strength of buried pipe to the strength obtained in a three-edge bearing test [56]. Spangler's theory proposed that the bedding factor for a particular pipeline and, consequently, the supporting strength of the buried pipe, is dependent on two installation characteristics: the width and quality of the contact between the pipe and bedding and the magnitude of the lateral pressure and the portion of the vertical height of the pipe over which it acts.

The soil around the conduit is initially divided into prisms by imaginary vertical lines that extend from either side of the conduit to the top of the embankment. The design equations are derived based on an analysis of the forces acting on a thin slice of soil located within the interior prism.

Earth loads on the buried conduits are predicted by applying a factor to the weight of soil overlaying the pipe. The load factor is calculated based on frictional forces that are assumed to develop along these vertical planes. The frictional forces are generated by differential settlements between the prism of soil directly above the pipe and those on either side. The direction of these shear forces can increase or decrease the load on the pipe depending upon the direction of the differential settlement between the two prisms, as shown in Figure 2.4. Greater settlement above the conduit results in earth pressures that are less than the overburden. Earth pressures greater than the overburden pressure occur when greater settlement occurs in the exterior prisms.

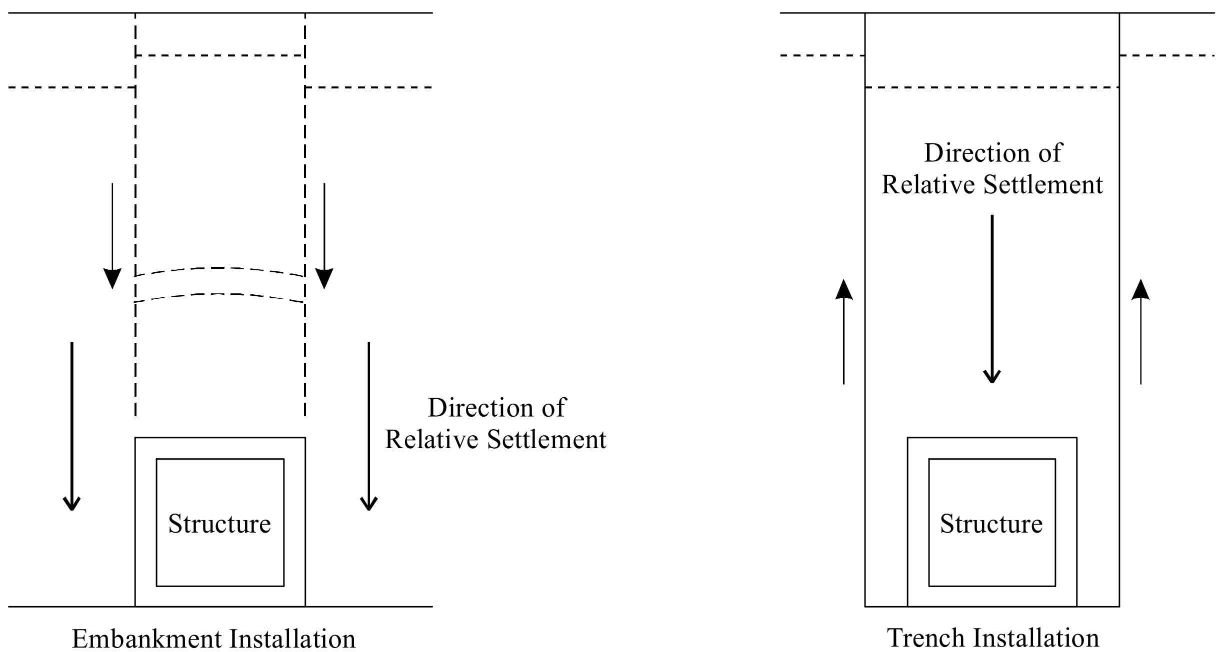


Figure 2.4 Arching Action within Fill Material for Buried Conduit Installations

2.2.3.2 Embankment and Ditch Conduits

The essential features of embankment installations are illustrated in Figure 2.3(b). Embankment installations are classified based upon their location relative to the original ground as being either positive or negative projection.

For positive projection, a conduit installed on a non-yielding foundation is considerably stiffer than the surrounding fill material. As a result, greater settlement will occur in the exterior prisms than within the interior prism. As the soil in the exterior prism moves downward relative to the interior prism, it exerts a downward force due to the frictional nature of the backfill material. The resulting load on the conduit is equal to the weight of the overlying soil plus the frictional forces.

For negative projection, a conduit installed in a narrow trench beneath an embankment is defined as a negative projection installation. The frictional forces between the fill material and sides of the trench decrease the earth load on the conduit. The earth load on the conduit equals the weight of the overlying soil less the frictional forces.

Further subclassification is based upon frictional forces within the backfill material. If the magnitude of relative settlement between the prisms is sufficient that the frictional forces extend to the surface of the fill, the pipe is defined as being in a complete condition. Conversely, if the frictional forces cease to exist at an imaginary horizontal plane within the fill, the pipe is defined as being in an incomplete condition.

Marston [2] and Spangler [32] quantified the load on conduits installed by different construction conditions by solving differential equations based on the equilibrium conditions of a simplified free body of prisms, and proposed equations for predicting loads on conduits due to earth fill as follows:

$W = C_d \gamma B_d^2$ for ditch conduits (2-1)

$W = C_c \gamma B_c^2$ for positive projecting conduits (2-2)

$W = C_n \gamma B_c^2$ for imperfect ditch conduits (2-3a)

$W = C_n \gamma B_d^2$ for negative projecting conduits (2-3b)

Where C_d, C_c, and C_n = load coefficients; B_d = the horizontal width of ditch; and B_c = the out-to-out horizontal span of the conduit. Although graphical diagrams are provided for the computation of coefficients, there are still many practical difficulties because the load coefficients proposed contain certain parameters that cannot be determined readily, such as the settlement ratio and the height of the plane of equal settlement. Graphical diagrams for C_d, C_c, and C_n are presented in Figures 2.5, 2.6, and 2.7. For load coefficients, C_c in Figure 2.6, the rays are straight lines that can be represented by equations when the value of H/B_c exceeds the limits of the diagram. These equations are given in Table 2.2. Symbols used in the figures are defined as follows:

H = height of fill above top of conduit,

B_d = horizontal width of ditch at top of conduit,

B_c = out-to-out horizontal span of conduit,

K = ratio of active lateral unit pressure to vertical and sides of ditch,

μ = $\tan\phi$ = coefficient of internal friction of fill material,

μ' = $\tan\phi'$ = coefficient of friction between fill material and sides of ditch,

p = projection ratio, the vertical distance from the natural ground surface to the top of the structure divided by the structure height,

p′ = projection ratio in negative projection or imperfect ditch installation, the depth of the ditch divided by its width,

r_{sd} = settlement ratio defined by

$$r_{sd} = \frac{(s_m + s_g) - (s_f + d_c)}{s_m}$$

where

s_m = compression strain of the side columns of soil of height pB_c,

s_g = settlement of the natural ground surface adjacent to the conduit,

s_f = settlement of the conduit into its foundation,

d_c = shortening of the vertical height of the conduit.

In order to use the Marston-Spangler equations, it is essential to predetermine the value of the settlement ratio. Although the settlement ratio, r_{sd}, is a rational quantity in the development of the load formula, it is very difficult to predetermine the actual value that will be developed in a specific case. Spangler and Handy [31] recommended design values of the settlement ratio based on observations of the performance of actual culverts under embankments, as shown in Table 2.1.

2.2.3.3 Induced Trench Conduits

The induced trench installation attempts to simulate the benefits of a trench installation in an embankment situation. A compressible layer typically comprised of organic material, such as baled straw or woodchips, is incorporated within the backfill directly over the pipe. The compressible layer induces greater settlement within the interior prism than the exterior prisms. This relative settlement develops frictional forces within the backfill similar to those in a trench installation. The frictional forces transfer a portion of the weight of the soil in the interior prism to the exterior prisms, effecting a significant reduction in the earth load on the conduit. The induced trench installation developed by Spangler (1950) is presented in Figure 2.3(d). The earth load equations for the induced trench method are identical to the negative projecting condition, with the exception of the projection ratio. For an induced trench installation, the projection ratio, p', is defined as the ratio of the thickness of the compressible layer to the outside diameter of the conduit.

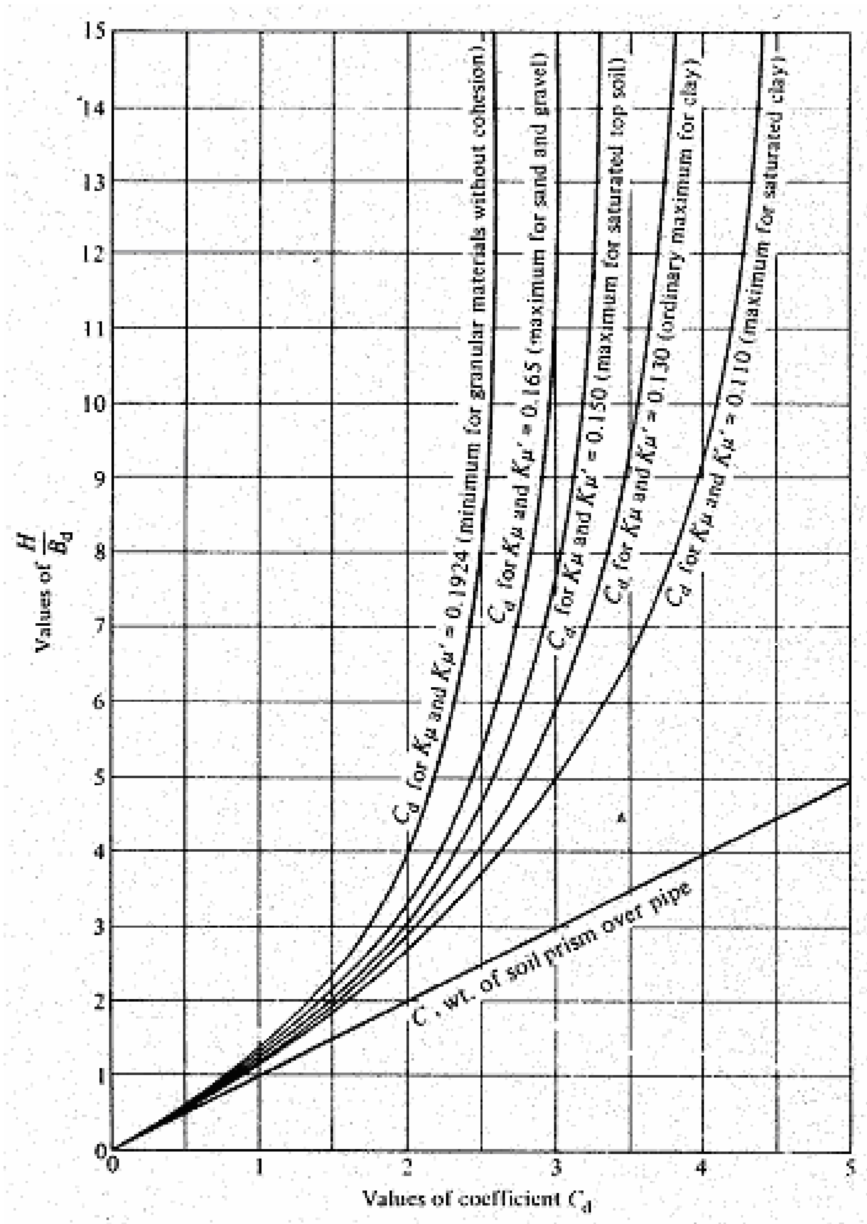


Figure 2.5 Diagrams for Coefficient C_d for Ditch Conduits

Table 2.1 Design Values of Settlement Ratio

Conditions	Settlement Ratio
Rigid culvert on foundation of rock or unyielding soil	+1.0
Rigid culvert on foundation of ordinary soil	+0.5 ~ +0.8
Rigid culvert on foundation of material that yields with respect to adjacent natural ground	~ +0.5
Flexible culvert with poorly compacted side fills	-0.4 ~ 0.0
Flexible culvert with well-compacted side fills	-0.2 ~ +0.2

Table 2.2 Values of C_c in Terms of H/B_c

Incomplete Projection Condition		Incomplete Ditch Condition	
$K\mu = 0.19$		$K\mu = 0.13$	
$r_{sd}p$	Equation	$r_{sd}p$	Equation
+0.1	$C_c = 1.23\ H/B_c - 0.02$	–0.1	$C_c = 0.82\ H/B_c + 0.05$
+0.3	$C_c = 1.39\ H/B_c - 0.05$	–0.3	$C_c = 0.69\ H/B_c + 0.11$
+0.5	$C_c = 1.50\ H/B_c - 0.07$	–0.5	$C_c = 0.61\ H/B_c + 0.20$
+0.7	$C_c = 1.59\ H/B_c - 0.09$	–0.7	$C_c = 0.55\ H/B_c + 0.25$
+1.0	$C_c = 1.69\ H/B_c - 0.12$	–1.0	$C_c = 0.47\ H/B_c + 0.40$
+2.0	$C_c = 1.93\ H/B_c - 0.17$		

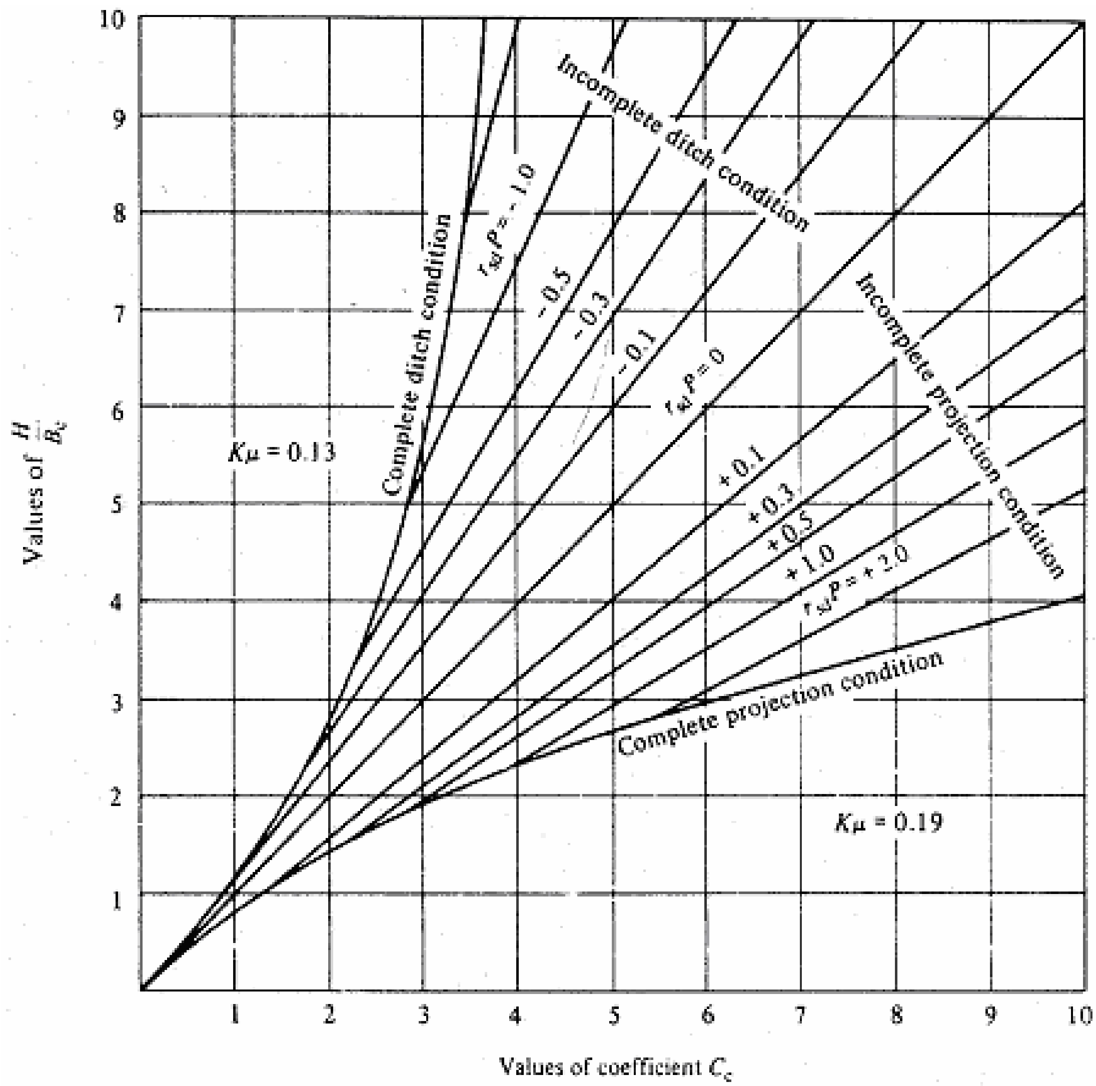
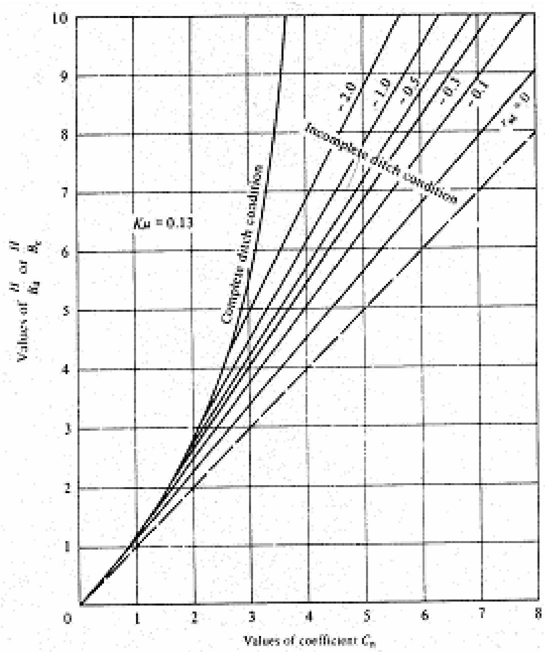
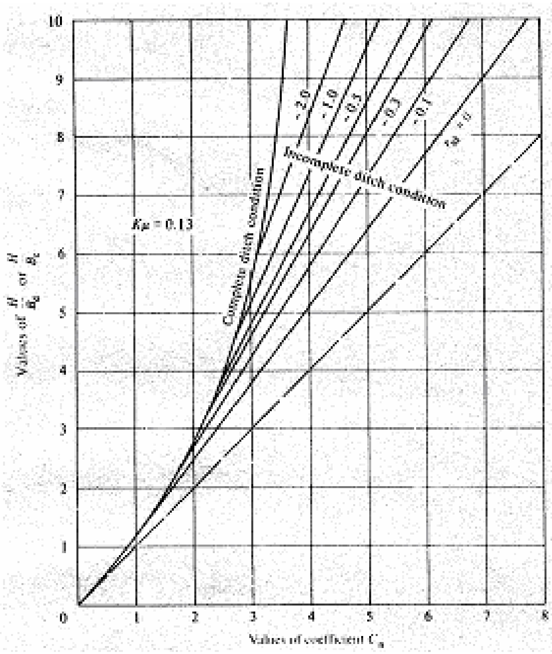


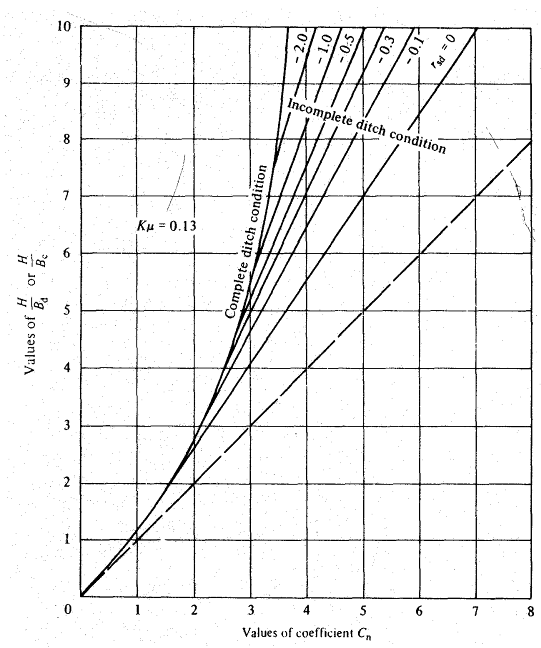
Figure 2.6 Diagrams for Coefficient C_c for Positive Projecting Conduits



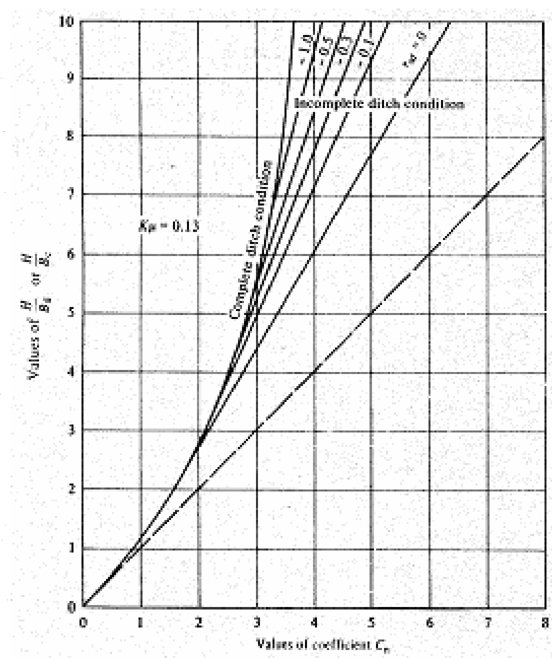
(a)



(b)



(c)



(d)

Figure 2.7 Diagrams for Coefficient C_n for Negative Projection Conduits and Imperfect Ditch Conditions (a) p′ = 0.5 (b) p′ = 1.0 (c) p′ = 1.5 (d) p′ = 2.0

2.3 Design Methods

2.3.1 Concrete Roadway Pipe

2.3.1.1 AASHTO Standard Installation Indirect Design

AASHTO Standard Installations were developed from a long-range research program of ACPA on the interaction of buried concrete pipe and soil in the early 1980s. Four AASHTO Standard Installations were produced as a result of numerous parametric studies using the finite element computer program SPIDA, Soil-Pipe Interaction Design and Analysis, for the direct design of buried concrete pipe.

The AASHTO Standard installations replacing the historical A, B, C, and D beddings of Marston and Spangler are presented in Figures 2.8 and 2.9. AASHTO Standard Installations differ significantly from Marston and Spangler's theory. Spangler's bedding factor research suffered from some severe limitations. First, for the embankment condition, Spangler developed a general equation for the bedding factor, which partially included the effects of lateral pressure. For the trench condition, Spangler established conservative fixed bedding factors, which neglected the effects of lateral pressure, for each of the three beddings [56]. Second, loads were considered as acting only at the top of the pipe. Third, axial thrust was not considered. The bedding width of test installations was also less than the width designated in his bedding configurations. Fourth, standard beddings were developed to fit assumed theories for soil support rather than ease of construction and method commonly used. Fifth, bedding materials and compaction levels were not adequately defined. AASHTO Standard Installations provide the basis for a more advanced design practice for pipe-soil installations based on the direct design of the pipe for its installed conditions. AASHTO Standard Installations also have several advantages over historical A, B, C, and D beddings because of the following considerations of practical construction [39]:

1) A flat foundation and bedding simplifies construction
2) Embedment soil cannot be compacted in the lower haunch area up to about 40 degrees from the invert.
3) AASHTO Standard Installations should permit the use of a range of embedment soils from the best quality granular soils that are easily compacted to various lesser quality soils that may be

readily available at a site. They should also include the option to use many native soils without compaction around the pipe for bedding, embedment and backfill.

4) Requirements for compaction with, or without, the use of high-quality embedment soils should be limited to those zones around the pipe where the embedment provides beneficial vertical or lateral support to the pipe.

The SPIDA studies were conducted for positive projection embankment conditions, which are the worst-case vertical load conditions for pipe, and which provide conservative results for other embankment and trench conditions. The parameter studies confirmed ideas postulated from past experience and proved the following concepts: [56]

1) Loosely placed, uncompacted bedding directly under the invert of the pipe significantly reduces stresses in the pipe.
2) Soil in those portions of the bedding and haunch areas directly under the pipe is difficult to compact.
3) The soil in the haunch area from the foundation to the pipe springline provides significant support to the pipe and reduces pipe stresses.
4) Compaction level of the soil directly above the haunch, from the pipe springline to the top of the pipe grade level, has negligible effect on pipe stresses. Compaction of the soil in this area is not necessary unless required for pavement structures.
5) Installation materials and compaction levels below the springline have a significant effect on pipe structural requirements.

The four AASHTO Standard Installations provide an optimum range of soil-pipe interaction characteristics. The Indirect Design Procedure for AASHTO Standard Installations is as follows:

1) Establish the pipe diameter and wall thickness.
2) Select the AASHTO Standard Installation to be used, Type I, II, III or IV.
3) Determine the vertical earth load and live load forces acting on the pipe.

4) Select the earth load and live load bedding factors for the selected installation (the live load bedding factor cannot be greater than the earth load bedding factor). These bedding factors are presented in the ACPA publication Design Data 40 [56] and in AASHTO Standard Specifications for Highway Bridges [6].
5) Apply factor of safety.
6) Divide the earth load and live load by their respective bedding factors and by the pipe diameter to determine the required D-Load strength. This D-Load is the service load condition.

Detailed explanations and several examples of indirect design procedure are presented in Chapter 6. Design Guides and Examples, which include the design equations and procedures developed in this study.

2.3.1.2 SIDD

Direct Design is designing specifically for the field condition anticipated loads and the resulting moments, both thrust and shear caused by such loadings. Design specifications for Direct Design are presented in Chapter 6. The Direct Design procedure is as follows:

1) Establish the pipe diameter and wall thickness.
2) Select the AASHTO Standard Installation to be used, Type I, II, III or IV.
3) Determine the vertical earth and live load forces acting on the pipe.
4) For the type of installation selected, determine the moments, thrusts and shears due to the applied loads. For each type of installation, design coefficients have been developed for the determination of the critical moments, thrusts and shears. Such coefficients are presented in the Concrete Pipe Technology Handbook, published by the American Concrete Pipe Association.
5) The structural design of the pipe is performed using established reinforced concrete design principles. Such design will include five performance modes:
    - Flexural
    - Diagonal and Radial tension
    - Concrete compression
    - Service load crack control

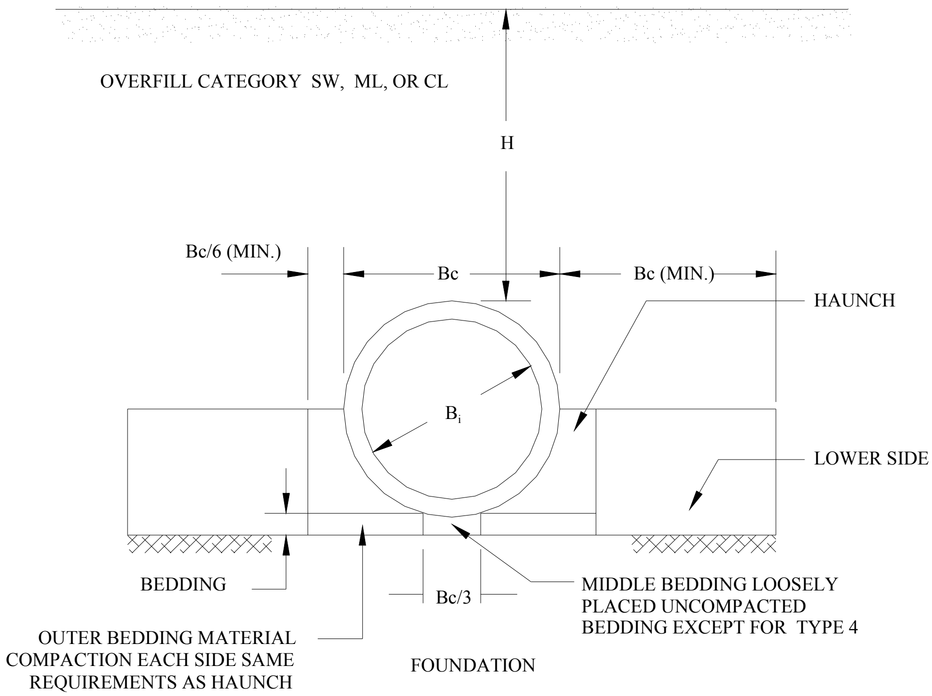


Figure 2.8 AASHTO Standard Embankment Installations

Table 2.3 AASHTO Standard Embankment Soils and Minimum Compaction Requirements

TYPE	BEDDING THICKNESS	HAUNCH AND OUTER BEDDING	LOWER SIDE
TREATED	Bc/24" (600MM) MIN., NOT LESS THAN 3" (75MM). IF ROCK FOR FOUNDATION USE Bc/12" (300) MIN., NOT LESS THAN 6" (150MM)	85% SW, 90% ML 0R 95% CL	85% SW, 90% ML 0R 95% CL
UNTREATED	NO BEDDING REQUIRED EXCEPT IF ROCK FOUNDATION USE Bc/12" (300MM) MIN., NOT LESS THAN 6" (150mm)	NO COMPACTION REQUIRED EXCEPT IF CL, USE 85% CL	NO COMPACTION REQUIRED EXCEPT IF CL, USE 85% CL

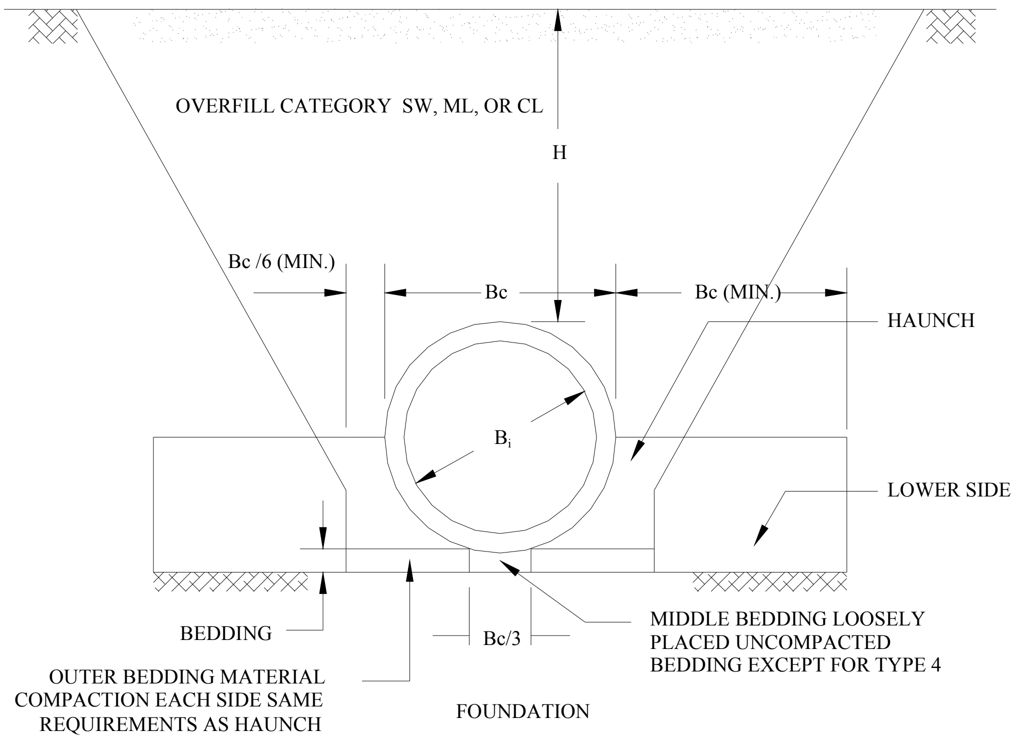


Figure 2.9 AASHTO Standard Trench Installations

Table 2.4 AASHTO Standard Trench Soils and Minimum Compaction Requirements

TYPE	BEDDING THICKNESS	HAUNCH AND OUTER BEDDING	LOWER SIDE
TREATED	Bc/24" (600MM) MIN., NOT LESS THAN 3" (75MM). IF ROCK FOR FOUNDATION USE Bc/12" (300) MIN., NOT LESS THAN 6" (150MM)	85% SW, 90% ML 0R 95% CL	85% SW, 90% ML 0R 95% CL OR NATURAL SOILS OF EQUAL FIRMNESS
UNTREATED	NO BEDDING REQUIRED EXCEPT IF ROCK FOUNDATION USE Bc/12" (300MM) MIN., NOT LESS THAN 6" (150mm)	NO COMPACTION REQUIRED EXCEPT IF CL, USE 85% CL	85% SW, 90% ML 0R 95% CL OR NATURAL SOILS OF EQUAL FIRMNESS

2.3.2 Box Culverts

2.3.2.1 AASHTO

The current AASHTO Standard Specifications for Highway Bridges [6] stipulate dead loads and wheel loads on the culverts through earth fills in Section 6, “Culverts,” and culvert design guidelines are provided in Section 16, “Soil-Reinforced Concrete Structure Interaction Systems.” Vertical and horizontal earth pressures on culverts may be computed by recognized or approximately documented analytical techniques based on the principles of soil mechanics and soil structure interaction, or design pressure can be calculated as being the results of an equivalent fluid weight as follows:

Culvert in trench, or culvert untrenched on yielding foundation

1) Rigid culverts, except reinforced concrete boxes
   - A. For vertical earth pressure 120 pcf
     For lateral earth pressure 30 pcf
   - B. For vertical earth pressure 120 pcf
     For lateral earth pressure 120 pcf
2) Reinforced concrete boxes
   - A. For vertical earth pressure 120 pcf
     For lateral earth pressure 30 pcf
   - B. For vertical earth pressure 120 pcf
     For lateral earth pressure 60 pcf
3) Flexible Culverts
   For vertical earth pressure 120 pcf
   For lateral earth pressure 120 pcf

Culvert untrenched on unyielding foundation

A special analysis is required.

The effects of soil-structure interactions can be taken into account based on the design earth cover, sidefill compaction, and bedding characteristics. These parameters may be determined by a soil-structure interaction analysis of the system. The loads given above may be used if they are multiplied by a soil-structure interaction factor, F_e, that accounts for the type and conditions of installation, so that the total earth load, W_e, on the reinforced concrete box section, either cast-in-place or precast, is:

$$W_e = F_e w B_c H \tag{2-4}$$

F_e may be determined by the Marston-Spangler theory of earth loads, as follows:

Embankment Installation

$$F_{el} = 1 + 0.20\frac{H}{B_c} \tag{2-5}$$

Trench Installation

$$F_{e2} = \frac{C_d B_d^2}{HB_c} \tag{2-6}$$

Where

w = unit weight of soil

H = backfill height;

B_c = out-to-out horizontal span of the conduit;

B_d = horizontal width of trench installation; and

C_d = load coefficient for trench installation.

F_{e1} need not be greater than 1.15 for installations with compacted fill at the sides of the box section, and need not be greater than 1.4 for installations with uncompacted fill at the sides of the box section. The load coefficient for trench installation, C_d, is included in Equation 2-6 as a variable. Although the current AASHTO equation is based on the Marston-Spangler theory, the load coefficient, C_d, based on the Marston theory, remains the same. Load coefficients for trench installation, C_d, provided in the graphical diagram for several soil types in AASHTO, are identical to those of the Marston-Spangler theory as described in the previous section and Figure 2.5. The maximum value of F_{e2} need not exceed F_{e1}.

2.3.2.2 Tadros et al

Tadros et al. [27, 63] proposed design equations for estimating earth pressure on positive projection box conduits. The design equations were developed based on numerical modeling performed with the finite element program CANDE-1980. Two subgrade soil conditions were modeled. Tadros et al's studies for bottom and side pressures, as well as top pressure, are remarkable.

The design equations for estimating earth pressure on positive projecting embankment installations are as follows:

1) For silty-clay soil

$$P_T = (0.984 + 0.0063H)(\gamma_s)(H) \tag{2-7}$$

$$P_S = (0.600)(\gamma_s)(H) \tag{2-8}$$

$$P_B = P_T + \frac{(57 + 26.3H_B)(2H_B)}{W_B} \tag{2-9}$$

2) For silty-sand soil

$$P_T = (0.970 + 0.0067H)(\gamma_s)(H) \tag{2-10}$$

$$P_S = (0.567)(\gamma_s)(H) \tag{2-11}$$

$$P_B = P_T + \frac{(114 + 16.2H_B)(2H_B)}{W_B} \tag{2-12}$$

Where,

P_T = pressure on the top slab, psf

P_S = pressure on the side wall, psf

P_B = pressure on the bottom slab, psf

H = fill height above the point considered, ft

γ_s = fill unit weight, pcf

H_B = overall height of box culvert, ft

W_B = overall width of box culvert, ft

2.3.2.3 Kim and Yoo

In Kim and Yoo's study [42], the pressure values on the top flange of box culverts were conservatively averaged and converted to effective densities, which are numerical values representing an equivalent hydro-static pressure. The value of the effective density is identical in nature to F_{e1} given by Equation 2-5. The predicted values on yielding foundations from both ABAQUS and ISBILD analyses lie between the values given for the compacted and the uncompacted side-fill by AASHTO [6]. It appears from this analysis that the value of the effective density is most sensitive to the foundation characteristics. In the case of an unyielding foundation, the value of the effective density from ABAQUS analysis showed somewhat higher value than that obtained from ISBILD.

Kim and Yoo [42] presented design equations for estimating earth pressures on positive projection box conduits, focusing on the fact that the value of the effective density is most sensitive to the foundation characteristics. The results of their analysis also indicated that the value of the effective density for an unyielding foundation would be much higher than that for a yielding foundation. Two proposed predictor equations are derived based on the data from ABAQUS and ISBILD analyses by means of a linear regression method.

$$D_E = 1.009H^{0.048} \quad \text{on Yielding Foundation} \qquad (2\text{-}13)$$

$$D_E = 1.016H^{0.068} \quad \text{on Unyielding Foundation} \qquad (2\text{-}14)$$

Where D_E and H stand for effective density value and filling height, respectively. Total earth load, W_e is determined by Equation 2-4.

The design equations given by AASHTO [6], Tadros et al [37, 63], and Kim and Yoo [42] have several shortcomings, which are as follows:

1) AASHTO and Kim and Yoo's design equations consider only top earth pressure.

2) The effects of foundation characteristics and sidefill treatment are not considered in Tadros et al's proposed equations.
3) There is no explanation for the unbalance between top and bottom. In other words, shear forces on the sidewall are not considered in the design except by Tadros et al.
4) The effects of interface conditions for shear forces on the sidewall are not considered in the design.

2.4 Imperfect Trench Installation

Modern design specifications have required buried conduits to be placed under increasing fill heights. The failure of buried conduits under these high fill situations can cause significant economic loss and environmental damage.

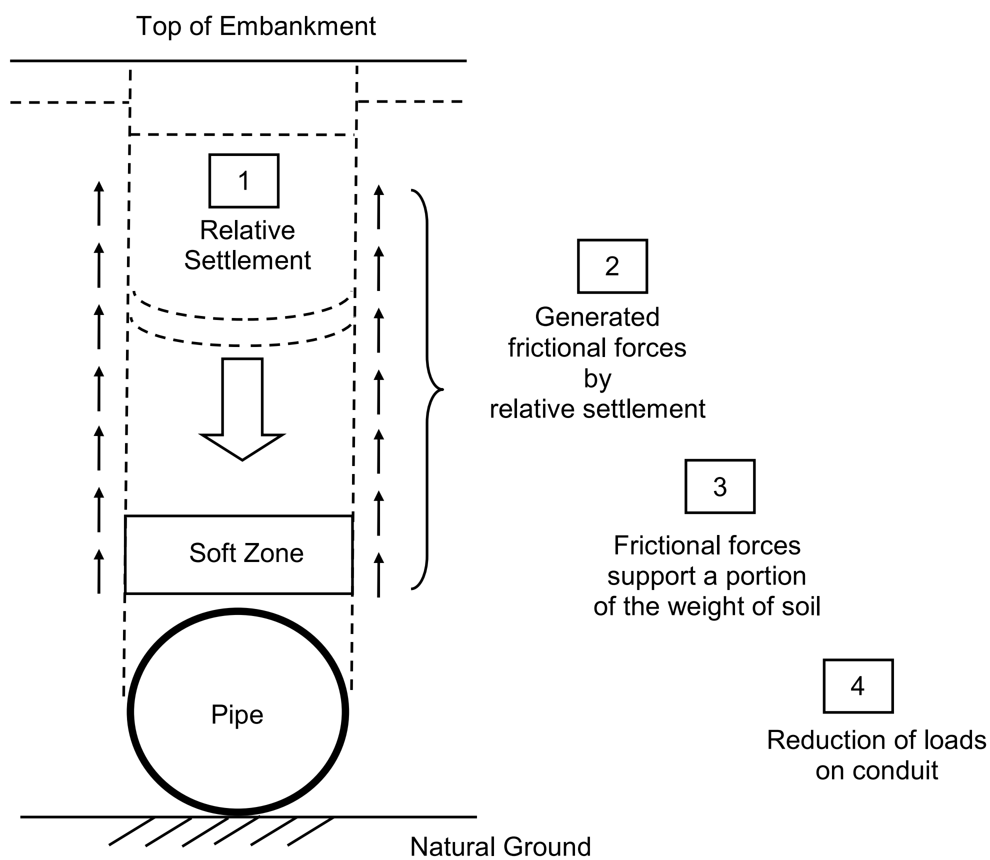


Figure 2.10 Mechanism of Imperfect Trench Installation

The induced trench installation is used to reduce earth pressures on buried conduits. Induced trench conduits are installed with a compressible material in the fill located directly above buried conduits. The compressible layers create frictional forces within the backfill that help support the weight of the soil overlying the conduit. The mechanism of imperfect trench installation is shown in Figure 2.10.

The imperfect trench method of pipe installation was developed by Marston [64]. The current design method developed by Spangler [65] is based predominantly on the work presented by Marston over 80 years ago. Sladen and Oswell [58] pointed out several shortcomings of Spangler's imperfect trench theory as follows:

1) The stiffness of the compressible layer is not considered. Earth pressures predicted from Spangler's design method are essentially independent of the stiffness of the compressible layer.
2) There are no specific guidelines for the optimum geometry of the soft zone.
3) The effects of horizontal stresses and shear forces on the sidewall of the conduit are not considered.
4) The mechanical properties of the backfill are not considered in the theory.

Imperfect trench designs based on the Marston-Spangler Theory have generally been successful. However, experimental studies have shown the predicted earth pressure to be highly conservative [58, 59, and 60]. This is caused in part by the conservative parameters used in the development of the design charts [57].

Vaslestad [60] proposed design equations for determining earth loads on induced trench installations. Earth loads on the conduit are determined by applying an arching factor the overburden pressure. This arching factor is based on the friction number, S_v, used by Janbu to determine friction on piles [61]. Vaslestad's equation for estimating earth pressures on an induced trench culvert is given as:

$$\sigma_v = N_A \gamma H \tag{2-15}$$

$$N_A = \frac{1 - e^{-2S_V \frac{H}{B}}}{2S_V \frac{H}{B}} \tag{2-16}$$

$S_V = |\gamma| \tan \rho K_A$ (2-17)

Where,

N_A = Arching factor

S_V = Janbu's friction number

B = Width of conduit

r = Roughness ratio = $\dfrac{\tan \delta}{\tan \rho} \leq 1$

$\tan \rho$ = mobilized soil friction

f = degree of soil mobilization

$\tan \phi$ =soil friction

$K_A = \dfrac{1}{\left[\sqrt{1+\tan^2 \rho} + \tan \rho \sqrt{1-|r|}\right]^2}$, Active earth pressure coefficient

Vaslestad reported that the design method shows good agreement between the earth pressure measured on a full scale induced trench installation and the results from the finite element analysis program CANDE [61]. However, Vaslestad also failed to consider the effect of shear force on the sidefill, foundation characteristics, and sidefill treatment.

Although the installation procedure for imperfect trench installation induces considerable reductions in earth pressure, the imperfect trench installation method has not frequently been utilized as it is generally regarded with some skepticism in terms of its long-term behavior. The American Concrete Pipe Association has omitted the imperfect trench design method from the 2001 edition of the Concrete Pipe Handbook to reflect this concern. Full-scale tests were conducted to examine the load reduction effect under imperfect trench installation over a period of 3 years [22]. The results showed there was no increase in vertical earth pressure.

The primary function of the imperfect trench method is to decrease the top earth pressure and increase the horizontal pressure. The supporting strength is enhanced by the lateral pressure that acts on projecting embankment conduits. Horizontal pressure generally does not exceed top earth pressure. However, the results of this study for box culverts show the bottom pressure does increase largely due to the increase of the shear force on the side wall under imperfect trench installation, which implies that the bottom pressure generally exceeds the top pressure. This study proposes an effective design method to prevent the non-uniform of earth pressures between the top and bottom slabs of box culverts due to imperfect trench installations, which are presented in Chapter 4.

2.5 Soil Models

2.5.1 Duncan Soil Model and Parameters

Kondner [14] has shown that the nonlinear stress-strain curves for both clay and sand may be approximated by a hyperbola with a high degree of accuracy. This hyperbola can be represented by an equation of the form:

$$(\sigma_1 - \sigma_3) = \frac{\varepsilon}{a + b\varepsilon} \tag{2-18}$$

Where σ_1, σ_3 = the maximum and minimum principal stresses, respectively; ε = the axial strain; and a, b = constants whose values may be determined experimentally. Both of these constants a and b have readily discernible physical meanings. As shown in Figures 2.11 and 2.12, a is the reciprocal of the initial tangent modulus, E_i and b is the reciprocal of the asymptotic value of the stress difference which the stress-strain curve approaches at infinite strain $(\sigma_1-\sigma_3)_{ult}$. The values of the coefficients a and b may be determined readily if the stress-strain data are plotted on transformed axes, as shown in Figure 2.12. When Equation 2-18 is rewritten in the following form:

$$\frac{\varepsilon}{(\sigma_1 - \sigma_3)} = a + b\varepsilon \tag{2-19}$$

Here, a and b are the intercept and the slope of the resulting straight line, respectively. By plotting stress-strain data in the form shown in Figure 2.12, it is straightforward to determine the values of the parameters a and b corresponding to the best fit between a hyperbola and the test data. It is commonly found that the asymptotic value of $(\sigma_1-\sigma_3)$ is larger than the compressive strength of the soil by a small amount, because the hyperbola remains below the asymptotic at all finite values of strain. The asymptotic value may be related to the compressive strength, however, by means of a factor R_f:

$$(\sigma_1 - \sigma_3)_f = R_f (\sigma_1 - \sigma_3)_{ult} \tag{2-20}$$

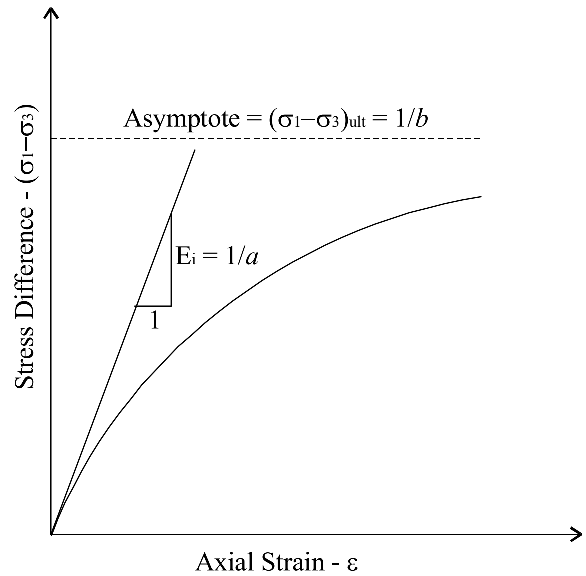


Figure 2.11 Hyperbolic Stress-Strain Curve

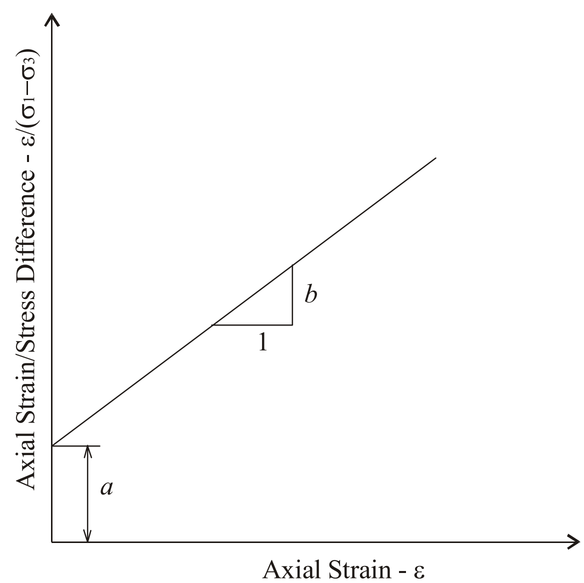


Figure 2.12 Transformed Hyperbolic Stress-Strain Curve

where $(\sigma_1\text{-}\sigma_3)_f$ = the compressive strength, or stress difference at failure; $(\sigma_1\text{-}\sigma_3)_{ult}$ = the asymptotic value of stress difference; and R_f = the failure ratio, which always has a value less than unity. For a number of different soils, the value of R_f has been found to be between 0.75 and 1.00, and is essentially independent of confining pressure. By expressing the parameters a and b in terms of the initial tangent modulus value and the compressive strength, Equation 2-18 may be rewritten as

$$(\sigma_1 - \sigma_3) = \frac{\varepsilon}{\left[\dfrac{1}{E_i} + \dfrac{\varepsilon R_f}{(\sigma_1 - \sigma_3)_f}\right]} \tag{2-21}$$

This hyperbolic representation of stress-strain curves has been found to be fairly useful in representing the nonlinearity of soil stress-strain behavior. Except for the case of unconsolidated-undrained tests on saturated soils, both the tangent modulus value and the compressive strength of soils have been found to vary with the confining pressure employed in the tests. Experimental studies by Janbu [15] have shown that the relationship between tangent modulus and confining pressure may be expressed as

$$E_i = K \cdot p_a \left(\frac{\sigma_3}{p_a}\right)^n \tag{2-22}$$

where E_i = the initial tangent modulus; σ_3 = the minimum principal stress; p_a = atmospheric pressure expressed in the same pressure units as E_i and σ_3; K = a modulus number; and n = the exponent determining the rate of variation of E_i with σ_3.

If it is assumed that failure will occur with no change in the value of σ_3, the relationship between compressive strength and confining pressure may be expressed conveniently in terms of the Mohr-Coulomb failure criterion as

$$(\sigma_1 - \sigma_3)_f = \frac{2c\cos\phi + 2\sigma_3 \sin\phi}{1 - \sin\phi} \tag{2-23}$$

where c, ϕ = the Mohr-Coulomb strength parameters. Equations 2-22 and 2-23, in combination with Equation 2-4, provide a means of relating stress to strain and confining pressure using the five parameters K, n, c, ϕ, and R_f. Nonlinear, stress-dependent stress-strain behavior may be approximated in finite element analyses by assigning different modulus values to each of the elements into which the soil is subdivided for purposes of analysis. The modulus value assigned to each element is selected on the basis of the stresses or strains in each element. Because the modulus values depend on the stresses, and the stresses in turn depend on the modulus values, it is necessary to make repeated analyses to ensure that the modulus values and stress conditions correspond for each element in the system.

The stress-strain relationship expressed by Equation 2-21 may be employed in incremental stress analyses because it is possible to determine the value of the tangent modulus corresponding to any point on the stress-strain curve. If the value of the minimum principal stress is constant, the tangent modulus, E_t, may be expressed as:

$$E_t = \frac{\partial(\sigma_1 - \sigma_3)}{\partial \varepsilon} \tag{2-24}$$

If Equation 2-24 is then differentiated, and the strain, ε, derived from Equation 2-21, the initial tangent modulus, E_i, in Equation 2-21 and the compressive strength, $(\sigma_1-\sigma_3)_f$, in Equation 2-21 are substituted into the result of the differentiation, the following expression is obtained for the tangent modulus:

$$E_t = \left[1 - \frac{R_f\left(1-\sin\phi\right)\left(\sigma_1-\sigma_3\right)}{2c\cos\phi + 2\sigma_3\sin\phi}\right]^2 Kp_a\left(\frac{\sigma_3}{p_a}\right)^n \tag{2-25}$$

For the hyperbolic stress-strain relationships, the same value for the unloading-reloading modulus, E_{ur}, is used for both unloading and reloading. The value of E_{ur} is related to the confining pressure by an equation of the same form as Equation 2-22:

$$E_{ur} = K_{ur}\,p_a\left(\frac{\sigma_3}{p_a}\right)^n \tag{2-26}$$

where K_{ur} is the unloading-reloading modulus number. The value of K_{ur} is always larger than the value of K (for primary loading). K_{ur} may be 20% greater than K for stiff soils such as dense sands. For soft soils such as loose sands, K_{ur} may be three times as large as K. The value of the exponent n is always very similar for primary loading and unloading, and in the hyperbolic relationships it is assumed to be the same. The value of the tangent Poisson's ratio may be determined by analyzing the volume changes that occur during a triaxial test. For this purpose, it is convenient to calculate the radial strains during the test using the equation

$$\varepsilon_r = \frac{1}{2}(\varepsilon_v - \varepsilon_a) \tag{2-27}$$

where ε_v and ε_a are the volumetric and axial strains. Taking compressive strains as positive, the value of ε_a is positive and the value of ε_r is negative. The value of ε_v may be either positive or negative. If the variation of ε_a versus ε_r is plotted, as shown in Figure 2.13, the resulting curve can usually be reasonably accurately represented by a hyperbolic equation of the form:

$$\varepsilon_a = \frac{-\varepsilon_r}{\nu_i - d\varepsilon_r} \tag{2-28}$$

As shown in Figures 2.13 and 2.14, this equation may be transformed as follows:

$$-\frac{\varepsilon_r}{\varepsilon_a} = \nu_i - d\varepsilon_r \tag{2-29}$$

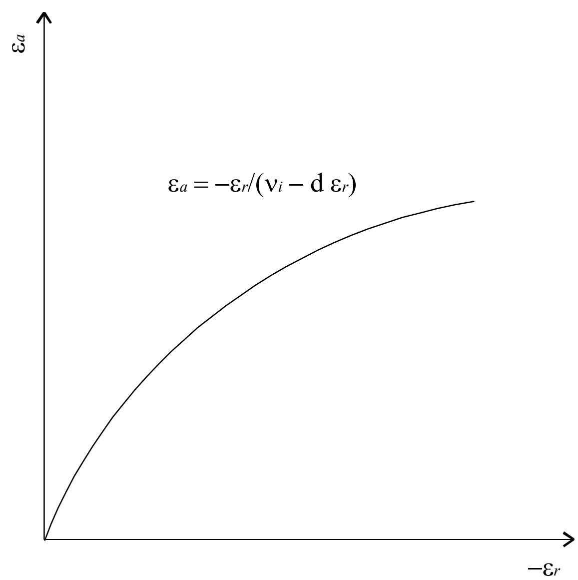


Figure 2.13 Hyperbolic Axial Strain-Radial Strain Curve

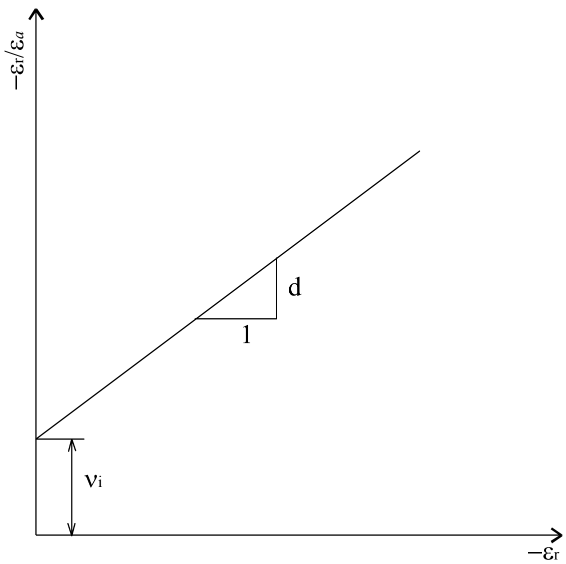


Figure 2.14 Transformed Hyperbolic Axial Strain-Radial Strain Curve

In Equation 2-29, ν_i is the initial Poisson's ratio at zero strain and d is a parameter representing the change in the value of Poisson's ratio with radial strain. For saturated soils under undrained conditions, there is no volume change and ν_i is equal to one half for any value of confining pressure. This variation of ν_i with respect to σ_3 may be expressed by the equation

$$\nu_i = G - F \log_{10}\left(\frac{\sigma_3}{p_a}\right) \tag{2-30}$$

where G is the value of ν_i at a confining pressure of one atmosphere, and F is the reduction in ν_i for a ten-fold increase in σ_3. After differentiating Equation 2-28 with respect to ε_r, substituting Equation 2-30, and eliminating the strain using Equations 2-18 to 2-21, the tangent value of Poisson's ratio may be expressed in terms of the stresses as follows:

$$\nu_t = \frac{G - F\,log\left(\dfrac{\sigma_3}{p_a}\right)}{\left\{1 - Kp_a\left(\dfrac{\sigma_3}{p_a}\right)^n\left[\dfrac{d\left(\sigma_1-\sigma_3\right)\left(1-sin\,\phi\right)}{2c\,cos\,\phi + 2\sigma_3\,sin\,\phi}\right]\right\}^2} \tag{2-31}$$

where σ_1, σ_3 = maximum and minimum principal stresses, respectively; K = modulus number; n = modulus exponent; c = cohesion intercept; ϕ = friction angle; G, F, d = Poisson's ratio parameters; and p_a

= atmospheric pressure. There are nine parameters involved in the hyperbolic stress-strain and volume change relationships, and the roles of these parameters are summarized in Table 2.5.

Nonlinear volume change can also be accounted for by employing the constant bulk modulus instead of Poisson's ratio parameters. The assumption that the bulk modulus of the soil is independent of stress level (σ_1-σ_3) and that it varies with confining pressure provides reasonable approximation to the shape of the volume change curves. According to the theory of elasticity, the value of the bulk modulus is defined by

$$B = \frac{\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3}{3\Delta\varepsilon_v} \tag{2-32}$$

Table 2.5 Summary of the Hyperbolic Parameters

Parameter	Name	Function
K, K_{ur}	Modulus number	Relate E_i and E_{ur} to σ_3
N	Modulus exponent	
C	Cohesion intercept	Relate (σ_1-σ_3) to σ_3
ϕ	Friction angle	
R_f	Failure ratio	Relates $(\sigma_1-\sigma_3)_{ult}$ to $(\sigma_1-\sigma_3)_f$
G	Poisson's ratio parameter	Value of ν_i at $\sigma_3=p_a$
F	Poisson's ratio parameter	Decrease in ν_i for ten-fold increase in σ_3
D	Poisson's ratio parameter	Rate of increase of ν_t with strain

where B is the bulk modulus, $\Delta\sigma_1$, $\Delta\sigma_2$, and $\Delta\sigma_3$ are the changes in the values of the principal stress, and $\Delta\varepsilon_v$ is the corresponding change in volumetric strain. For a conventional triaxial test, in which the deviator stress (σ_1-σ_3) increases while the confining pressure is held constant, Equation 2-32 may be expressed as:

$$B = \frac{(\sigma_1 - \sigma_3)}{3\varepsilon_v} \tag{2-33}$$

The value of the bulk modulus for a conventional triaxial compression test may be calculated using the value of (σ_1-σ_3) corresponding to any point on the stress-strain curve. When values of B are calculated from tests on the same soil specimen at various confining pressures, the bulk modulus will usually be found to increase with increasing confining pressure. The variation of B with confining pressure can be approximated by an equation of the form:

$$B = K_b p_a \left(\frac{\sigma_3}{p_a}\right)^m \tag{2-34}$$

where K_b is the bulk modulus number and m is the bulk modulus exponent, both of which are dimensionless, and p_a is atmospheric pressure. Experimental studies of this soil model, sometimes called the modified Duncan model, for most soils, has resulted in values of m varying between 0.0 and 1.0. If a bulk modulus is known, the tangent Poisson's ratio can be determined from the basic theory of elasticity by the following equation:

$$\nu_t = \frac{1}{2} - \frac{E_t}{6B} \tag{2-35}$$

Although the hyperbolic relationship outlined previously has proven to be quite useful for a wide variety of practical problems, it has some significant limitations [33]: 1) Being based on the generalized Hooke's law, the relationships are most suitable for analysis of stresses and movements prior to failure. The relationships are capable of accurately predicting nonlinear relationships between loads and movements, and it is possible to continue the analyses up to the stage where there is local failure in some elements. However, once a stage is reached where the behavior of the soil mass is controlled to a large extent by properties assigned to elements which have already failed, the results will no longer be reliable, and they may be unrealistic in terms of the behavior of real soils at and after failure. 2) The hyperbolic relationships do not include volume changes due to changes in shear stress, or shear dilatancy. They may, therefore, be limited in the accuracy with which they can be used to predict deformations in dilatant soils, such as dense sands under low confining pressures. The values of the tangent Poisson's ratio calculated using Equation 2-24 may exceed 0.5 for some combinations of parameter values and stress values, so it needs to be specified to be less than 0.5 in the computer program. 3) The parameters are not fundamental soil properties, but only values of empirical coefficients that represent the behavior of the soil under a limited range of conditions. The values of the parameters depend on the density of the soil, its water content, the range of pressures used in testing, and the drain conditions. In order that the parameters will be representative of the real behavior of the soil under field conditions, the laboratory test conditions must correspond to the field conditions with regard to these factors.

2.5.2 Selig Bulk Modulus and Parameters

Both the Duncan and Selig parameters were deriving using the same Young's modulus obtained from constant confining pressure triaxial tests. However, Selig's model incorporated an alternative method for obtaining bulk modulus based on a hydrostatic compression test [28]. In this test, the soil specimen is compressed under an increasing confining pressure applied equally in all directions. According to Equation 2-32, tangent bulk modulus B is the slope of the hydrostatic stress-strain curve. Selig observed that the curve relating σ_m and ε_{vol} was found to be reasonably represented by the hyperbolic equation

$$\sigma_m = \frac{B_i \varepsilon_{vol}}{1-(\varepsilon_{vol} / \varepsilon_u)} \quad (2\text{-}36)$$

where B_i = initial tangent bulk modulus, and ε_u =ultimate volumetric strain at large stress. The tangent bulk modulus B is determined by differentiating Equation 2-36 and substituting for ε_{vol} from Equation 2-36. The result is Selig's bulk modulus expression.

$$B = B \ \ (1+\sigma_m /(B_i \varepsilon_u))^2 \quad (2\text{-}37)$$

To determine the parameters B_i and ε_u ,the test results from the left side of Figure 2.15 are plotted in the linearized hyperbolic form of Figure 2.16. Equation 2-36 will be a straight line in Figure 2.16. Once B_i and ε_u are known, the test results can be represented by Equation 2-37.

Resent studies [66] have shown that the hyperbolic formulation for bulk modulus, Equation 2-37, better represents soil behavior in a hydrostatic compression test than Duncan's power formulation, Equation 2-26, thus favoring the use of Selig's model.

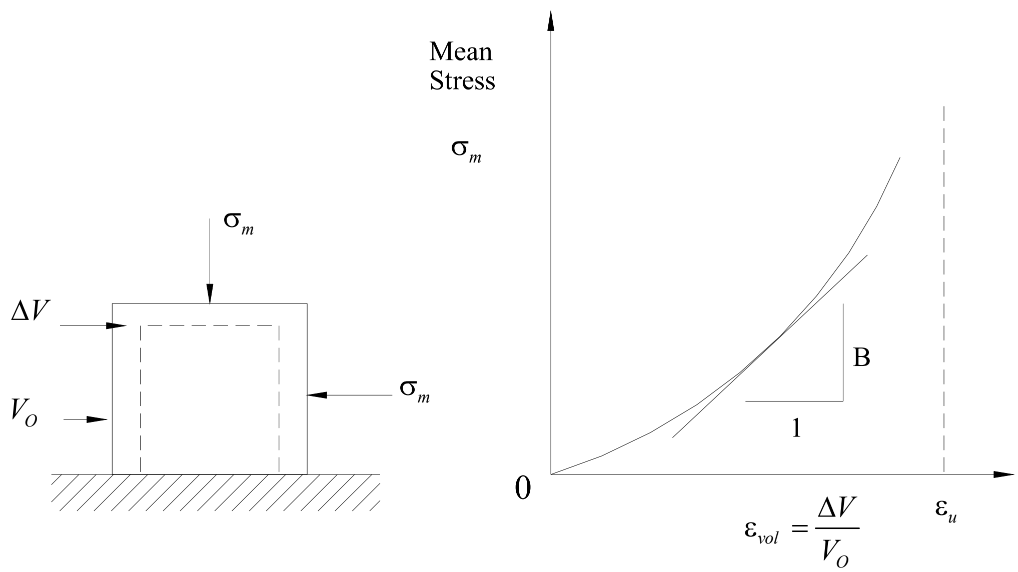


Figure 2.15 Hydrostatic Compression Test

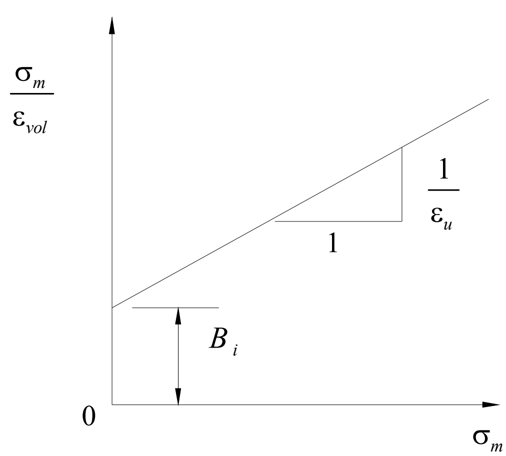


Figure 2.16 Linear Transformation of Hyperbola for Bulk Modulus

CHAPTER 3

Soil-Structure Modeling

3.1 General

This chapter describes the methodology used in the numerical analyses presented in Chapter 4. The structural effects of earth and other loads on the soil-structure system were analyzed using a Finite Element Method (FEM). Katona et al [51] pioneered the application of the finite element method for the solution of buried pipe problems. Their FHWA-sponsored project produced the well-known public domain computer program CANDE (Culvert ANalysis and DEsign). Duncan et al [33], Leonards et al [18], Anderson et al [52], and Sharp et al [53] also made contributions to the development of the finite element method for buried structures problems.

A finite element analysis of a soil-structure interaction system is different from a finite element analysis of a simple linearly elastic continuum in several ways. First, soil has a nonlinear stress-strain relationship. As a result, large load increments can lead to significant errors in evaluating stress and strain within a soil mass. Nonlinear incremental analysis procedures in Finite Element Program are used to simulate nonlinear soil properties and incremental construction sequences. Nonlinear incremental analysis procedures employ the hyperbolic stress-strain relationship, along with incremental analysis procedures based on plane strain linear elastic elements.

Second, different element types must be used to represent the pipe and the soil. It may be necessary to allow movement between the soil and the walls of the pipe, requiring the use of an interface element. Many investigators have either employed or commented on the necessity of modeling interface behavior in the finite element analysis of buried conduit installations. However, Kim and Yoo [42] and McVay [16] showed that the effect of interface behavior is insignificant for the interaction of soil-structure systems. The compaction process is also not considered in this study, as McVay [16] found that reasonable results were obtained without including numerical representations of such effects.

In this study, a general finite element programs, ABQUS version 6.3.1, NASTRAN 2004, the most commonly used program for analysis of buried pipe, CANDE-89, and SPIDA version 3C for the design of buried concrete pipe, were used to simulate and analyze a finite element model of the soil-structure system.

3.2 Soil-Structure Modeling

3.2.1 Modeling Techniques

The finite element model in this study was developed for installations that have a uniform application of loads and support along the longitudinal axis of the conduits. Half of the conduit needs to be represented in the model, as the geometry, properties, and loading conditions are symmetrical. Symmetry was thus used to reduce the size of the problem. The boundary conditions along the line of symmetry must be properly established in order to model the full system behavior.

3.2.1.1 Geometry

Results from parametric study have shown that in the one half models, the width of soil layer need not extend further than 2.5 times the horizontal span of the conduits. When the width of the soil layers increases to 5 times the horizontal span, the vertical earth load on the conduit shows an increment of only 0.5% over the results obtained using 2.5 times the horizontal span of the conduit. The distance from the bottom of the conduit to the bottom of the foundation is at least one and half times the vertical height of the conduit. The fill height from the top of the conduit is 3 times the vertical height of the conduit. CANDE-89 and SPIDA use each 1.5 time and 3 times of vertical height of conduit for fill height.

3.2.1.2 Types and Elastic Properties of soil and conduit

The conduits were modeled using a beam element. The elastic properties of the conduits used in the analysis are given in Table 3.1. A plane strain linear elastic element was used for the soil element. The elastic properties of the soil used in the finite element analysis were defined by two stiffness parameters, the tangent Young's modulus of elasticity, E_t and bulk modulus, B. These two parameters can be calculated from the Duncan soil model and Selig's bulk modulus as discussed in Section 2.6. The types of soil properties that are needed in Duncan and Selig's formulations are given in the Appendices.

Table 3.1 Elastic Properties of Conduits

Modulus of Elasticity, E (psi)	Poisson's Ratio, ν	Unit Weight, γ (pcf)
3,600,000	0.2	145

3.2.1.3 Loads and Boundary Conditions

Lateral boundaries should be restrained with vertical rollers. Bottom boundaries may be restrained by pinning. It is important to make sure that two nodes of the cut plane of the beam are fixed for rotation, for symmetry must not be violated.

For fill heights deeper than 3 times the vertical height of the conduit, surcharge loads may be used to represent the remaining soil weight.

3.2.1.4 Nonlinear Incremental Analysis

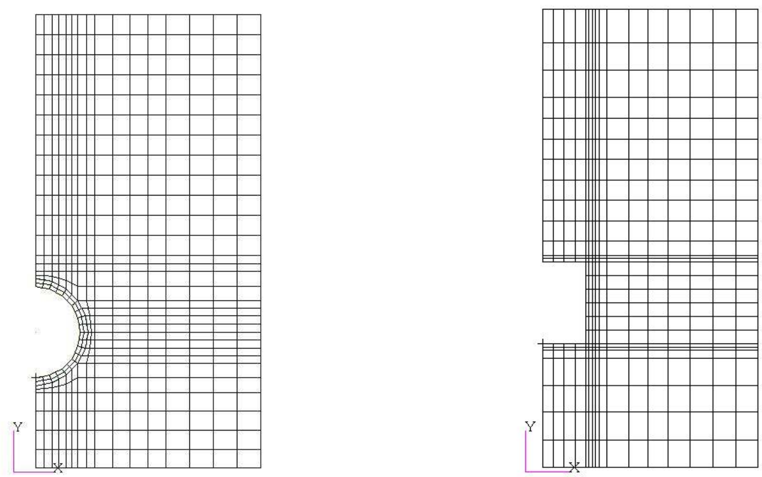
One way of performing approximate nonlinear analyses of soil behavior is to perform incremental analyses, changing the soil property values for each stage of the analysis. The incremental nature of the formulation allows the structural system to accumulate a response history using an analytical technique called incremental construction, so that the history of the structural responses is obtained during the construction process [28].

Using constant material properties for the ABAQUS and NASTRAN input, tangent Young's modulus and tangential Poisson's ratio can be evaluated for each construction layer using Equations 2-21, 2-31 and 2-33 by assuming that the soil layers are in principal states without any live loads. Thus, for constant material property values of the ith incremental soil layer's element group, the following principal stresses are assumed:

$$\sigma_1^{(i)} = \gamma_i (H_i / 2) + \sum_{j=j+1}^{n} \gamma_j H_j \tag{3-1}$$

$$\sigma_3^{(i)} = K \sigma_1^{(i)} \tag{3-2}$$

where Hi, and γi are the depth and density of the ith soil layer, respectively (numbering from the bottom to the top of the backfill or original ground). These values are substituted for the principal stresses in Equation 2-21 for the evaluation of tangent Young's modulus and tangential Poisson's ratio. The coefficient of lateral pressure, K, is 0.5. For the soil properties in SPIDA and CANDE-89, Duncan and Selig’s parameters from the soil properties table in the Appendices are entered in the input file directly. Tangent Young's modulus and tangential Poisson's ratio are then evaluated automatically using Duncan and Selig’s formulations. A typical model and incremental sequence of soil-structure system used in this study are shown in Figures 3.1 and 3.2.



(a) Concrete Pipe   (b) Box Culverts

Figure 3.1 Typical Soil-Structure Finite Element Model

3.2.2 Proposed Installations and Imperfect Trench Installation

Proposed Installations are used instead of four types of AASHTO Standard Installation in this study. Proposed Installations consist of a combination of TREATED and UNTREATED installations, which conform to Type 3 and Type 4 AASHTO Standard Installations, respectively. The validity of Proposed Installations will be verified later in Chapter 4. Soil properties chosen to represent the foundation, bedding, and side fill can have an extremely significant effect on the conduit responses obtained. Each set of soil properties for the foundation, bedding, and sidefill in Proposed Installations can be determined from Duncan and Selig’s formulations and the soil parameters given in the Appendix E.

Properties of the soft materials used in imperfect trench installation are given in Appendix E. Poisson’s ratio was assumed to be 0.1 in this study. The EPS experienced negative Poisson’s ratio during compression tests [54]. The soil properties used in each of the soil zones of the TREATED and UNTREATED installations are given in Tables 3.2 and 3.3.

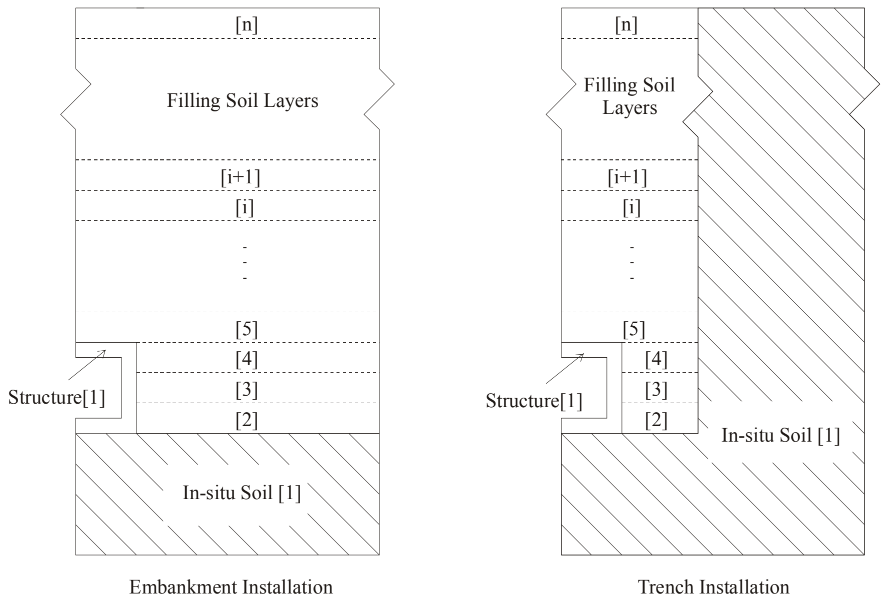


Figure 3.2 Incremental Sequences for Embankment and Trench Installations

Table 3.2 Soil Property No. used in the Analysis of Concrete Roadway Pipe [Appendix E]

	TREATED	UNTREATED
Foundation	11	11 or 14
Middle Bedding	8 or 9	8 or 9
Sidefill	25	26

Table 3.3 Soil Properties No. used in the Analysis of Box Culverts [Appendix E]

	TREATED	UNTREATED
Foundation (Yielding)	11	11
Foundation (Unyielding)	20	20
Leveling coarse	25	25
Sidefill	23	26

3.3 Verification

3.3.1 Concrete Roadway Pipe

In order to assess the validity of the soil modeling technique adopted in this study, the D-loads given in the Maximum Fill Height tables from ACPA were reanalyzed using ABAQUS, NASTRAN, and SPIDA. As can be seen from Table 3.4, the D-loads obtained using ABAQUS, NASTRAN, and SPIDA with Duncan and Selig's parameters are in good agreement with those of ACPA, within 7% in all cases, and with a less than 3% difference for the Type 3 and Type 4 installations used in this study. Therefore, it was concluded that the finite element analysis results obtained using ABAQUS and the soil modeling technique adopted are valid.

Table 3.4 Verification of Soil Modeling Techniques

	B_i (in), H(ft)	D-load (lb/ft/ft) (%) Difference with ACPA			
		ACPA	SPIDA	NASTRAN	ABAQUS
Type 1	72, 20	1050	1120 (6.6)	1111 (5.8)	1112 (5.9)
Type 2	72, 40	2925	2944 (0.6)	3122 (6.7)	3123 (6.7)
Type 3	72, 32	3000	2980 (-1.3)	2924 (-2.5)	2936 (-2.1)
Type 4	72, 22	2800	2881 (2.8)	2821 (0.7)	2807 (0.3)

Note: B_i = pipe inside diameter, H = fill height

3.3.2 Box Culverts

The pressure distributions for the top and bottom pressures on box culverts obtained using ABAQUS and NASTRAN with Duncan and Selig's parameters show good agreement with those of

CANDE-89 in both embankment and imperfect trench installations. The soil pressure directly above the sidewall is substantially higher than the soil pressure at the center of the top slab, where the largest relative vertical deflection is expected to occur. It appears that the inclusion of reinforcing steel in the calculation of the slab stiffness has a negligible effect on the soil pressure distribution, as expected from ACI [35] Article 8.6, "Stiffness."

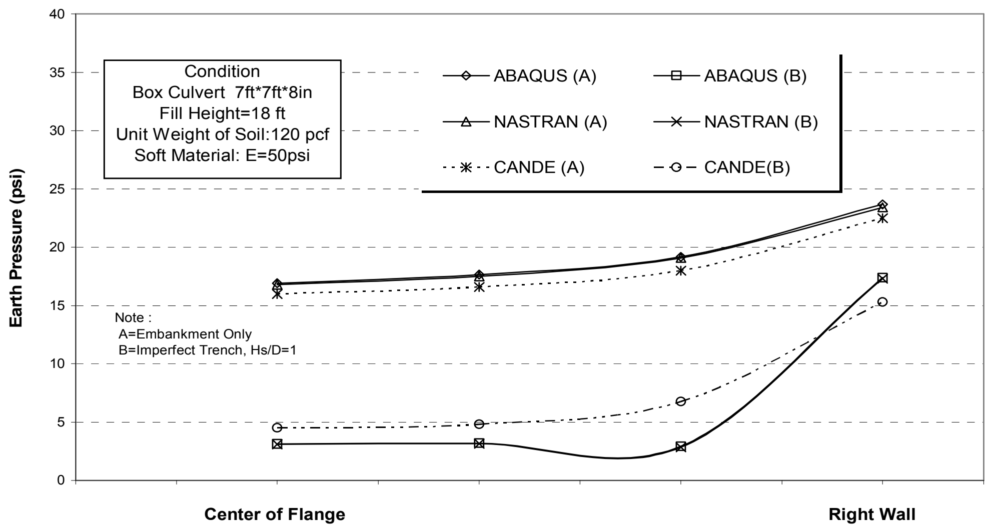


Figure 3.3 Comparison of Top Pressure Distributions (ABAQUS, NASTRAN, and CANDE-89)

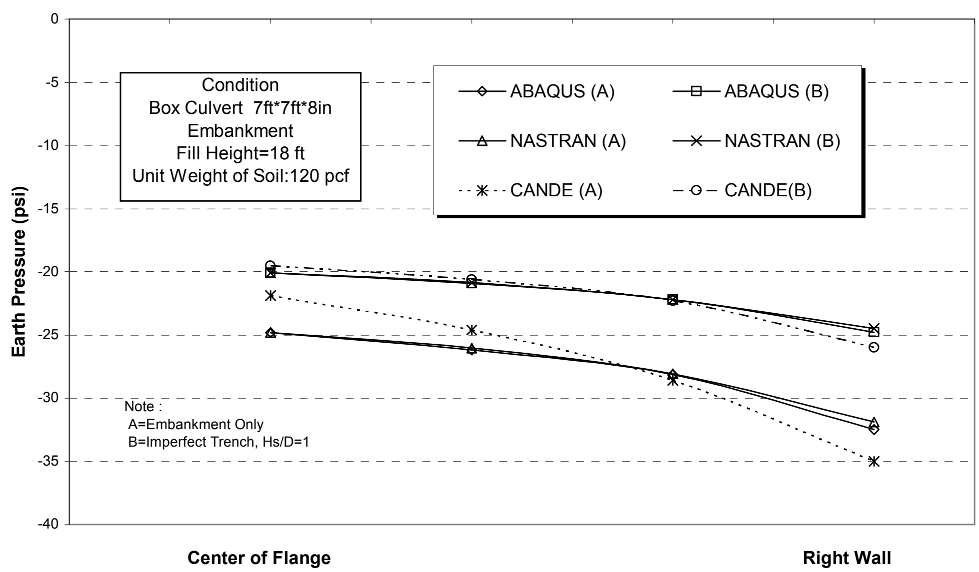


Figure 3.4 Comparison of Bottom Pressure Distributions (ABAQUS, NASTRAN, and CANDE-89)

CHAPTER 4
Numerical Analysis

4.1 General

This chapter gives the results of the numerical analysis performed in this study. The numerical analyses focused on the interaction of bedding and fill heights for deeply buried rigid conduits under embankment and imperfect trench installations. The earth load transmitted to the conduit is significantly affected by the installation method and bedding type.

Bedding under a conduit is generally compacted in order to control the conduit grade by minimizing the settlement after construction. The results of computer modeling indicate that using an uncompacted middle bedding reduces both the load on the conduit and the invert bending moments [55]. However, the outer bedding should still be compacted to provide support to the haunch area of the conduit and to provide an alternate vertical load path around the bottom of the conduit.

The haunch area of the concrete pipe provides a significant portion of the support of the earth load. Both field tests and computer models show that the bending moments are greater in an untreated haunch. A significant void in the haunch area can lead to longer term soil movements and corresponding reduced support to the pipe [55]. A significant point relative to the Heger distribution is that the difficulty in obtaining a specified level of soil compaction under the haunches of the pipe is recognized in the soil pressure distribution by conservatively assuming all installations will have voids and soft inclusions in the haunch area. In reality, the haunch area has a soft spot about 30 degree from the invert, which is taken into account in model with the “void” zone. Material and compaction levels on the lower side also have a significant effect on the soil-structure interaction. The abovementioned characteristics of the soil-structure interaction were modeled using the finite element computer program. The analyses for concrete pipes were based on the four Standard Installations developed by ACPA and adopted by AASHTO. However, Proposed Installations were used rather than the four types of AASHTO Standard Installation in this study. Proposed Installations consist of TREATED and UNTREATED installations, which conform to Type 3 and Type 4 AASHTO Standard Installations, respectively. A verification of the validity of Proposed Installations is presented in this chapter. The foundation and compaction level of the side fill have significant influence on the behavior of box culverts. In this study, the effect of different foundations and compaction levels on the side fill of box culverts was evaluated and quantified using the finite element method.

The primary objective of this study was to evaluate the effect of the main variables and the effectiveness of the imperfect trench installation. The results will provide an increased understanding and developed design guides for imperfect trench installation. The width, height, location, and properties of soft materials were used as the variables of imperfect trench installation in this study. Finite element programs introduced in Chapter 3 were used to estimate earth pressure around buried conduits. Finite element modeling techniques and properties for analyses are presented in Chapter 3. Terminology for pipe parameters used in the analyses is shown in Figure 4.1.

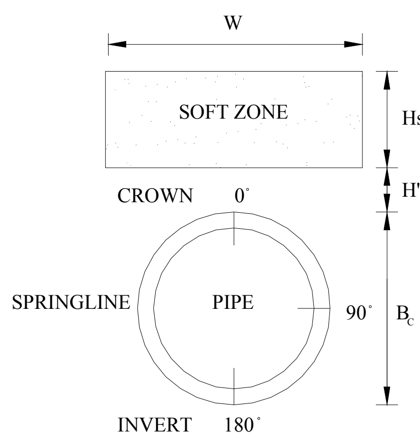


Figure 4.1 Terminologies for Pipe Parameters

4.2 Concrete Roadway Pipe

4.2.1 Verification of the Validity of Proposed Installations

Proposed Installations in Figures 4.2 and 4.3 are used in place of the four types of AASHTO Standard Installation in this study. Proposed Installations consist of a combination of TREATED and UNTREATED installations, which conform to Type 3 and Type 4 AASHTO Standard Installations, respectively. A Type 1 AASHTO Standard Installation requires the highest construction quality and degree of inspection.

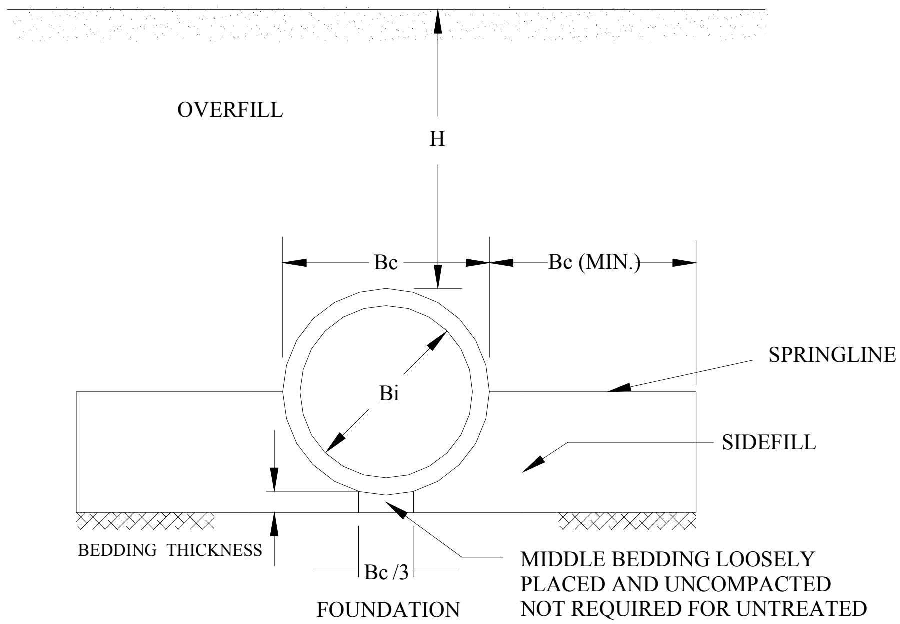


Figure 4.2 Proposed Embankment Installations for Concrete Round Pipes

Table 4.1 Proposed Embankment and Imperfect Trench Soils and Minimum Compaction Requirements for Concrete Round Pipes

TYPE	BEDDING THICKNESS	SIDEFILL COMPACTION
TREATED	Bc/24" MIN., NOT LESS THAN 3" . IF ROCK FOR FOUNDATION USE Bc/12" MIN., NOT LESS THAN 6"	85% SW, 90% ML 0R 95% CL
UNTREATED	NO BEDDING REQUIRED EXCEPT IF ROCK FOUNDATIONS USE Bc/12" MIN., NOT LESS THAN 6"	NO COMPACTION REQUIRED EXCEPT IF CL, USE 85%

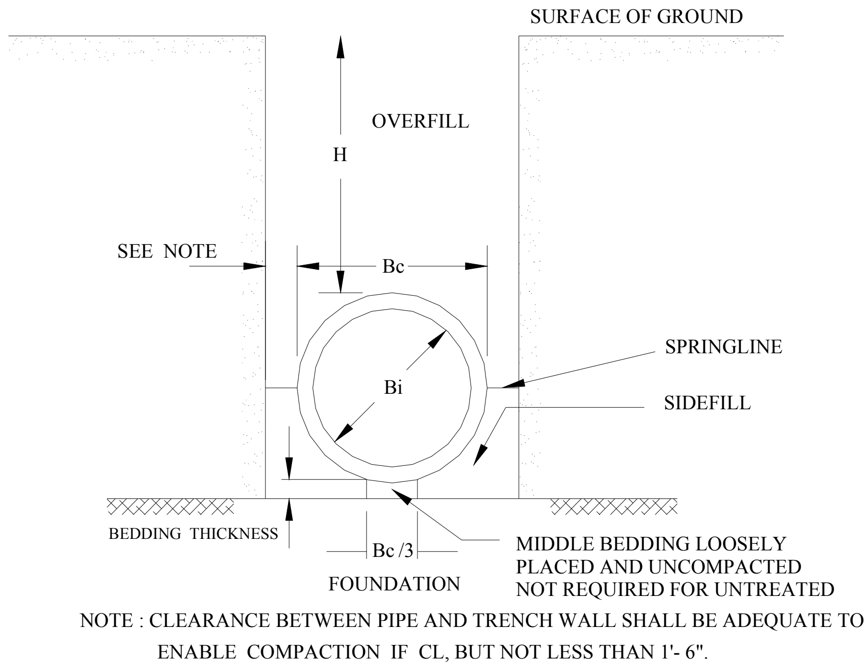


Figure 4.3 Proposed Trench Installations for Concrete Round Pipes

Table 4.2 Proposed Trench Soils and Minimum Compaction Requirements for Concrete Round Pipes

TYPE	BEDDING THICKNESS	SIDEFILL COMPACTION
NO ADVANTAGE OF TREATED BEDDING	Bc/24" MIN., NOT LESS THAN 3" . IF ROCK FOR FOUNDATION USE Bc/12" MIN., NOT LESS THAN 6"	NO COMPACTION REQUIRED EXCEPT IF CL, USE 85%

For Type 2 and Type 3, construction quality required is reduced. The earth load, however, shows only a slight difference (within 5%) between Types 1, 2, and 3 as shown in the results for the ACPA and finite element analyses in Figure 4.4. In particular, the difference in the earth loads between Type 2 and Type 3 is negligible. Therefore, Proposed Installations are an effective installation method for maximizing bedding effect with the least effort for construction quality. The results of a survey of the five neighboring states showed that most states use only one or two bedding types. These survey results are presented in the Appendices.

4.2.2 Embankment Installation

Pressure distributions for AASHTO Standard Embankment Installations are shown in Figure 4.5. For Types 1, 2, and 3, the pressure distributions show only a slight difference except for the case of type 4, which was developed for conditions where there is little or no control over either materials or compaction. As shown in Figure 4.5, low pressures of Type 1, 2, and 3 in invert are induced from a loose middle bedding. Maximum pressure occurs at about 155 degrees from the invert, which indicates why the material and construction quality of the haunch area are important in the pipe-soil system.

The principal objective of a soil-structure interaction analysis is to determine the earth load and its pressure distribution. In a finite element analysis, the results of the analysis will give the pressure distribution at each of the pipe nodes in the pipe model. The earth load is the summation of the downward vertical components of pressure. Similarly, the summation of the horizontal components of pressure in one direction equals the horizontal load on the pipe. For pipe installations with a vertical axis of symmetry, the laws of statics require that the total vertical earth load above the pipe springline is equal to twice the earth load thrust in the pipe wall at the springline. Thus, the VAF (Vertical Arching Factor) is calculated from the results of the soil-structure interaction analysis in Equation 4-1. Similarly, the total horizontal earth load on one side of the pipe is equal to the summation of the earth load thrusts in the pipe wall at the crown and the invert. Thus, the HAF (Horizontal Arching Factor) is calculated from the results of the soil-structure interaction analysis in Equation 4-2.

$$VAF = \frac{W_E}{PL} = \frac{2N_{sp}}{PL} \tag{4-1}$$

$$HAF = \frac{W_h}{PL} = \frac{N_c + N_i}{PL} \tag{4-2}$$

where,

PL = Prism Load

W_E , W_H = Total vertical and horizontal earth loads, lb/ft

N_{sp} = Thrust in the pipe wall at the springline

N_p = Thrust in the pipe wall at the crown

N_i = Thrust in the pipe wall at the invert

4.2.3 Imperfect Trench Installation

Applying this approach to the design of an imperfect trench installation follows the procedure shown in Figure 4.6. The objective of an imperfect trench installation is to reduce the earth load. The reduced fill height leads to a reduction in the earth load, which makes it possible to use a lower strength pipe. The pipe designed for the reduced fill heights can be used under the required fill height, which results in a reduction of the construction costs. The design examples for imperfect trench installation are presented in Chapter 7. Figure 4.7 shows the earth pressure distributions around the pipe for various heights of soft zone. This graph shows that the pressure on the pipe invert is not affected by the height of the soft zone. and the pressure on the pipe crown is reduced largely as the height of soft zone increases by Hs/D=1.0, the ratio of the height of the soft zone to the outside diameter of the pipe. In the case of Hs/D>1.0, the pressure distribution shows a negligible change as the height of the soft zone increases.

4.2.3.1 Optimum Geometries for the Soft Zone

Figure 4.8 shows that the highest earth load reduction occurs when the soft zone is placed immediately on the pipe. Figure 4.9 shows variations in the earth load reduction rates, with a diminishing return characteristic, as the Hs/B_C and W/B_C increase. This increase in the earth load reduction rate slows down after Hs/B_C=0.5 and W/B_C=1.5. In this study, the optimum geometries of soft zone are suggested to be Geometry I and Geometry II, based on numerous parameter studies for Hs, H', and W of soft zone. Figure 4.10 shows how a lower strength concrete pipe can be used by using an imperfect trench installation with a relatively high fill height, which would result in a significant reduction in the construction costs.

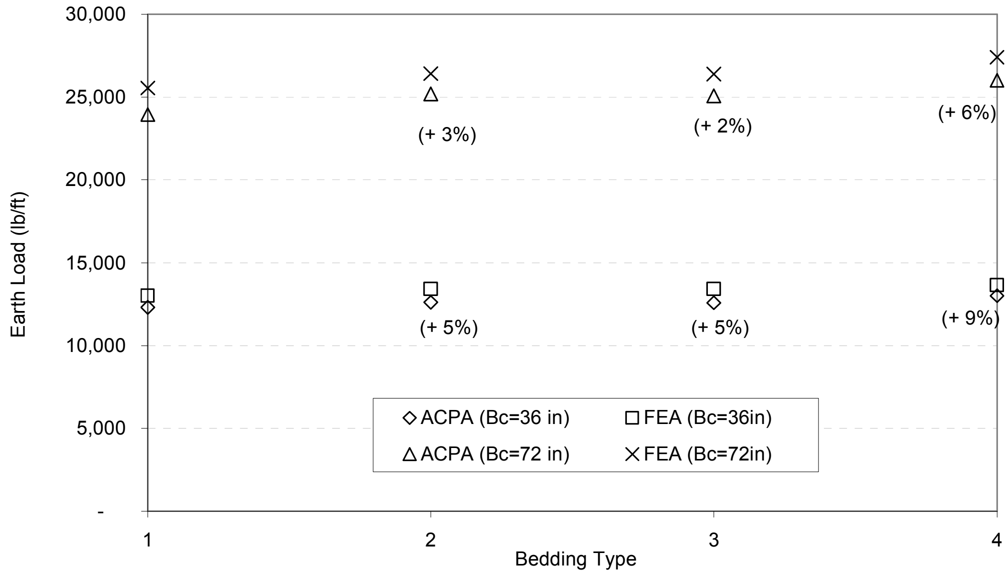


Figure 4.4 Comparison of Earth Load for the AASHTO Standard Installations

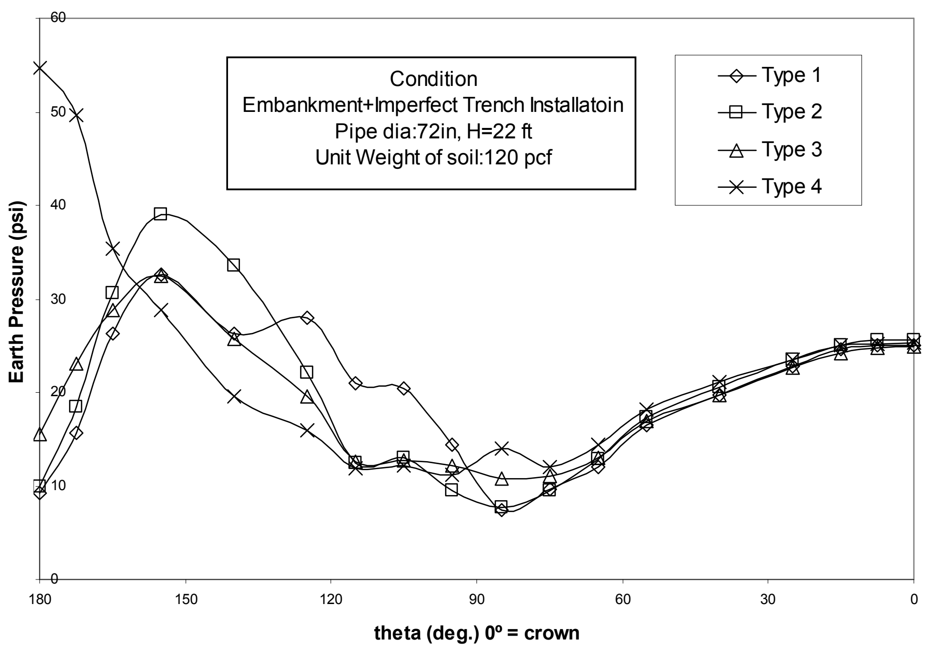


Figure 4.5 Pressure Distributions for the AASHTO Standard Installations

Given: H_{req} (Required Fill Height)

$H_{equiv} = H_{req} (1 – R)$
where R= reduction rate of soft material (0 ~1)

Determination of D-load and Pipe Class
from
AASHTO Indirect Pipe Selection

Figure 4.6 Design Application of Imperfect Trench Installation

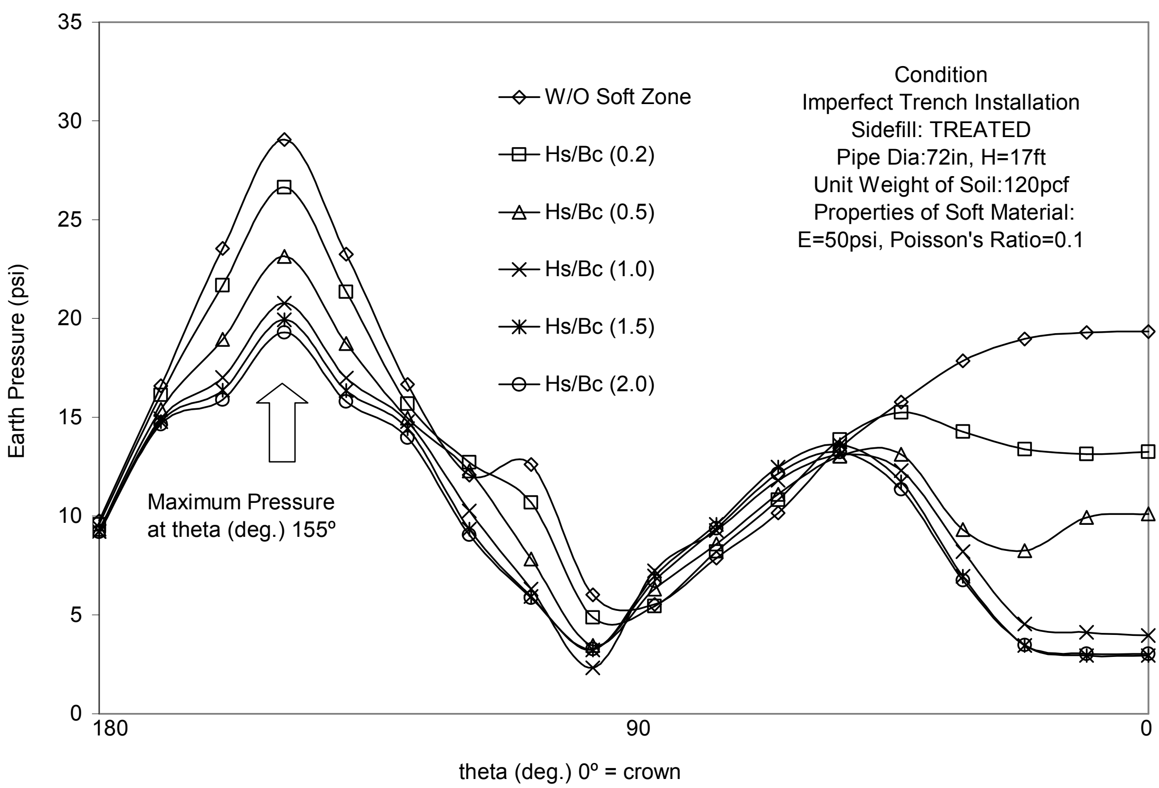


Figure 4.7 Pressure Distribution vs. Height of Soft Zone (H_s)

4.2.3.2 Geometry I and II of Soft Zone

Figure 4.11 shows the effect of different soft zone geometries on the pressure distribution around the pipe. Geometry I and Geometry II, shown in Figures 4.12 and 4.13 are highly effective in reducing the top and bottom pressures on the pipe. Pronounced differences in the load reduction effects of Geometry I and Geometry II are shown in the lower haunch area to about 155 degrees and in the area up to 90 degrees from the crown. Geometry II was developed to maximize the reduction effect of imperfect trench installation and is more effective for the reduction of pressure, which is caused by the wide contact area between the soft material and the pipe. Figure 4.11 also shows the superiority of Geometry II for the reduction of the D-load.

Figure 4.15 shows how the TREATED installation takes advantages over the UNTREATED installation, not only in the embankment installation, but also in an imperfect trench installation for reducing the effect of the earth load.

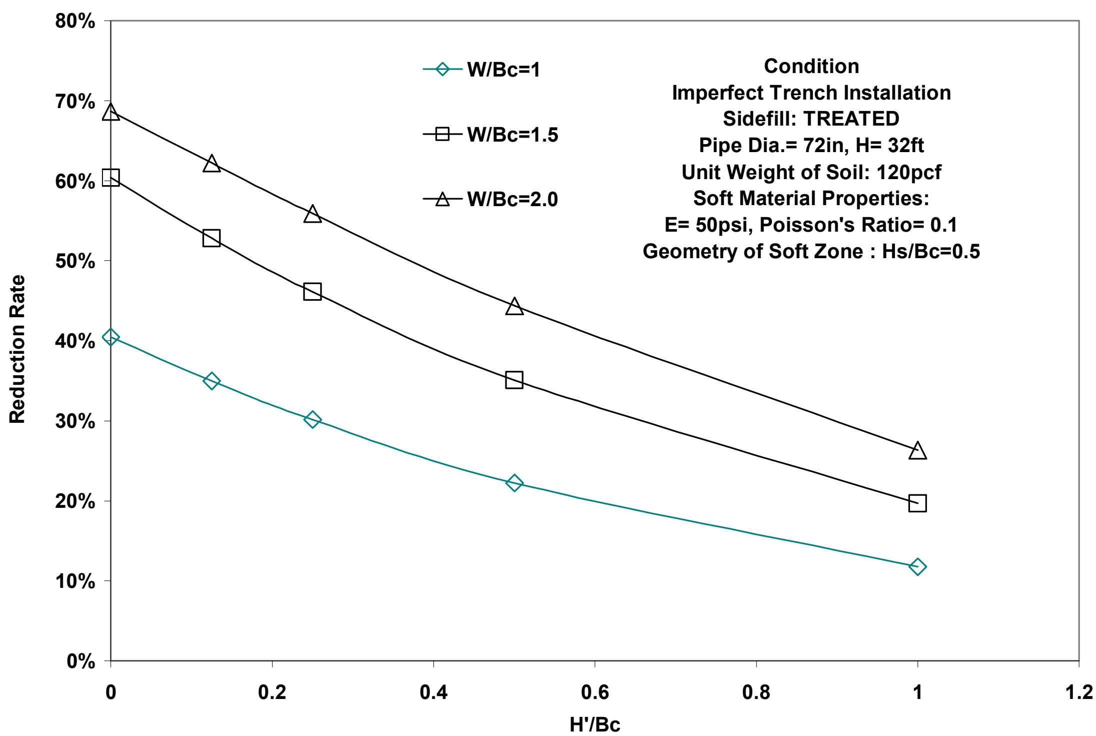


Figure 4.8 Reduction Rate vs. H'/B_c (Different W/B_c)

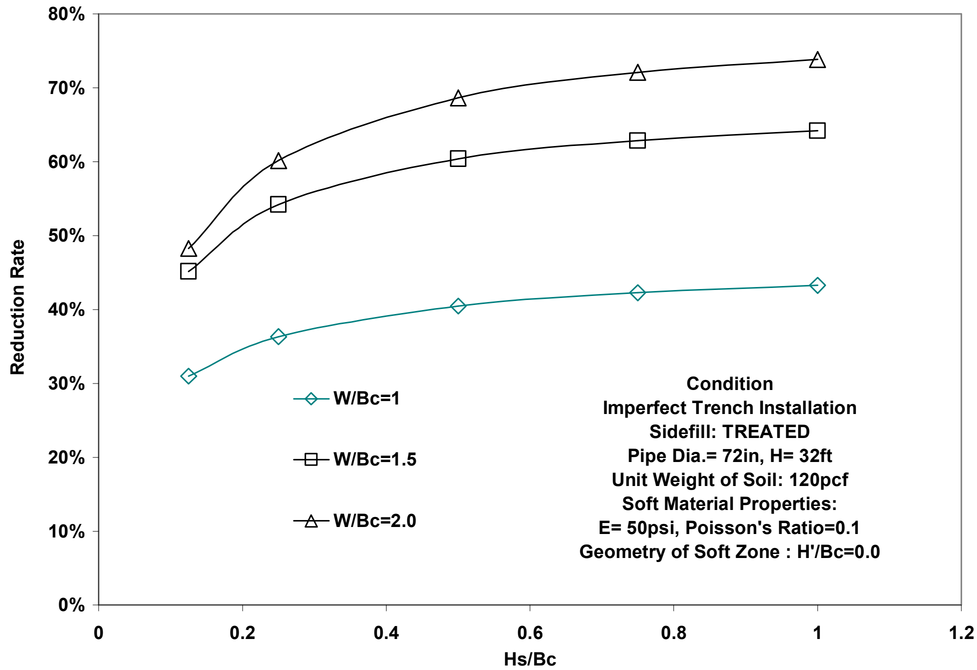


Figure 4.9 Reduction Rate vs. H_s/B_c (Different W/B_c)

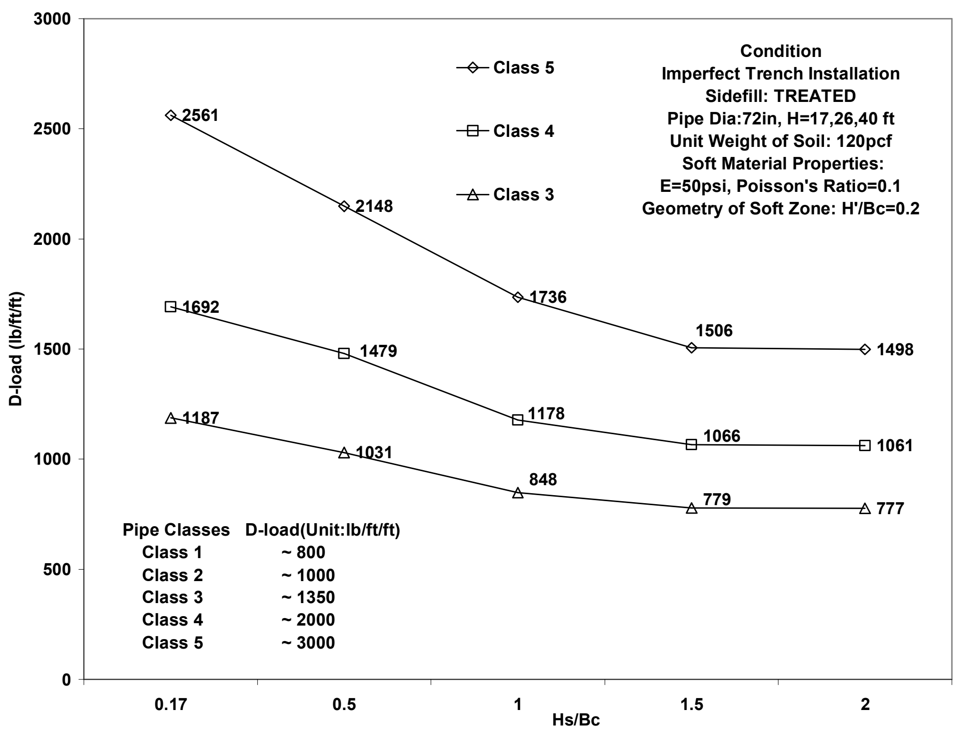


Figure 4.10 D-load Reduction vs. H_s/B_c

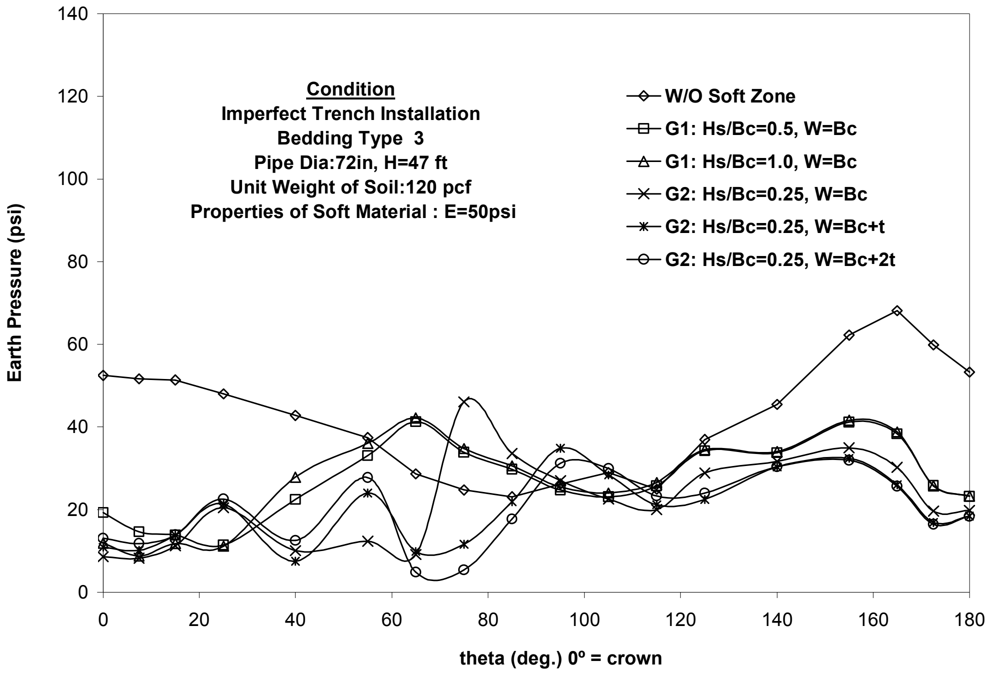


Figure 4.11 Pressure Distributions vs. Height (H_s) and Width (W) of Soft Zone

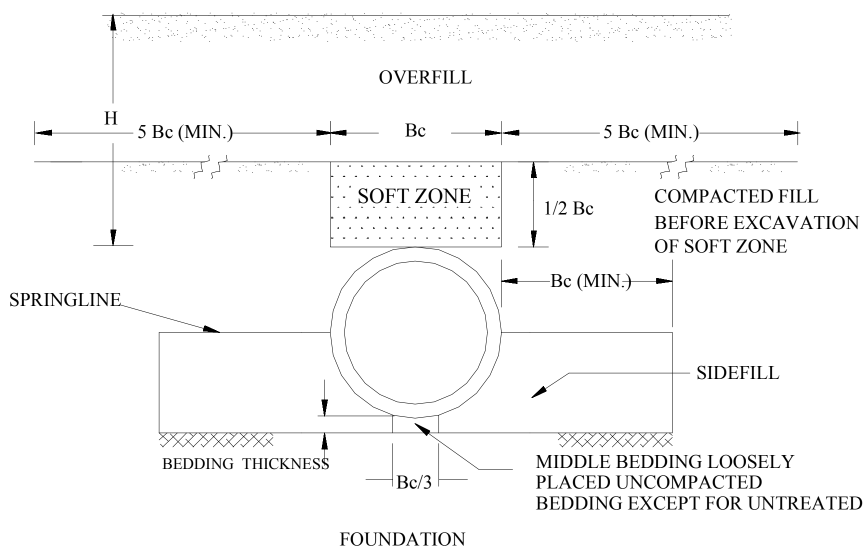


Figure 4.12 Geometry I of Soft Zone for Concrete Round Pipes

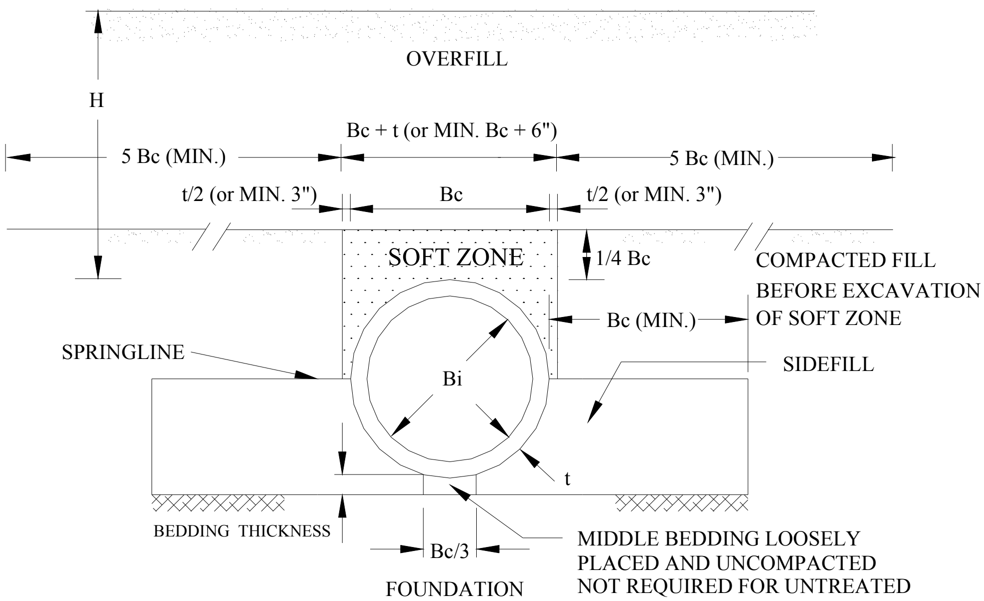


Figure 4.13 Geometry II of Soft Zone for Concrete Round Pipes

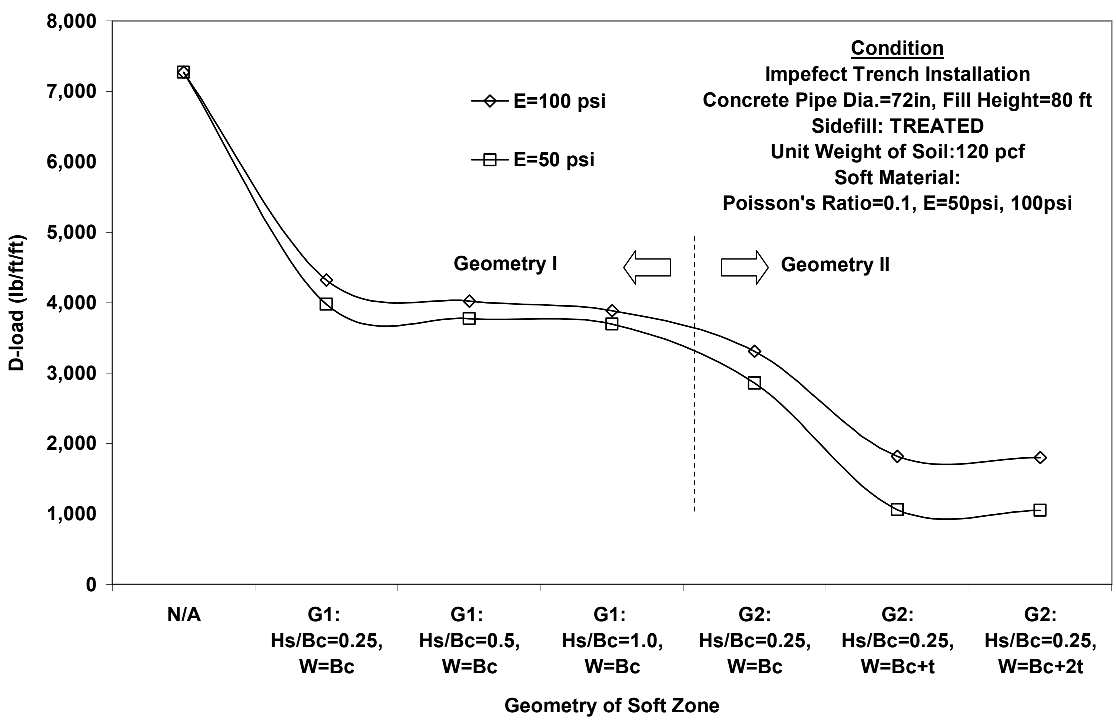


Figure 4.14 D-load Reduction Rate vs. Geometry Type of Soft Zone

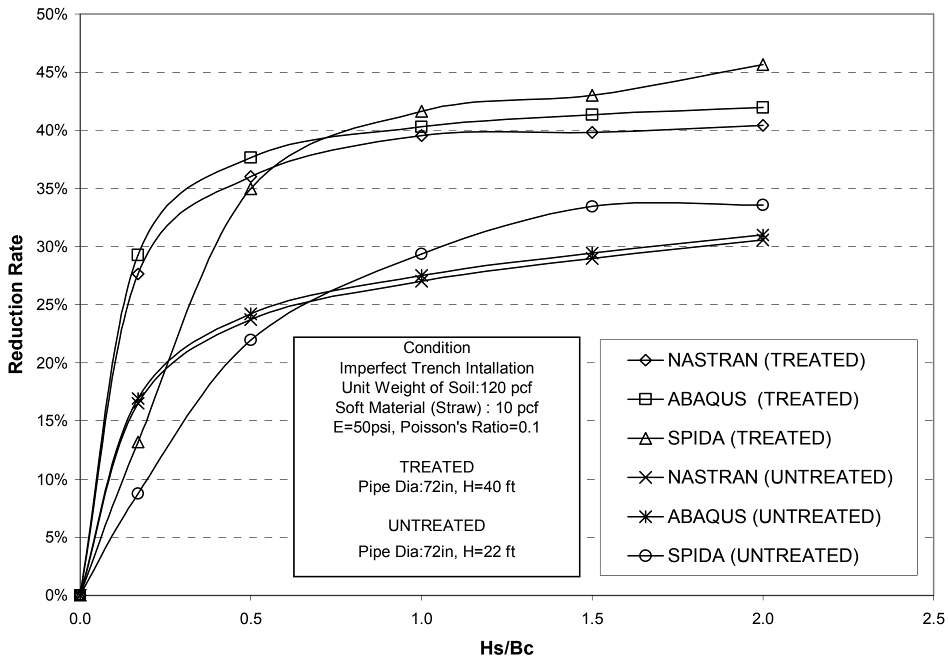


Figure 4.15 Comparison of Reduction Rate between TREATED and UNTREATED installations

4.2.3.3 Equations Developed for the Reduction Rate for Concrete Pipe

The properties of the soft material, fill height, width, height, and location of soft zone, and pipe or culvert size can be considered in terms of the variables affecting the reduction rate for imperfect trench installations. In this study, the location, height, and width of the soft zone were confined to two types, Geometry I and Geometry II. The optimum geometries of the soft zone were discussed earlier in this chapter. Therefore, the remaining variables will now be considered in order to derive an equation for reduction rate.

The effect of pipe size and fill height on the reduction rate show only slight differences, less than 1% regardless of size, and less than 1% per 10 ft, as shown in Figure 4.16. The average Poisson's ratio of EPS in compression tests was negative, with a value of -0.01[57]. In this study, Poisson's ratio was taken to be 0.1. Poisson's ratio has an insignificant effect on the reduction rate, as shown in Figure 4.17.

In conclusion, the reduction rate is highly affected by the modulus of elasticity (E_s) of soft materials, which means the reduction rate is a function of E_s. Based on the results of numerous parameter studies with Finite Element programs, proposed equations for reduction rates were derived by means of a linear regression method. The proposed equations for the condition of the foundation and sidefill compaction are presented in Table 4.3.

4.2.3.4 Verification of Proposed Equations

To verify the proposed equations, their results were compared with those obtaining using FEA. The relationship between the required fill height and the equivalent fill height were expressed as H_{equiv} = H_{req} (1 – R). The equations for reduction rate, R, are given in Table 4.3. The results obtained using the proposed equations and those of the Finite Element Analyses were in good agreement, with less than a 2% difference, except for the case where the modulus of elasticity (E_s) of the soft material was 50 psi in Geometry II. In this case, the reduction rate in FEA showed an excessively high value. E_s=100psi was thus the minimum used for the conservative design in this study.

Table 4.3 Proposed Equations for the Reduction Rate of Soft Materials – Concrete Pipe

Soft Zone	Foundation	Sidefill Compaction	Reduction Rate Equations
Geometry I	Yielding Or Unyielding	TREATED	$R = 0.445 - 6\times10^{-4} E_s$
		UNTREATED	$R = 0.345 - 4\times10^{-4} E_s$
Geometry II	Yielding Or Unyielding	TREATED	$R = 0.824 e^{-0.0027 E_s}$
		UNTREATED	$R = 0.821 e^{-0.0037 E_s}$

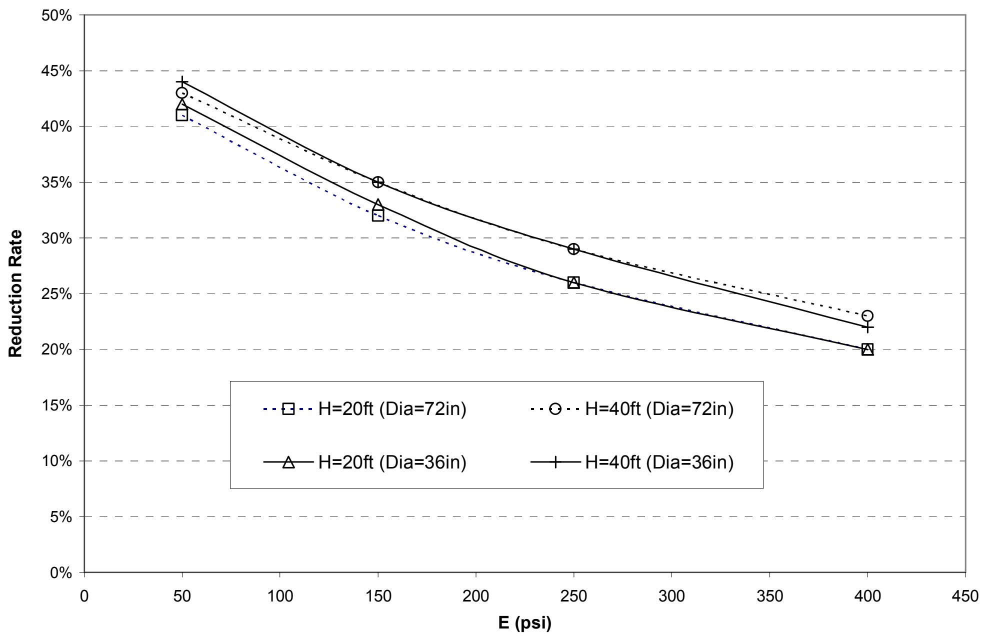


Figure 4.16 Reduction Rate vs. Pipe Size and Fill Height

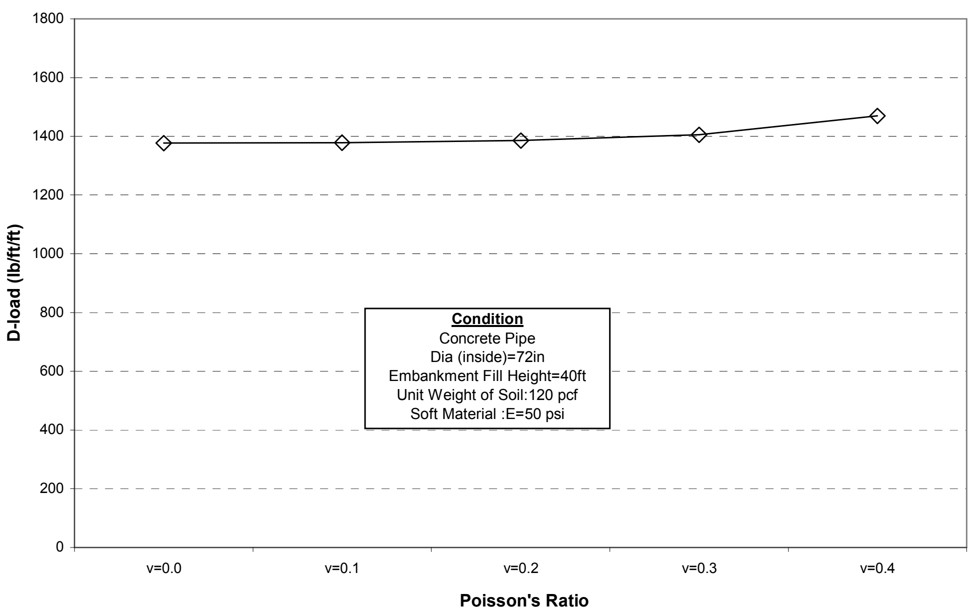


Figure 4.17 Reduction Rate vs. Poisson’s Ratio

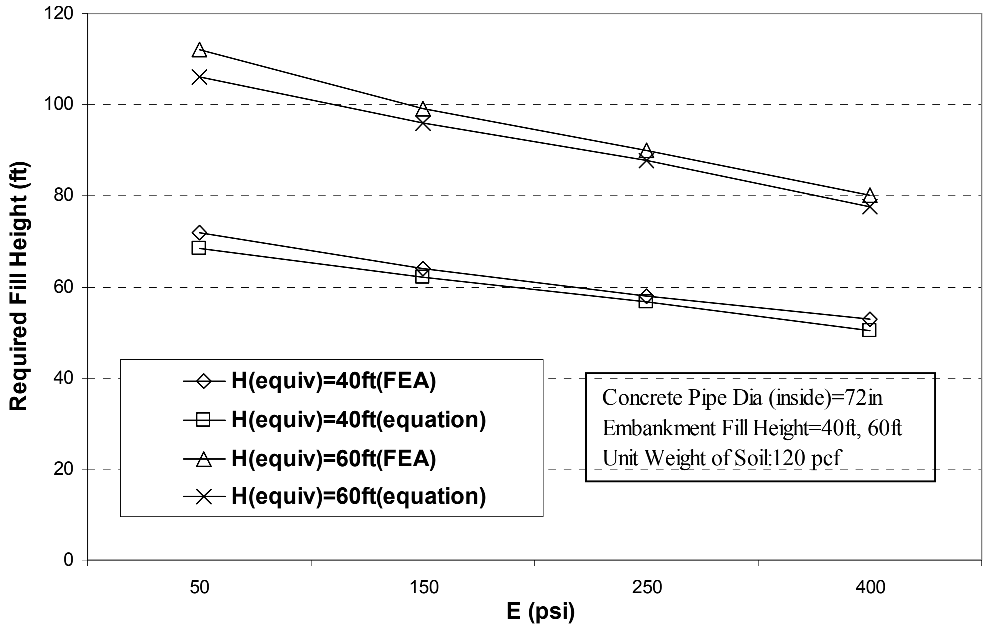


Figure 4.18 Proposed Equations vs. FEA (Geometry I, TREATED)

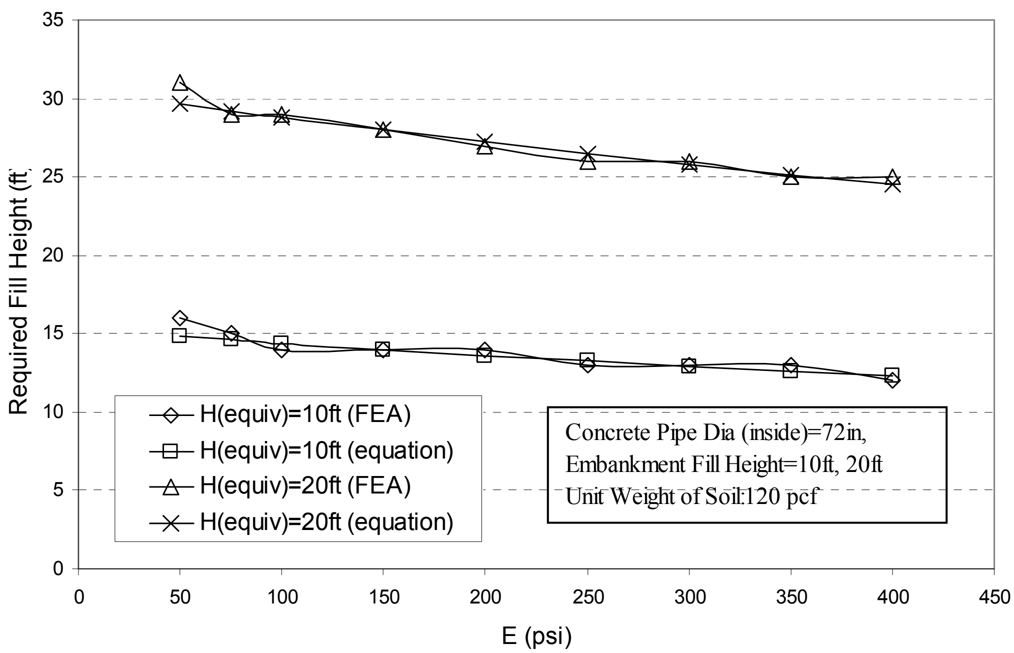


Figure 4.19 Proposed Equations vs. FEA (Geometry I, UNTREATED)

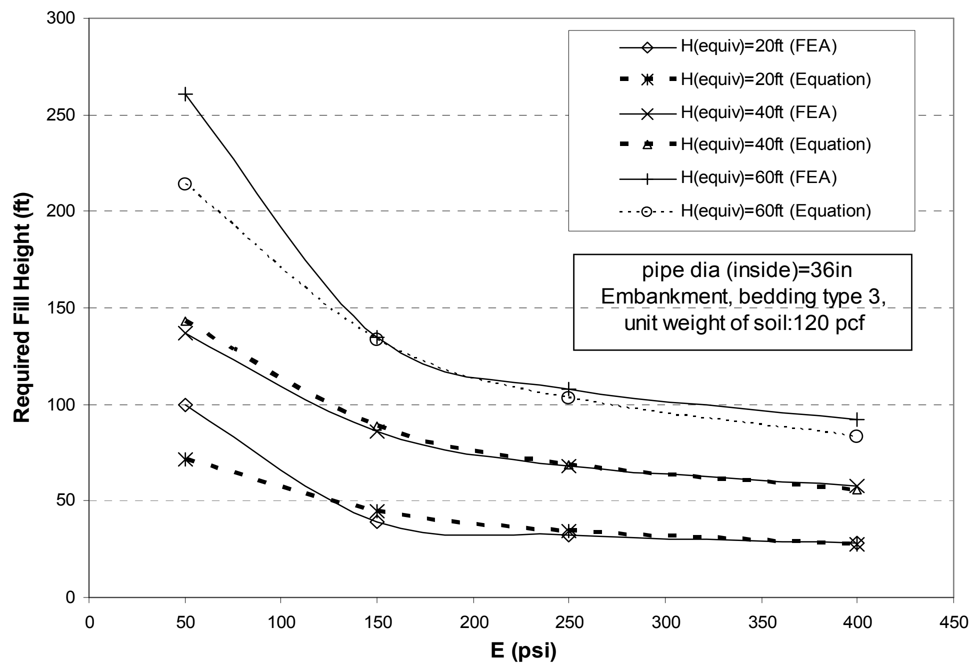


Figure 4.20 Proposed Equations vs. FEA (Geometry II, TREATED)

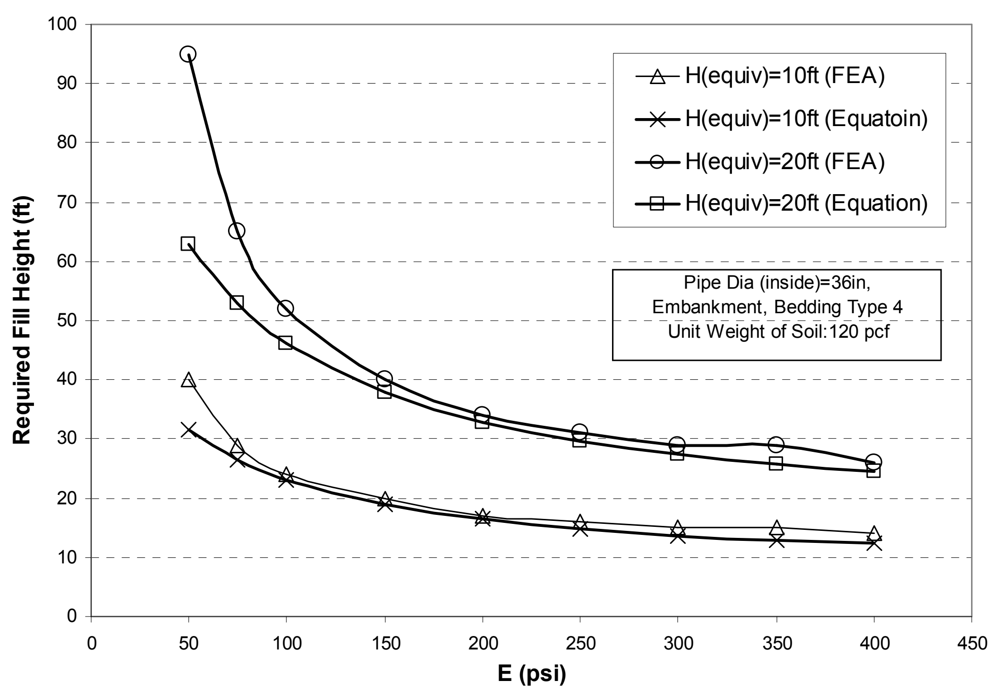


Figure 4.21 Proposed Equations vs. FEA (Geometry II, UNTREATED)

4.3 Box Culverts

It has been generally accepted that the vertical loads on box culverts are composed of the top earth load and the dead load. To take this into account, the total load acting on the bottom slab should be equal to the summation of the top earth load and dead load. However, the results of FEA show that the magnitude of the bottom load always exceeds the summation of the top earth load and dead load. In particular, this inequality of the vertical load is much bigger for an UNTREATED sidefill, which is why the shear force on a sidewall occurs in a downward direction. Figure 4.23 shows that the total earth load acting on the bottom slab is the summation of the top slab earth load, dead load, and shear force on the sidewall. The portion of the shear force for the total vertical load reaches 24 ~ 27% for the case of UNTREATED sidefill, which is why the design loading of box culverts should be based on the bottom pressure. Figure 4.23 also shows that TREATED sidefill is more effective in reducing the shear force on the sidewall than UNTREATED sidefill. Terminologies used in the analysis of box culverts are shown in Figure 4.22.

4.3.1 Determination of Compaction Level

The provisions for determining the compaction level in the sidefill are not specified in AASHTO [6]. The geometry of compacted area and minimum compaction requirements of soils for the bedding and sidefill are proposed in this study. Figure 4.24 shows that the compaction level at the sidefill need not exceed half the vertical rise of the box culvert. Figures 4.25 and 4.26 show drawings of the proposed installations for box culverts.

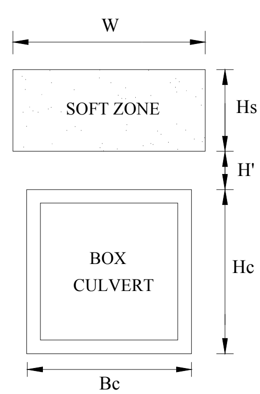


Figure 4.22 Terminologies used for the Box Culvert Parameters

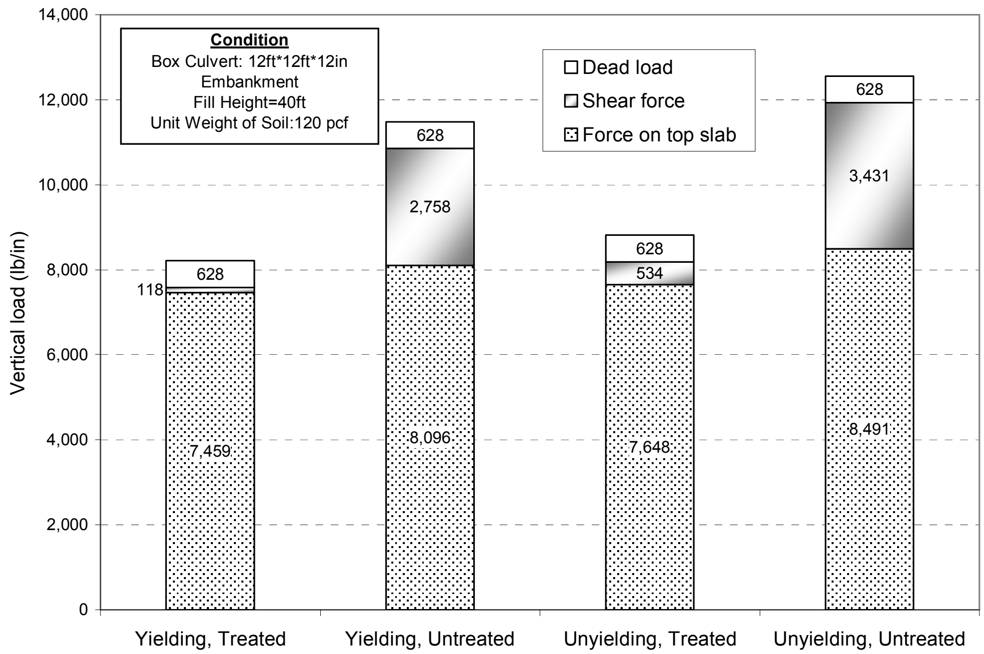
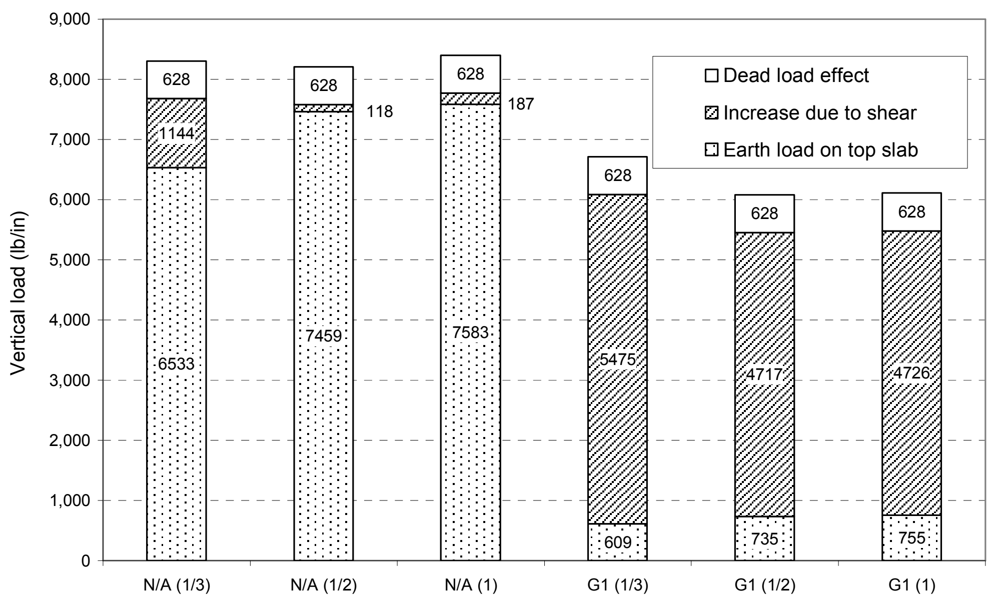


Figure 4.23 Vertical Load Distribution vs. Foundation and Sidefill Compaction



note: 1. Box Culvert (12ft x12ft), fill height = 40ft, yielding foundation
2. N/A and G1 means embankment installation and Geometry I of imperfect trench installation
3. (1/3) means compaction level is 1/3 of vertical rise of box culvert

Figure 4.24 Effect of Compaction Level at the Sidefill.

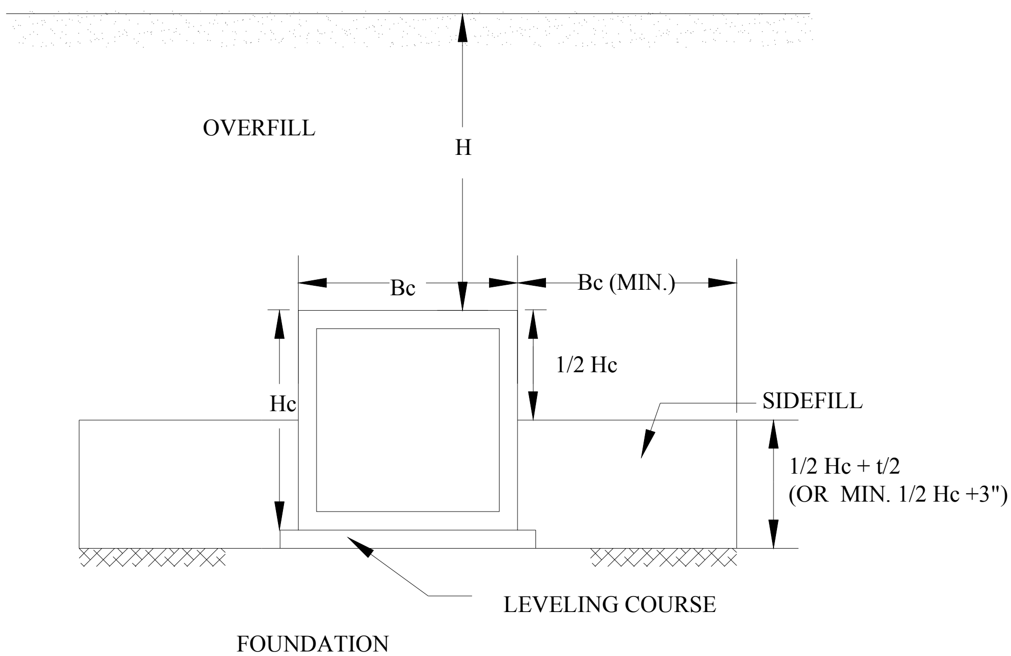
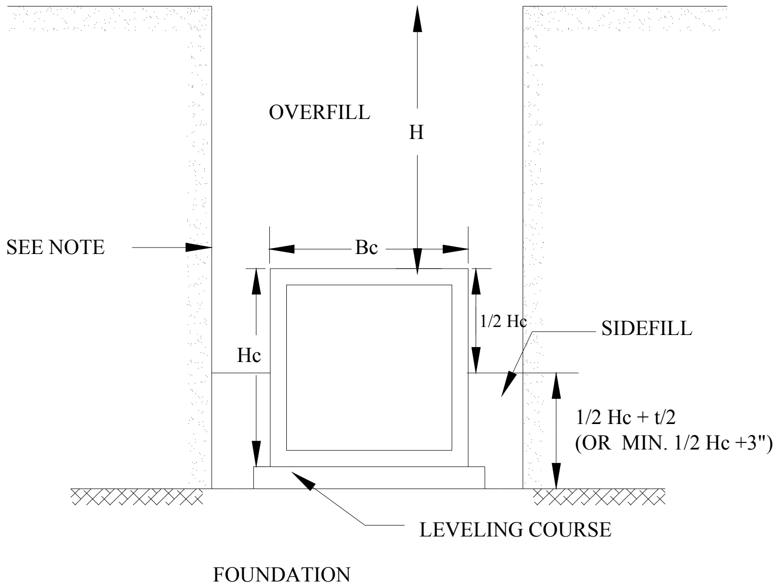


Figure 4.25 Embankment Installations for Box Culverts

Table 4.4 Embankment Installation Soils and Minimum Compaction Requirements for Box Culverts

TYPE	LEVELING COURSE BEDDING THICKNESS	SIDEFILL
TREATED	USE t / 2 MIN. NOT LESS THAN 3". IN YIELDING AND UNYIELDING FOUNDATION	85% SW, 90% ML 0R 95% CL
UNTREATED	USE t / 2 MIN. NOT LESS THAN 3". IN YIELDING AND UNYIELDING FOUNDATION	NO COMPACTION REQUIRED EXCEPT IF CL, USE 85% CL



NOTE: CLEARANCE BETWEEN BOX AND TRENCH WALL SHALL BE ADEQUATE TO ENABLE COMPACTION IF CL, BUT NOT LESS THAN 1'-6".

Figure 4.26 Trench Installations for Box Culverts

Table 4.5 Trench Installation Soils and Minimum Compaction Requirements for Box Culverts

TYPE	LEVELING COURSE BEDDING THICKNESS	SIDEFILL
NO ADVANTAGE OF TREATED BEDDING	USE t / 2 MIN. NOT LESS THAN 3". IN YIELDING AND UNYIELDING FOUNDATION	NO COMPACTION REQUIRED EXCEPT IF CL, USE 85% CL

4.3.2 Effective Density

The total earth load includes the additional load as well as the prism load of soil directly above the box culverts. The additional load is accounted for by using a soil-structure interaction factor, or effective density, that accounts for the type and conditions of installation. This factor is multiplied by the prism load to give the total load of soil on the box culverts. The Soil-structure interaction factor, F_e in AASHTO is given in Chapter 7. The provisions of F_e in AASHTO are based only on the top earth pressure, which can lead to unconservative design. Therefore, F_e including the shear force effect should be used in the design loading of box culverts. The effective density as a function of the ratio of the fill height to the out-to-out horizontal span of the culvert is plotted in Figures 4.27 and 4.28. The proposed equations in Table 4.6 are derived based on Figures 4.27 and 4.28 by means of a linear regression method. The equations use average values for yielding and unyielding foundations for the simplified design, as the effect of the foundation on effective density is insignificant, a less than 10% difference.

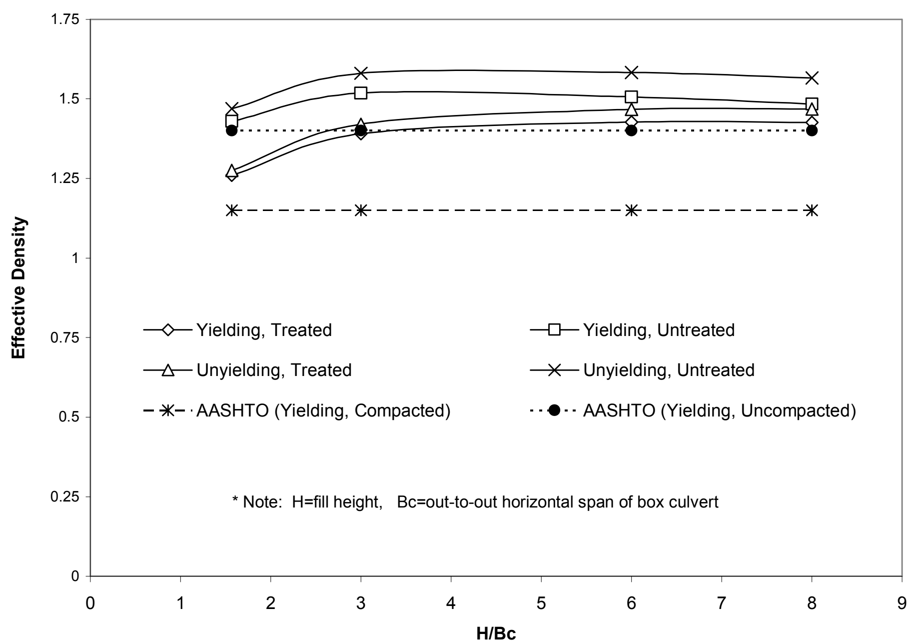


Figure 4.27 Effective Density (Top Earth Load only) vs. H/B_c

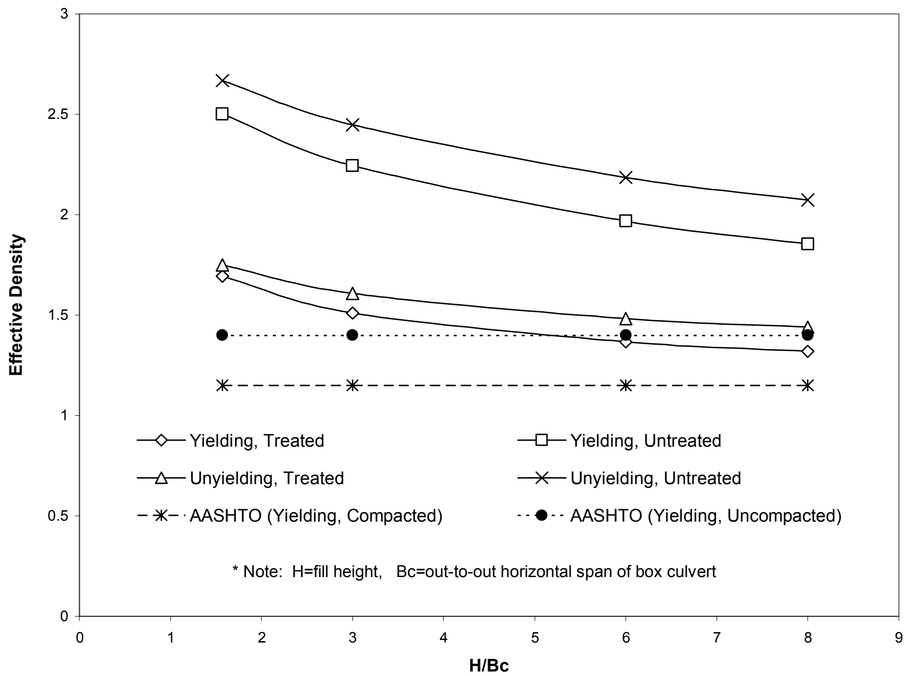


Figure 4.28 Effective Density (including Shear Force) vs. H/B_c

Table 4.6 Proposed Equations for Soil-Structure Interaction Factor

Sidefill	F_{e1} for top earth pressure	F_{e1} for Bottom pressure
TREATED	$F_{e1} = -0.009(H/B_c)^2 + 0.109(H/B_c) + 1.131$	$F_{e1} = 1.823(H/B_c)^{-0.136}$
UNTREATED	$F_{e1} = -0.007(H/B_c)^2 + 0.077(H/B_c) + 1.357$	$F_{e1} = 2.803(H/B_c)^{-0.169}$

4.4 Imperfect Trench Installation

The reduction rate of bottom pressure for an imperfect trench installation is relatively low in comparison with that for top pressure. This is caused by the phenomenon where the reduced top earth pressure that results from the reverse arching effect due to the relative settlement in the soft zone moves to the sidewall, which in turn results in an increase in the downward shear force. The results from FEA show that the inclusion of soft material between the sidewall and soil can prevent an increase in the shear force due to the imperfect trench installation. Geometry II of the soft zone was developed to prevent the increase of shear force. Figure 4.35 proves that Geometry II of the soft zone is highly effective in reducing the shear force on the sidewall.

Figure 4.29 shows that the highest earth load reduction occurs when the soft zone is placed immediately on the box culverts. Figure 4.30 shows variations in the earth load reduction rates with diminishing return characteristics as the H_s/H_c ratio increases. The increase in the earth load reduction rate slows down after H_s/B_c=0.5. In this study, optimum geometries for the soft zone are proposed as Geometry I and Geometry II based on Figures 4.29, 4.30, 4.31, and 4.32 and numerous parameter studies for H_s, H', and W of soft zone. Geometries I and II of the soft zone are shown in Figures 4.33 and 4.34. Figure 4.37 shows that Geometry II is more effective than Geometry I in reducing the earth load, which can be explained as the effect of the shear force reduction in Geometry II.

4.4.1 Pressure Distribution on the Whole Sides of Box Culverts

Typical effective density distributions on the top slab of box culverts with H'/H_c =1.25 are shown in Figure 4.36. Figure 4.37 shows the typical effective density distributions with W/B_c =1 and H'/H_c =1.25. This study focused on the pressure distribution on the bottom slab, and sidewall, as well as the top slab. As mentioned earlier, the maximum pressure on box culverts generally occurs at the bottom slab, which is due to the increased downward force resulting from the shear effect at the sidewall. Geometry II, with a soft zone next to the sidewall, is a highly effective method for reducing the shear force on the sidewall, which also results in a reduction of the bottom pressure. Figure 4.38 shows the pressure distributions on all sides and the reduction effect of the increased shear force due to the imperfect trench installation.

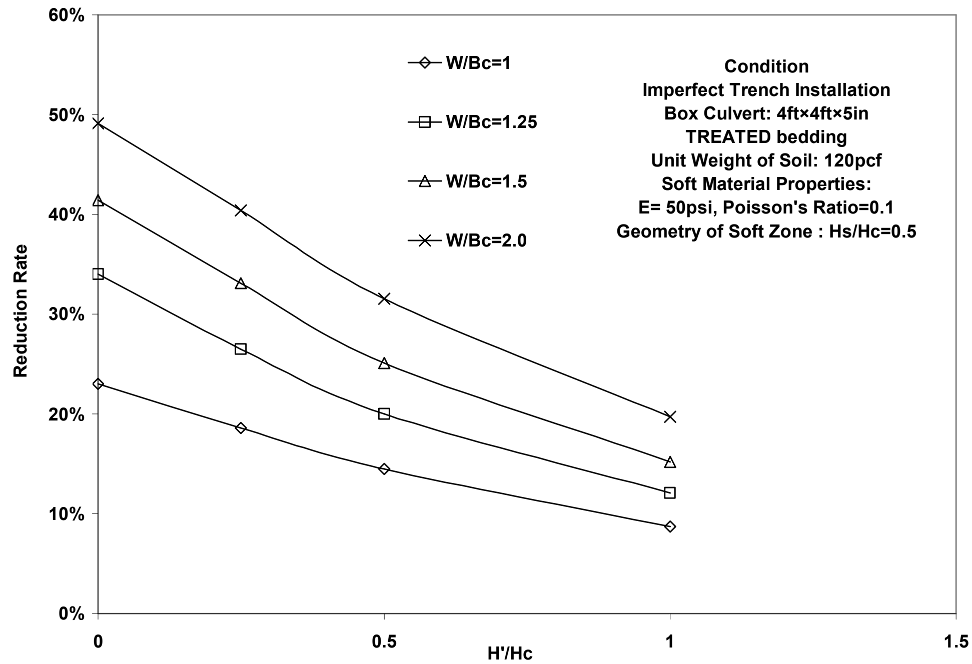


Figure 4.29 Reduction Rate vs. H'/H_c (Different W/B_c)

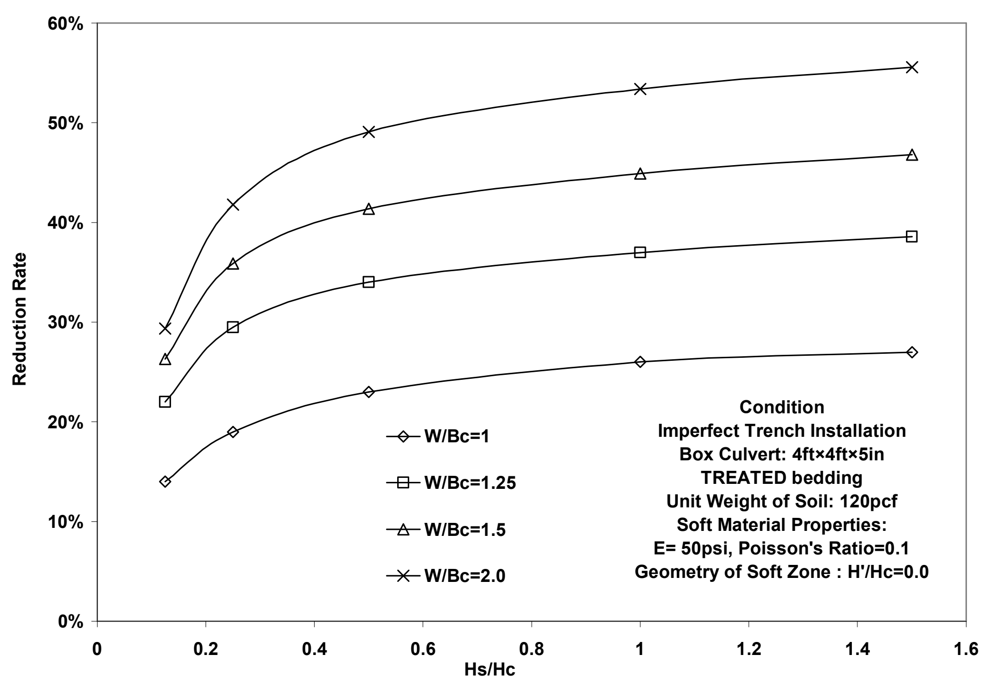


Figure 4.30 Reduction Rate vs. H_s/H_c

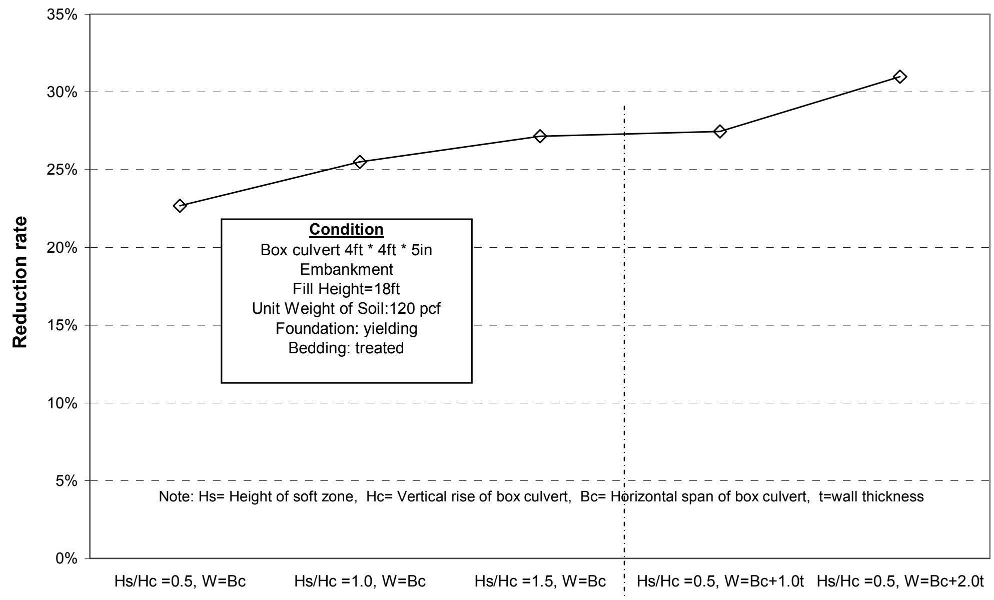


Figure 4.31 Reduction Rate vs. W, Width of Geometry I Soft Zone

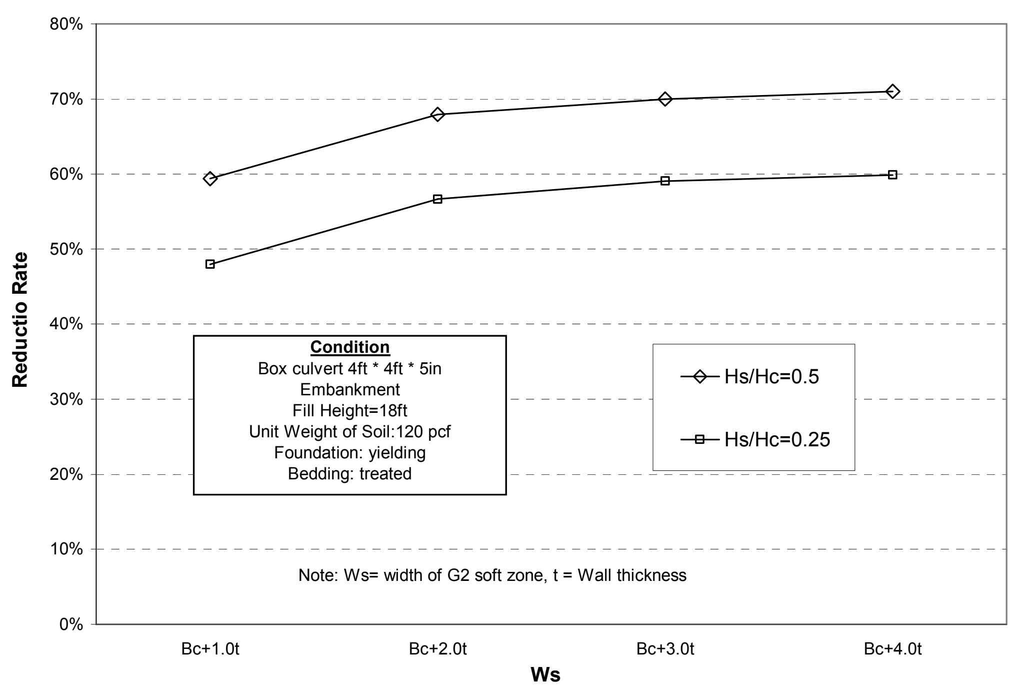


Figure 4.32 Reduction Rate vs. W_s, Width of Geometry II Soft Zone

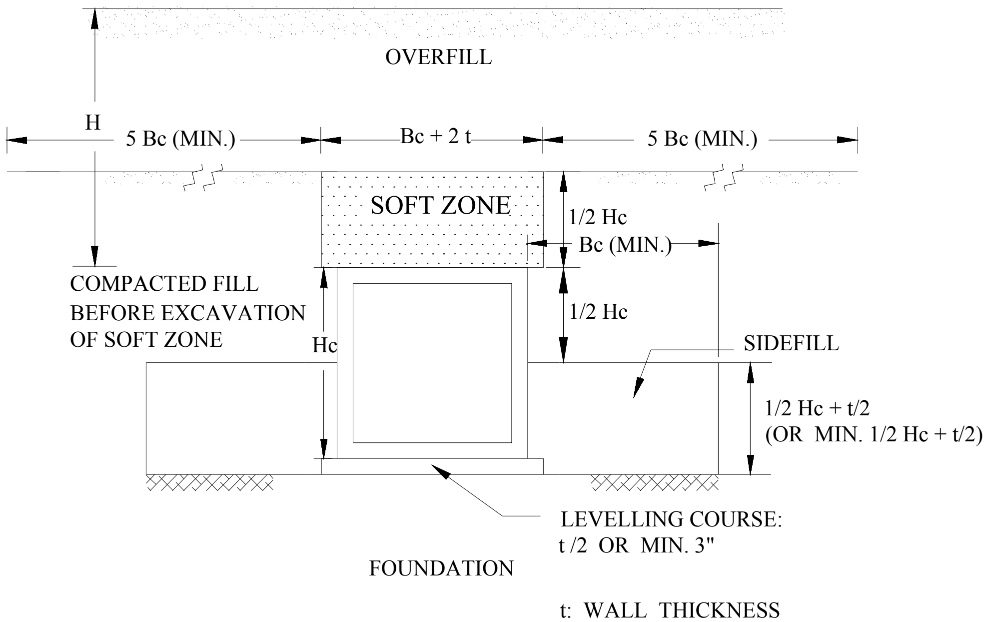


Figure 4.33 Geometry I of Soft Zone for Box Culverts

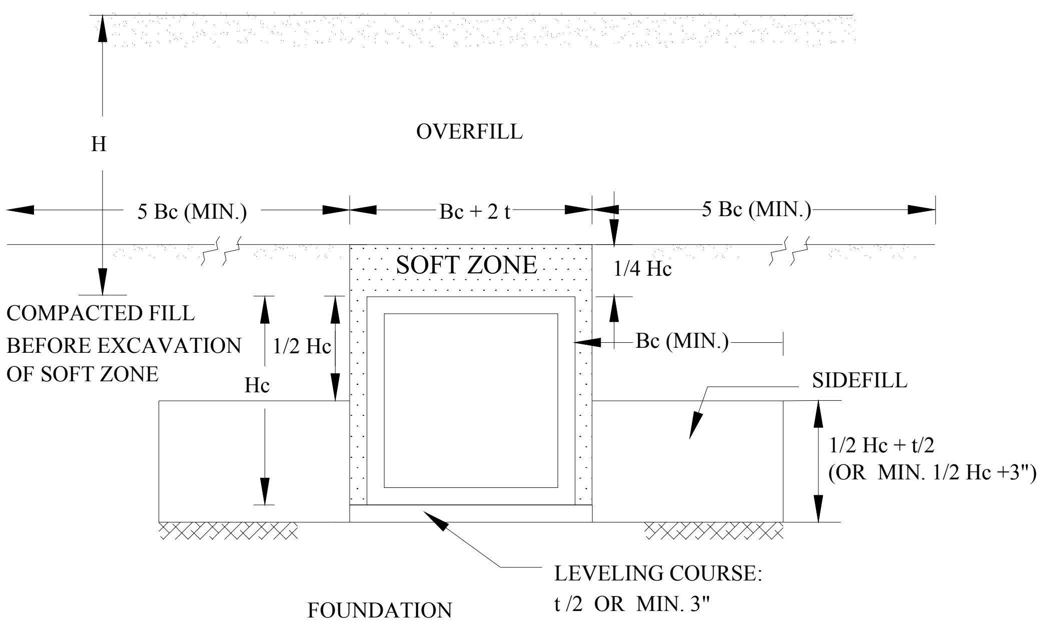


Figure 4.34 Geometry II of Soft Zone for Box Culverts

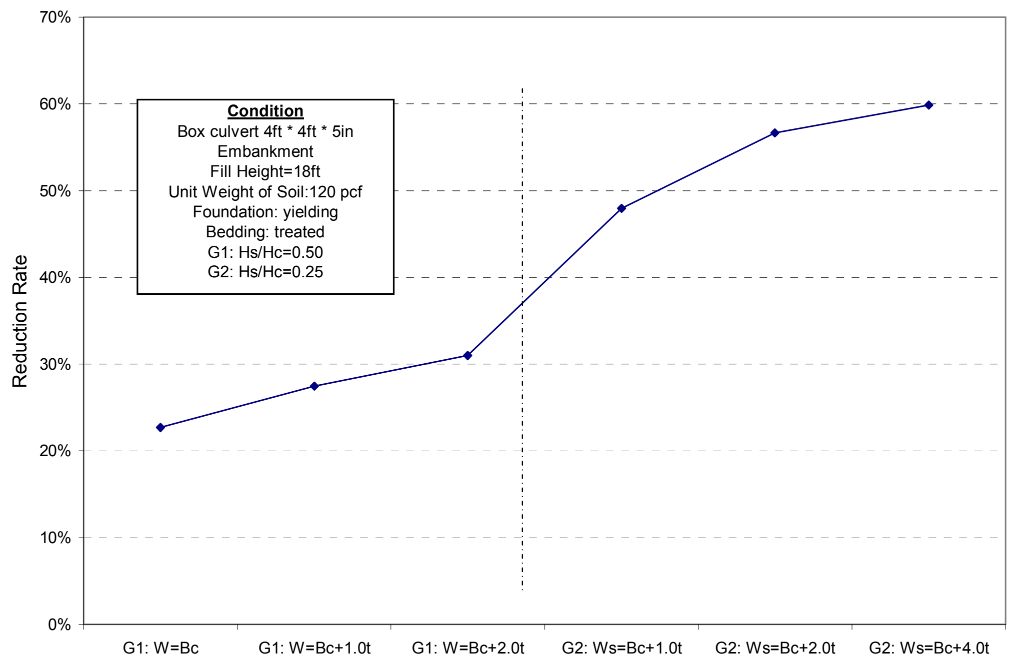


Figure 4.35 Reduction Rate Comparison of Geometries I and II of the Soft Zone

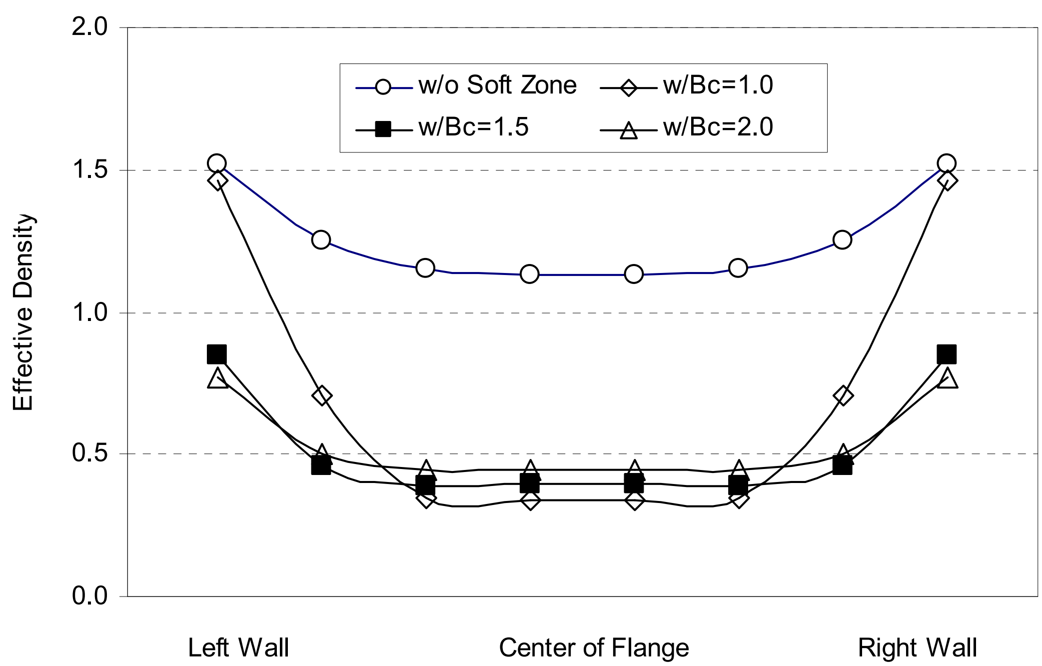


Figure 4.36 Effective Density Distribution acting on the Top Slab (Different W/B_c, H'/H_c=1.25)

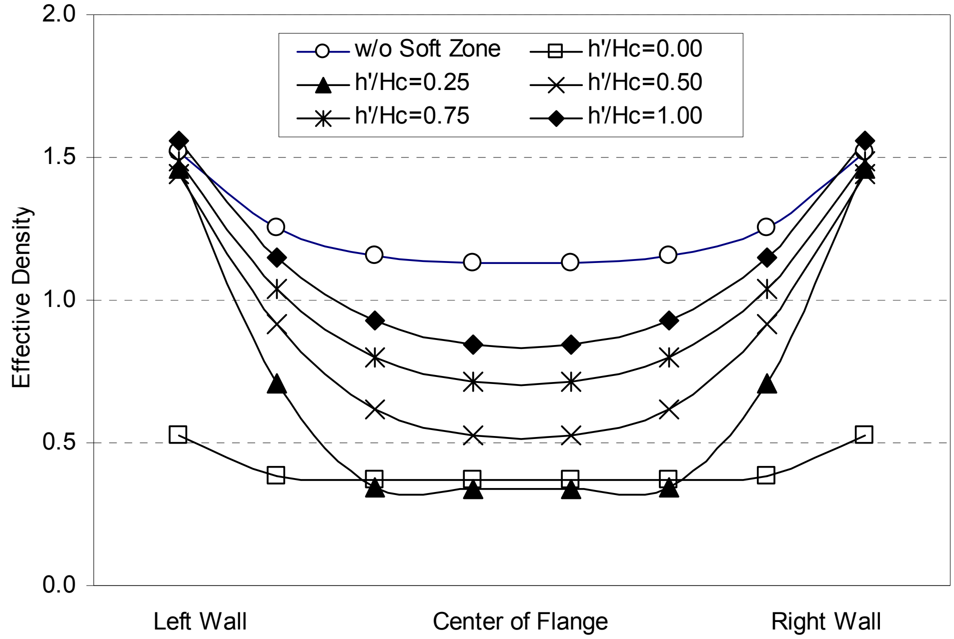


Figure 4.37 Effective Density Distribution acting on the Top Slab (W/B_c=1, different H'/H_c)

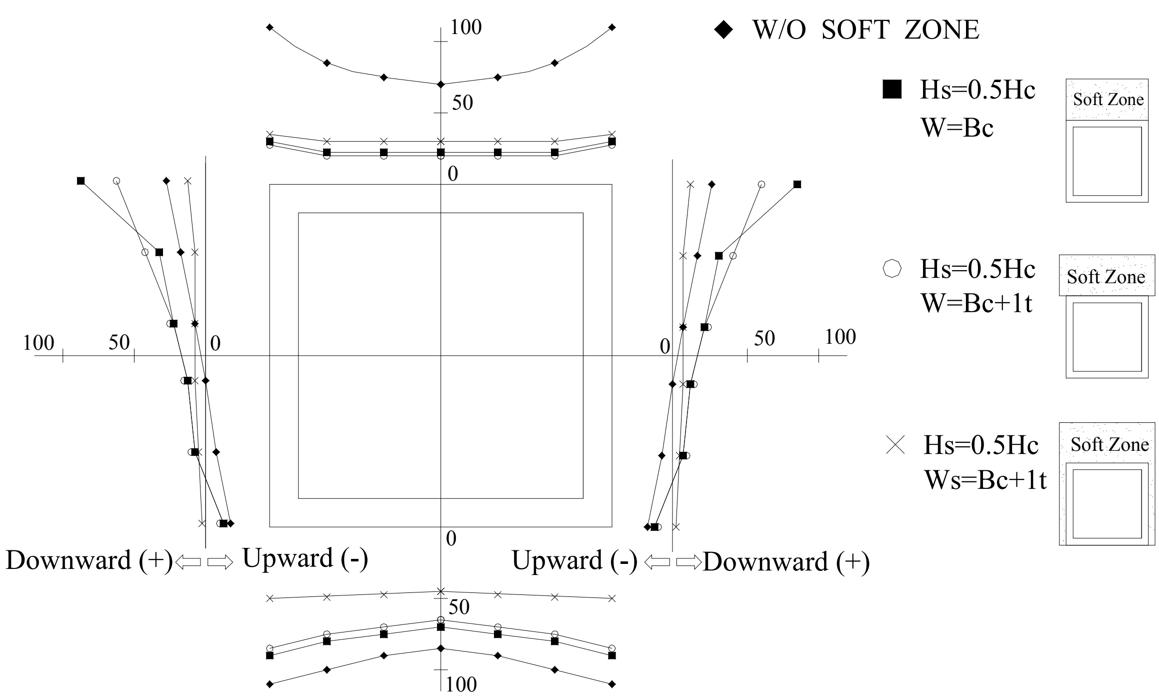


Figure 4.38 Earth Pressure and Shear Force Distribution on Box Culverts

due to Imperfect Trench Installation

4.4.2 Development and Verification of Equations for Reduction Rate of Box Culverts.

As discussed for the equations for the concrete pipe, reduction rates are highly affected by the modulus of elasticity (E_s) of soft materials, which means the reduction rate is a function of E_s. Based on the results of numerous parameter studies with Finite Element programs, the proposed equations for the reduction rates were derived by means of a linear regression method from Figures 4.39 and 4.40 based on the bottom pressure on the culvert. The proposed equations for the condition of the foundation and sidefill compaction are presented in Table 4.7.

To verify the proposed equations, the results from the proposed equations were compared with those from FEA. The relationship between the required fill height and the equivalent fill height was expressed as $H_{equiv} = H_{req}(1 - R)$. The reduction rate, R is given in Table 4.7. The results obtained using the proposed equations and those from the Finite Element Analyses were in good agreement, with less than a 1% difference, as shown in Figure 4.41.

Table 4.7 Proposed Equations for the Reduction Rate of Soft Materials – Box Culverts

Soft Zone	Foundation	Sidefill Compaction	Equations for Reduction Rate
Geometry I	Yielding	TREATED	$R = -0.0003\, E_s + 0.275$
		UNTREATED	$R = -0.0004\, E_s + 0.355$
	Unyielding	TREATED	$R = -0.0004\, E_s + 0.374$
		UNTREATED	$R = -0.0005\, E_s + 0.407$
Geometry II	Yielding	TREATED	$R = 0.7334\, e^{-0.0041 E_s}$
		UNTREATED	$R = 0.6488\, e^{-0.0032 E_s}$
	Unyielding	TREATED	$R = 0.8212\, e^{-0.0037 E_s}$
		UNTREATED	$R = 0.7193\, e^{-0.0030 E_s}$

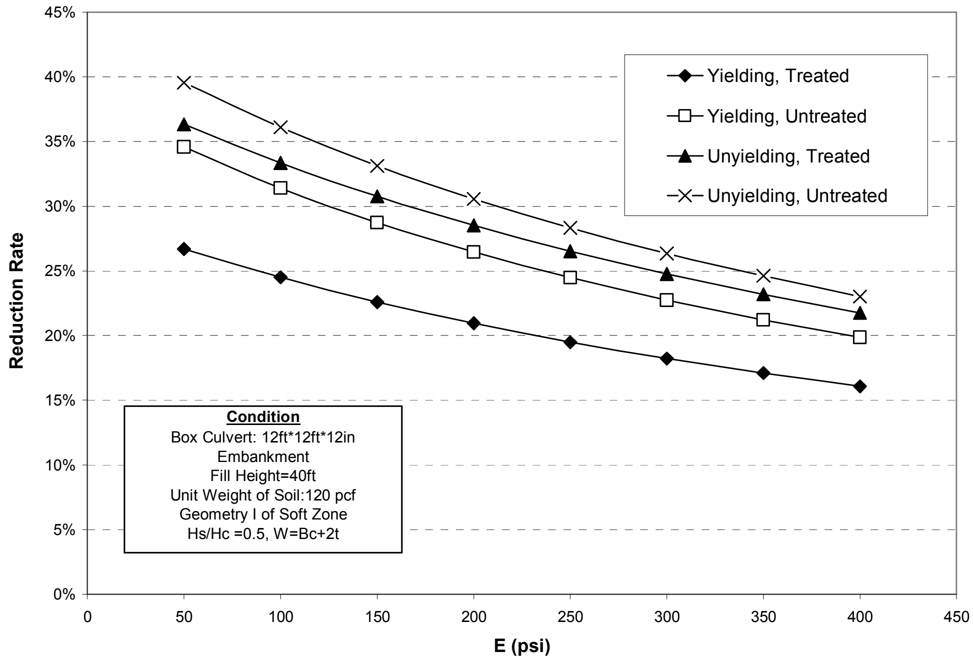


Figure 4.39 Reduction Rate of Geometry I vs. E

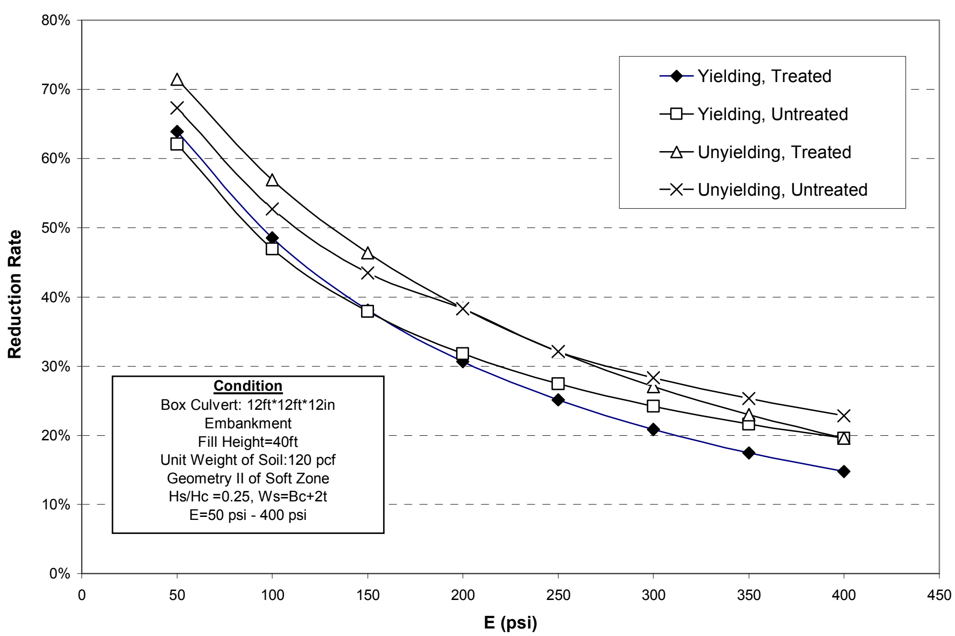


Figure 4.40 Reduction Rate of Geometry II vs. E

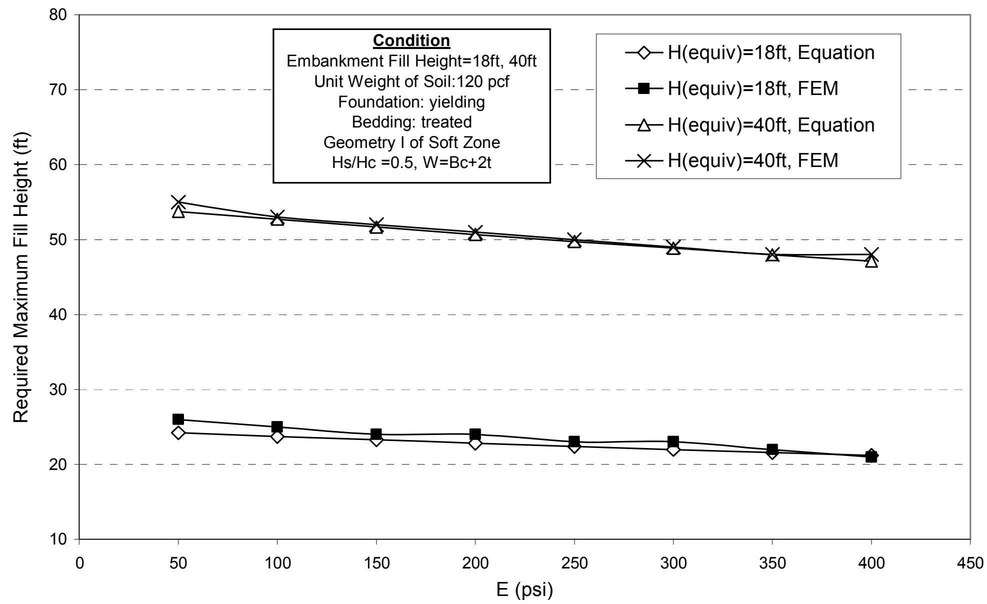
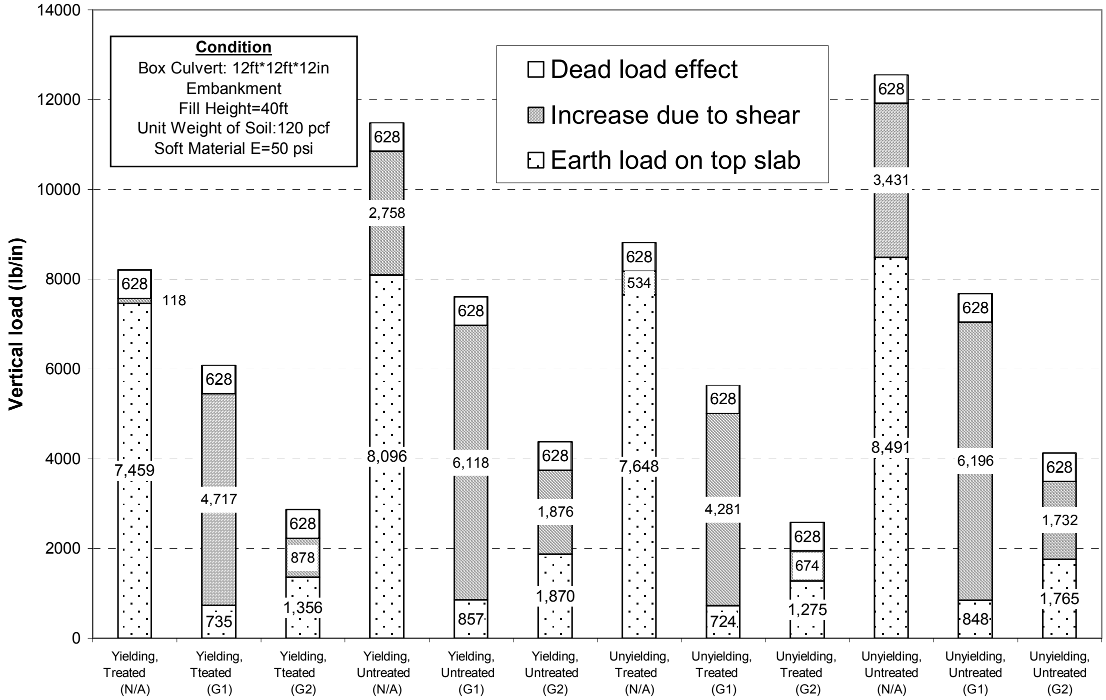


Figure 4.41 Proposed Equation for Reduction Rate vs. FEA (Geometry I, TREATED)

4.5 Summary

All results of the analyses of the bedding and fill heights for box culverts are shown in Figure 4.39 and are summarized as follows:

1) The total vertical earth loads, or bottom loads, acting on the box culverts are composed of the top earth load, dead load, and shear force on the sidewall. Therefore, the design loading of box culverts should be based on the bottom pressure.
2) The earth load on the box culverts is more affected by the sidefill treatment than the foundation.
3) TREATED sidefill is effective in reducing the shear force on the sidefill.
4) Imperfect Trench Installation reduces the top earth pressure and increases the shear force on the sidewall. The total vertical earth load is reduced.
5) Using Geometry II for the soft zone relieves the increased effect of the downward shear force on sidefill due to the reverse arching effect of imperfect trench installation.



Note: 1. N/A, G1 and G2 means embankment installation, Geometry I and Geometry II of imperfect trench installation

Figure 4.42 Vertical Earth Load vs. Embankment Installation and Imperfect Trench Installation (GI, GII)

CHAPTER 5

Special Provisions

5.1 Introduction

5.1.1 General

This chapter contains the Special Provisions for Sections 524, 530, and 850 of ALDOT Standard Specifications, 2002 [67] regarding concrete roadway pipe and box culverts. The main revisions that are included in the Special Provisions are to replace the historical C and C-1 beddings based on the work of Marston and Spangler in 1933 with Proposed Installations of the ALODT project, 930-592 and to provide the design guides for imperfect trench installation. Proposed Installations use two types among the AASHTO Standard Installations developed by ACPA to allow the engineer to take into consideration modern installation techniques.

During the course of this study, it become clear that many provisions in current AASHTO and ASCE Specifications in the materials, design, and construction for concrete pipes and box culverts appear to be disjointed and confusing. Therefore, a summary review on current AASHTO and ASCE provisions for concrete pipes and box culverts for fast reference is provided in Appendix F.

5.1.2 Referenced Documents

The referenced documents of the Special Provisions and a summary review on AASHTO and ASCE Specifications in Appendix F are as follows:

1) AASHTO Standards**:**

- M 170, Standard Specification for Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe
- M 206, Standard Specification for Reinforced Concrete Arch Culvert, Storm Drain, and Sewer Pipe
- M 242, Standard Specification for Reinforced D-Load Culvert, Storm Drain, and Sewer Pipe
- M 207, Standard Specification for Reinforced Concrete Elliptical Culvert, Storm Drain, and Sewer Pipe
- M 262, Standard Specification for Concrete Pipe and Related Products
- M 175, Standard Specification for Perforated Concrete Pipe
- M 240, Standard Specification for Blended Hydraulic Cement

• M 32, Standard Specification for Steel Wire, Plain, for Concrete Reinforcement

• M 225, Standard Specification for Steel Wire, Deformed, for Concrete Reinforcement

• M 221, Standard Specification for Steel Welded Wire Fabric, Deformed, for Concrete Reinforcement

• M 31, Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement

• M 6, Standard Specification for Fine Aggregate for Portland Cement Concrete

• M 80, Standard Specification for Coarse Aggregate for Portland Cement Concrete

• M 259, Standard Specification for Precast Reinforced Concrete Box Sections for Culverts, Storm Drains, and Sewers

• M 273, Standard Specification for Precast Reinforced Concrete Box Sections for Culverts, Storm Drains, and Sewers with Less Than 0.6m of Cover Subjected to Highway Loadings

• M 85, Standard Specification for Portland Cement

• M 55, Standard Specification for Steel Welded Wire Fabric, Plain, for Concrete Reinforcement

• T 280, Standard Specification for Concrete Pipe, Manhole Sections or Tile

• T 99, Standard Specification for Moisture-Density Relations of Soils Using a 5.5 lb Rammer and 12 in. Drop

• Standard Specifications for Highway Bridges, 2002

2) ASTM Standards:

• C76, Standard Specification for Reinforced Concrete Culvert, Storm Drain, and Sewer

• C789, Standard Specification for Precast Reinforced Concrete Box Sections for Culverts, Storm Drains, and Sewers

• C655, Standard Specification for Reinforced Concrete D-Load Culvert, Storm Drain and Sewer Pipe

• C507, Standard Specification for Reinforced Concrete Elliptical Culvert, Storm Drain, and Sewer Pipe

• C507, Standard Specification for Reinforced Concrete Arch Culvert, Storm Drain and Sewer Pipe

3) ALDOT Standard Specifications, 2002:

• Section 214, Structure Excavation and Backfill for Drainage Structures

• Section 524, Reinforced Concrete Box Culverts

- Section 530, Roadway Pipe Culverts
- Section 831, Precast Concrete Products
- Section 846, Pipe Culverts Joints Sealers
- Section 850, Roadway Pipe

4) ASCE Standards:

- Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standard Installation (SIDD)

5) ACPA :

- Concrete Pipe Design Manual
- Concrete Pipe Technology Handbook
- Concrete Pipe Handbook

5.2 Special Provisions

ALDOT 2002	SPECIAL PROVISIONS
	Special Provisions Subject: Excavation and Backfilling for Reinforced Concrete Box Culverts Alabama Standard Specifications for Highway Construction, 2002 Edition Shall be amended as follows:
SECTION 524 Reinforced Concrete Box Culverts	SECTION 524 Reinforced Concrete Box Culverts
524.03 Construction Requirements (a) Excavation and backfilling	524.03 Construction Requirements (a) Excavation and backfilling ***1. Conventional***
Excavation and backfilling shall be in accordance with the provisions of Section 214. In addition, for precast culverts the foundation requirements of Section 214 shall be modified to require a bedding of at least a 4 inch {100mm} compacted layer of foundation backfill placed between graded forms set 1 foot {300mm} outside each outside wall of the box culvert. The foundation backfill shall be fine graded off the	Excavation and backfilling shall be in accordance with the provisions of Section 214. In addition, for precast culverts the foundation requirements of Section 214 shall be modified to require a bedding of at least a ***3 inch {75mm}*** compacted layer of foundation backfill placed between graded forms set 1 foot {300mm} outside each outside wall of the box culvert. The foundation backfill shall be fine graded off the

ALDOT 2002	SPECIAL PROVISIONS
forms, compacted as directed by the Engineer, and shaped to fit the bottom of the precast section. After placement of the sections, the forms may be removed.	forms, compacted as directed by the Engineer, and shaped to fit the bottom of the precast section. After placement of the sections, the forms may be removed. ***For embankment installation with compacted fill (TREATED) on the sides of the box culvert, the minimum horizontal compaction distance of the fill on each side shall be constructed to at least one time of out-to-out horizontal span of box culverts in each direction from the outside edge of the conduit. The vertical height of side fill compaction shall be constructed to the middle level of box culverts from foundation. The compaction of compacted area shall conform to the requirements of the Special Highway Drawings in the ALDOT project, 930-592.*** ***For embankment installation with uncompacted fill (UNTREATED) on the sides of the box section, backfilling shall be in accordance with the provisions of Section 214.*** ***For trench installation, the trench shall be excavated as specified in Section 214. Clearance between box culvert and trench wall shall be adequate to enable***

specific compaction as specified in the Special Highway Drawings of the ALDOT project, 930-592. Caution shall be used to keep the sides of the trench vertical and to specified dimension. However, if the excavation is greater than ten feet deep, OSHA requirements mandate sloping sidewalls or use of a trench box for worker safety. Extra wide excavation to accommodate pans or other unsuitable excavating equipment will not be permitted.

2. Imperfect Trench

Excavation and backfilling except soft zone shall be in accordance with the provisions of Section 214. Geometry of soft zone shall conform to the requirements of the Special Highway Drawings of the ALDOT project, 930-592. The roadway conduits shall be placed, backfilled as specified in Subarticle 524.03(b). For Geometry of soft zone, the fill shall be constructed to a quarter of out-to-out vertical rise of box above the top of the conduit. The minimum distance of the fill on each side shall be constructed to at least 5 times of out-to-out horizontal span of the pipe or box in each direction from the outside edge of the

conduit. A trench equal in width to the out-to-out horizontal span plus 2 times of wall thickness of the conduit shall be dug in the fill directly down to the bottom for box culverts. Care shall be exercised to keep the sides of this trench as nearly vertical as possible.

The trenches shall then be refilled with soft materials like EPS (Geofoam), woodwaste, sawdust, woodchips, tirechips, and hay. After this loose backfill is completed, the remainder of the fill up to subgrade elevation shall be constructed as specified in the provisions of Section 214.

ALDOT 2002	SPECIAL PROVISIONS
	Special Provisions Subject: Excavation, bedding and Backfilling for Roadway Pipe Culverts Alabama Standard Specifications for Highway Construction, 2002 Edition Shall be amended as follows:
SECTION 530 Roadway Pipe Culverts	SECTION 530 Roadway Pipe Culverts
530.03 Construction Requirements (b) Excavation of Trench, Details of trenching and bedding of pipe will be shown on the plans. All pipe 48 inches {1200mm} or less in horizontal diameter shall be laid in a trench extending at least 1 foot {300mm} above the elevation of the top of the pipe. For such pipe, where the ground surface is less than 1 foot {300mm} above the elevation of the top of the pipe, the Contractor shall first construct and compact the fill to a minimum height of 1 foot {300mm} above the elevation of the top of the pipe and for a minimum distance of 10 feet {3m} in each direction from the outside edge of the	530.03 Construction Requirements (b) Excavation of Trench, Details of trenching and bedding of pipe will be shown on the plans. All pipe 48 inches {1200mm} or less in horizontal diameter shall be laid in a trench extending at least 1 foot {300mm} above the elevation of the top of the pipe. For such pipe, where the ground surface is less than 1 foot {300mm} above the elevation of the top of the pipe, the Contractor shall first construct and compact the fill to a minimum height of 1 foot {300mm} above the elevation of the top of the pipe and for a minimum distance of 10 feet {3m} in each direction from the outside edge of the

ALDOT 2002	SPECIAL PROVISIONS
pipe. The trench shall then be excavated as specified in Section 214. Caution shall be used to keep the sides of the trench vertical and to specified dimensions. Extra wide excavation to accommodate pans or other unsuitable excavating equipment will not be permitted. Excavation above subgrade will be classified and paid for as roadway excavation. Excavation below subgrade will be classified and paid for as structure excavation except that no payment will be made for excavating that part of a fill section placed more than 1 foot {300 mm} above the top of the pipe.	pipe. The trench shall then be excavated as specified in Section 214. Caution shall be used to keep the sides of the trench vertical and to specified dimensions. Extra wide excavation to accommodate pans or other unsuitable excavating equipment will not be permitted. Excavation above subgrade will be classified and paid for as roadway excavation. Excavation below subgrade will be classified and paid for as structure excavation except that no payment will be made for excavating that part of a fill section placed more than 1 foot {300 mm} above the top of the pipe.
For pipe over 48 inches {1200 mm} in horizontal diameter, trenching will be required only where the original ground is above the elevation of the bottom of the pipe, and backfilling shall be performed as specified in Item 210.03(d)2.	For pipe over 48 inches {1200 mm} in horizontal diameter, trenching will be required only where the original ground is above the elevation of the bottom of the pipe, and backfilling shall be performed as specified in Item 210.03(d)2.
Should the material encountered at the elevation of the trench floor not be suitable to support the structure, removal of unsuitable material and placement of foundation backfill shall be performed and will be paid for as specified in Section 214. Temporary drainage necessary for proper installations shall be provided by the Contractor without additional compensation.	Should the material encountered at the elevation of the trench floor not be suitable to support the structure, removal of unsuitable material and placement of foundation backfill shall be performed and will be paid for as specified in Section 214. Temporary drainage necessary for proper installations shall be provided by the Contractor without additional compensation.

ALDOT 2002	SPECIAL PROVISIONS
(c) Pipe Bedding	(c) Pipe Bedding
1. General	1. General
All pipe culverts placed under this Section shall be placed in a prepared bed of one of the types noted herein, Unless otherwise provided, a Class “C” Bedding shall be used.	All pipe culverts placed under this Section shall be placed in a prepared bed of one of the types noted herein, Unless otherwise provided, a Class “C” Bedding shall be used.
2. Class A Bedding	2. Class A Bedding
The pipe culvert shall be bedded in a continuous concrete cradle conforming to plan details.	The pipe culvert shall be bedded in a continuous concrete cradle conforming to plan details.
3. Class B bedding	3. Class B bedding
The pipe shall be bedded with ordinary care in a prepared foundation bed to a depth of not less than 30 percent of the vertical diameter of the pipe plus 4 inches {100mm}. The thickness of the foundation bed shall be a minimum of 4 inches {100mm} in thickness and shall be shaped to fit the pipe for at least 15 percent of the vertical outside diameter. Recesses in the trench bottom shall be shaped to accommodate the bell of the pipe when bell and spigot type pipe is used. “Ordinary” care in this Article shall mean sufficient care to insure that the permissible variations listed in Item 530.03(a) 2 will not be	The pipe shall be bedded with ordinary care in a prepared foundation bed to a depth of not less than 30 percent of the vertical diameter of the pipe plus 4 inches {100mm}. The thickness of the foundation bed shall be a minimum of 4 inches {100mm} in thickness and shall be shaped to fit the pipe for at least 15 percent of the vertical outside diameter. Recesses in the trench bottom shall be shaped to accommodate the bell of the pipe when bell and spigot type pipe is used. “Ordinary” care in this Article shall mean sufficient care to insure that the permissible variations listed in Item 530.03(a) 2 will not be

ALDOT 2002	SPECIAL PROVISIONS
exceeded. The bedding material shall be sand or an approved selected sandy soil.	exceeded. The bedding material shall be sand or an approved selected sandy soil.
4. Class C bedding	4. ***UNTREATED installation*** (formerly Class "C" bedding)
The pipe shall be bedded with ordinary care in a loosened soil foundation shaped to fit the lower part of the pipe exterior with reasonable closeness for at least 10 percent of its overall height. Use of a template for shaping will not be required. The shaped foundation shall be loosened by pulverizing the soil to a minimum depth equal to 0.125 times the diameter of the pipe or 3 inches {75mm} maximum. "Ordinary care" in this Article shall mean sufficient care to insure that the permissible variations listed in Item 530.03(a) 2 will not be exceeded.	The pipe shall be bedded with ordinary care in a loosened soil foundation shaped to fit the lower part of the pipe exterior with reasonable closeness for at least 10 percent of its overall height. Use of a template for shaping will not be required. The shaped foundation shall be loosened by pulverizing the soil to a minimum depth equal to 0.125 times the diameter of the pipe or 3 inches {75mm} maximum. "Ordinary care" in this Article shall mean sufficient care to insure that the permissible variations listed in Item 530.03(a) 2 will not be exceeded.
Where ledge rock, rocky or gravelly soil, hard pan, or other unyielding foundation material is encountered at a culvert site, the pipe shall be bedded as follows: The hard unyielding material shall be excavated below the elevation of the bottom of the pipe, or pipe bell, for a depth of at least 12inches {300mm}, or ½ inch for each foot {40mm for each meter} of fill over the top of the pipe, whichever is greater, but not more than 24inches {600mm}. Payment for this material	Where ledge rock, rocky or gravelly soil, hard pan, or other unyielding foundation material is encountered at a culvert site, the pipe shall be bedded as follows: The hard unyielding material shall be excavated below the elevation of the bottom of the pipe, or pipe bell, for a depth of at least 12inches {300mm}, or ½ inch for each foot {40mm for each meter} of fill over the top of the pipe, whichever is greater, but not more than 24inches {600mm}. Payment for this material

ALDOT 2002	SPECIAL PROVISIONS
shall be made under structure excavation. The width of the excavation shall be 12 inches {300mm} greater than the outside diameter or span of the pipe and shall be filled with selected fine compressible material, such as silty clay or loam taken from selected grading operations or areas beyond the right of way and paid for as foundation backfill. This material shall then be lightly compacted in 6 inches {150mm} compacted lifts and shaped as specified above.	shall be made under structure excavation. The width of the excavation shall be 12 inches {300mm} greater than the outside diameter or span of the pipe and shall be filled with selected fine compressible material, such as silty clay or loam taken from selected grading operations or areas beyond the right of way and paid for as foundation backfill. This material shall then be lightly compacted in 6 inches {150mm} compacted lifts and shaped as specified above. ***5. TREATED installation*** ***The pipe shall be bedded as specified in the Special Highway Drawings of ALDOT project, 930-592. The bedding thickness shall be a minimum of 3 inches in thickness. The bedding thickness of the pipe shall be a minimum of 1/12 of out-to-out horizontal span or not less than 6 inches under rock foundation to avoid placing the pipe directly on hard or variable subgrade. The middle of bedding shall be placed in a loosened soil foundation shaped to fit the lower part of the pipe exterior with reasonable closeness. Use of a template for shaping will not be required.*** ***The side fill shall be constructed to springline of pipe from foundation. The minimum***

ALDOT 2002	SPECIAL PROVISIONS
	horizontal distance of each side fill shall be constructed to at least 1 time of out-to-out horizontal span of the pipe in each direction from the outside edge of the pipe. Soils and minimum compaction requirements of each area of side fill shall conform to the Special Highway Drawings of ALDOT project, 930-592.
	The TREATED installation permits the use of soils in the haunch and bedding zones having easily attained compaction requirement, justifying less stringent inspection requirements with granular and some native soils. Silty clays may be used in the haunch zone if adequately compacted as specified in the Special Highway Drawings of ALDOT project, 930-592.
6. Class C-1 bedding	***6. Imperfect Trench***
When so specified on the plans, Class C-1 bedding or imperfect trench method shall be used as follows:	***Refer to 530.03 (e) 3***
The pipe shall be placed and backfilled as specified in Subarticles 530.03(d) and (e) to a	

ALDOT 2002	SPECIAL PROVISIONS
point 1 foot {300mm} above the top of the pipe. The fill shall then be continued as specified in Section 210 for a minimum distance of 10 feet {3m} in each direction from the outside edge of the pipe and to a height equal to outside diameter of the pipe plus 1 foot {300mm} above the top of the pipe. Next, a trench equal in width to the outside diameter of the pipe shall be dug in the fill directly over the culvert down to an elevation 1 foot {300mm} above the top of the pipe. Care shall be exercised to keep the sides of this trench as nearly vertical as possible. The trenches shall then be refilled with loose, highly compressible soil, except that straw, hay, cornstalks, leaves, brush, or sawdust may be used to fill the lower ¼ to 1/3 of the trench. After this loose backfill is completed, the remainder of the fill up to subgrade elevation shall be constructed as specified in Section 210. Compensation for the extra excavation and backfill involved in the imperfect trench method shall be included in the unit price of other items and no direct payment will be made for this work. At the contractor's option, the embankment may be constructed full height prior to laying the pipe.	

ALDOT 2002	SPECIAL PROVISIONS
(e) Backfilling Pipe	(e) Backfilling Pipe
1. General	1. General
After the pipe has been installed, the pipe trench shall be backfilled with the best of the suitable material excavated from the trench; if none of this excavated material is suitable, material from the trench may be used unless unsuitable for embankment.	After the pipe has been installed, the pipe trench shall be backfilled with the best of the suitable material excavated from the trench; if none of this excavated material is suitable, material from the trench may be used unless unsuitable for embankment.
Backfilling will not be permitted until authorized by the Engineer. When mortar joints are used, backfilling shall not begin until the joints have cured or until authorized by the Engineer.	Backfilling will not be permitted until authorized by the Engineer. When mortar joints are used, backfilling shall not begin until the joints have cured or until authorized by the Engineer.
2. Placing and compaction of backfill	2. Placing and compaction of backfill
The backfill material shall be compacted at near optimum moisture content, in layers not exceeding 6 inches {150mm} compacted thickness, to a density of not less than 95 percent of AASHTO T99 density by methods detailed in Section 210. Mechanical tampers shall be used unless another method of compaction is approved in writing; inundation or jetting will not be permitted unless specified on the plans. Care shall be exercised to thoroughly compact the backfill under the haunches of the pipe and to insure that the material is in intimate contact with	The backfill material shall be compacted at near optimum moisture content, in layers not exceeding 6 inches {150mm} compacted thickness, to a density of not less than 95 percent of AASHTO T99 density by methods detailed in Section 210. Mechanical tampers shall be used unless another method of compaction is approved in writing; inundation or jetting will not be permitted unless specified on the plans. Care shall be exercised to thoroughly compact the backfill under the haunches of the pipe and to insure that the material is in intimate contact with

ALDOT 2002	SPECIAL PROVISIONS
the pipe. The backfill shall be brought up evenly in layers on both sides of the pipe for its full length until the trench is filled or up to subgrade elevation if the trench is in cut.	the pipe. The backfill shall be brought up evenly in layers on both sides of the pipe for its full length until the trench is filled or up to subgrade elevation if the trench is in cut.
When the top of the pipe is exposed above the top of the trench, embankment material shall be placed and compacted for a width on each side of the pipe equal to at least twice the horizontal inside diameter of the pipe, or 12 feet {4m} whichever is less. The embankment on each side of the pipe, for a distance equal to the horizontal inside diameter of the pipe, shall be of the same material and compacted in a normal manner except where the imperfect trench method is prescribed. All pipe after being bedded and backfilled as specified in this Section, should be protected by a 3 foot{0.6m} cover of fill before heavy equipment is permitted to cross during construction of the roadway.	When the top of the pipe is exposed above the top of the trench, embankment material shall be placed and compacted for a width on each side of the pipe equal to at least twice the horizontal inside diameter of the pipe, or 12 feet {4m} whichever is less. The embankment on each side of the pipe, for a distance equal to the horizontal inside diameter of the pipe, shall be of the same material and compacted in a normal manner except where the imperfect trench method is prescribed. All pipe after being bedded and backfilled as specified in this Section, should be protected by a 3 foot{0.6m} cover of fill before heavy equipment is permitted to cross during construction of the roadway.
	3. Imperfect Trench
	Excavation and backfilling except soft zone shall be in accordance with the provisions of Section 214. Geometry of soft zone shall conform to the Special Highway Drawings of ALDOT project, 930-592. The roadway conduits shall be placed, backfilled as

ALDOT 2002	SPECIAL PROVISIONS
	specified in Subarticle 524.03(b). ***For Geometry of soft zone, the fill shall be constructed to a quarter of out-to-out vertical rise of pipe above the top of the conduit. The minimum distance of the fill on each side shall be constructed to at least 5 times of out-to-out horizontal span of the pipe or box in each direction from the outside edge of the conduit.*** ***Next, a trench equal in width to the out-to-out horizontal span plus 1 time of wall thickness (or minimum the out-to-out horizontal span plus 6 inches) in case of the conduit shall be dug in the fill directly down to the springline of concrete roadway pipe. Care shall be exercised to keep the sides of this trench as nearly vertical as possible.*** ***The trenches shall then be refilled with soft materials like EPS (Geofoam), woodwaste, sawdust, woodchips, tirechips, and hay. After this loose backfill is completed, the remainder of the fill up to subgrade elevation shall be constructed as specified in the provisions of Section 214.***

3. Protection of pipe

The contractor shall exercise necessary care in installing and backfilling pipe, and it shall be his responsibility to see that the pipe is not damaged by lateral forces during backfilling, by heavy loads operating over the pipe, or by other causes. All damaged pipe shall be replaced or repaired by the Contractor at his own expense at the option of, and to the satisfaction of, the Engineer.

Any pipe not true to designated alignment and grade within specified tolerances, or any pipe that shows settlement due to faulty installation, shall be relaid or replaced by the Contractor repaired by the Contractor, at the option of the Engineer, without additional compensation. All pipe lines shall be thoroughly cleaned out prior to final acceptance.

4. Protection of pipe

The contractor shall exercise necessary care in installing and backfilling pipe, and it shall be his responsibility to see that the pipe is not damaged by lateral forces during backfilling, by heavy loads operating over the pipe, or by other causes. All damaged pipe shall be replaced or repaired by the Contractor at his own expense at the option of, and to the satisfaction of, the Engineer.

Any pipe not true to designated alignment and grade within specified tolerances, or any pipe that shows settlement due to faulty installation, shall be relaid or replaced by the Contractor repaired by the Contractor, at the option of the Engineer, without additional compensation. All pipe lines shall be thoroughly cleaned out prior to final acceptance.

ALDOT 2002	SPECIAL PROVISIONS
	Special Provisions Subject: Materials for Roadway Pipe Alabama Standard Specifications for Highway Construction, 2002 Edition Shall be amended as follows:
SECTION 850 Roadway Pipe	SECTION 850 Roadway Pipe
850.01 Concrete Pipe (a) General Concrete pipe shall be reinforced circular or reinforced arch concrete pipe. Circular concrete pipe shall comply with the requirements of AASHTO M 170, except that elliptical steel reinforcement will not be permitted unless such is permitted for special design pipe by details provided in the plans. Concrete arch pipe shall comply with the requirements of AASHTO M 206.	850.01 Concrete Pipe (a) General Concrete pipe shall be reinforced circular, reinforced arch, or ***reinforced elliptical*** concrete pipe. Circular concrete pipe shall comply with the requirements of AASHTO M 170, except that elliptical steel reinforcement will not be permitted unless such is permitted for special design pipe by details provided in the plans. Concrete arch pipe and elliptical pipe shall comply with the requirements of AASHTO M 206 and ***AASHTO M 207.***
(b) Special design When so permitted by the plans or in the	(b) Special design When so permitted by the plans or in the

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proposal, pipe of designs other than those shown in the standard plans may be permitted; however, such pipe must meet performance and test requirements specified in AASHTO M 170 and shall be installed under the same specifications as circular pipe.

(c) Classes of pipe

Circular pipe and arch pipe shall be of the following classes, corresponding to AASHTO M170 or AASHTO M 206 classes as tabulated herein.

AASHTO Class	ALDOT Class	Abbreviation
Class II	Class 2 Reinf. Conc. Pipe	CL.2 R.C.Pipe
Class III	Class 3 Reinf. Conc. Pipe	CL.3 R.C.Pipe
Class IV	Class 4 Reinf. Conc. Pipe	CL.4 R.C.Pipe
Class V	Class 5 Reinf. Conc. Pipe	CL.5 R.C.Pipe
Class II	Class 2 Reinf. Conc. Arch Pipe	CL.2R.C. Arch Pipe
Class III	Class 3 Reinf. Conc.Arch Pipe	CL.3 R.C. Arch Pipe
Class IV	Class 4 Reinf. Conc.Arch Pipe	CL.4 R.C. Arch Pipe

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proposal, pipe of designs other than those shown in the standard plans may be permitted; however, such pipe must meet performance and test requirements specified in AASHTO M 170, ***M 206, or M 207*** and shall be installed under the same specifications as circular pipe, arch pipe, or elliptical pipe.

(c) Classes of pipe

Circular pipe and arch pipe shall be of the following classes, corresponding to AASHTO M170 or AASHTO M 206 classes as tabulated herein.

AASHTO Class	ALDOT Class	Abbreviation
Class II	Class 2 Reinf. Conc. Pipe	CL.2 R.C.Pipe
Class III	Class 3 Reinf. Conc. Pipe	CL.3 R.C.Pipe
Class IV	Class 4 Reinf. Conc. Pipe	CL.4 R.C.Pipe
Class V	Class 5 Reinf. Conc. Pipe	CL.5 R.C.Pipe
Class II	Class 2 Reinf. Conc. Arch Pipe	CL.2R.C. Arch Pipe
Class III	Class 3 Reinf. Conc.Arch Pipe	CL.3 R.C. Arch Pipe
Class IV	Class 4 Reinf. Conc.Arch Pipe	CL.4 R.C. Arch Pipe

ALDOT 2002	SPECIAL PROVISIONS
	Concrete elliptical Pipe shall be of classes, corresponding to AASHTO M 207.
(d) Materials	(d) Materials
Coarse aggregate, fine aggregate, cement, steel reinforcement, and water shall meet the requirements of AASHTO M 170 or M 206, whichever is applicable, except as modified in applicable Sections of Division 800, Materials.	Coarse aggregate, fine aggregate, cement, steel reinforcement, and water shall meet the requirements of AASHTO M 170, M 206 ***or M 207***, whichever is applicable, except as modified in applicable Sections of Division 800, Materials.
(e) Acceptance	(e) Acceptance
All precast products furnished shall meet the requirements of Section 831.	All precast products furnished shall meet the requirements of Section 831.
(f) Handling and storage	(f) Handling and storage
Pipe shall be handled, transported, delivered, and stored in a manner that will not injure or damage the pipe. Pipe shall not be shipped before it has been inspected and approved. Pipe that is damaged during shipment or handling will be rejected even though satisfactory before shipment. Pipe dropped from platforms or vehicles or in the pipe trench will be rejected.	Pipe shall be handled, transported, delivered, and stored in a manner that will not injure or damage the pipe. Pipe shall not be shipped before it has been inspected and approved. Pipe that is damaged during shipment or handling will be rejected even though satisfactory before shipment. Pipe dropped from platforms or vehicles or in the pipe trench will be rejected.

CHAPTER 6

Design Guides and Examples

6.1 General

These design guides cover the indirect design of reinforced concrete pipes and box culverts intended to be used for the conveyance of sewage, industrial wastes, and storm water, and for the construction of culverts. Most of these design guides and examples are developed for positive projection embankment conditions, which are the worst case vertical load conditions for rigid conduits, and which provide conservative results for other embankment and trench conditions. Design guides and examples for trench conditions and negative projection embankment conditions are also given in this chapter.

6.2 Design Methods

6.2.1 Direct Design vs. Indirect Design

Traditionally, there are two methods of structural design that have been used for designing buried concrete pipe. These are the indirect design method and the direct design method. While the direct design procedures have been used for over 40 years and have long been included in detail in Section 16 of the AASHTO Standard Specifications for Highway Bridges, most engineers and designers are more familiar with the indirect method when specifying concrete pipe. By definition, direct design is designing specifically for the anticipated loads in the field and the resulting moments, thrust and shear caused by such loadings. Indirect design (the D-Load concept) is designing for a concentrated test load that is determined by the relationship between the field calculated moment and the test moment for the same load. This relationship is called a bedding factor.

The direct design procedure, in the past, applied the forces acting on the pipe using the "Paris" or "Olander" force distribution scheme. In recent years, however, based on a 20-year in-depth study of pipe-soil interaction, a modified soil pressure distribution has been developed that is a function of soil type and compaction. These soil pressure configurations are called the Heger distribution, and are referred to as Type I through Type IV, depending on the pipe bedding, soil type, and compaction level. All four types have been incorporated into ASCE, AASHTO and ACPA standards. A significant point relative to the

Heger distribution is that the difficulty in obtaining specified soil compaction under the haunches of the pipe has been recognized in the soil pressure distribution by conservatively assuming all installations will have voids and soft inclusion in the haunch area.

The Direct Design Procedure is as follows;

1) Establish the pipe diameter, wall thickness, and sidefill type.
2) Select the Proposed Installation to be used among TREATED or UNTREATED installation.
3) Determine the vertical earth and live load forces acting on the pipe.
4) For the type of installation selected, determine the moments, thrusts and shears due to the applied loads. For each type of installation, design coefficients have been developed for the determination of the critical moments, thrusts and shears. These coefficients are presented in the Concrete Pipe Technology Handbook [39], published by the American Concrete Pipe Association.
5) The structural design of the pipe is performed using established reinforced concrete design principles and will include five performance modes:

   - Flexural
   - Diagonal tension
   - Radial tension
   - Concrete compression
   - Service load crack control

The Indirect Design Procedure is as follows;

1) Establish the pipe diameter, wall thickness, and sidefill type.
2) Select the Proposed Installation to be used among TREATED or UNTREATED installation.
3) Determine the vertical earth load and live load forces acting on the pipe.
4) Select the earth load and live load bedding factors for the selected installation, taking into account that the live load bedding factor cannot be greater than the earth load bedding factor. These bedding factors are presented in the ACPA publication Design Data 40 [56] and in AASHTO Standard Specifications for Highway Bridges, 2002 [6].

5) Apply a factor of safety.

6) Divide the earth load and live load by their respective bedding factors and by the pipe diameter to determine the required D-Load strength. This D-Load is the service load condition.

In comparing Indirect Design with Direct Design, one recognizes in the 3-edge bearing test that the maximum moment and shear is at the same location, which is not the case in the field. Also, in view of the concentrated load and reaction that exists in the indirect (D-Load) design test, failure modes can exist that are not typical for the Direct Design pipe, and often require special steel reinforcing assemblies that are actually unnecessary in the field. While either of these methods can be used with reliability, the direct design method is best suited for the larger diameters of pipe and high load installations.

6.2.2 Indirect Design for Concrete Roadway Pipe

6.2.2.1 Determination of Earth Load

1) Positive Projection Embankment Soil Load

Concrete pipe can be installed in either an embankment or trench condition, as discussed previously. The type of installation has a significant effect on the loads carried by the rigid pipe. Although narrow trench installations are most typical, there are many cases where the pipe is installed in a positive projecting embankment condition, or in a trench with a width significant enough that it should be considered a positive projecting embankment condition. In this condition, the soil alongside the pipe will settle more than the soil above the rigid pipe structure, thereby imposing an additional load on the prism of soil directly above the pipe. With Proposed Installations, this additional load is accounted for by using a Vertical Arching Factor. This factor is multiplied by the prism load (weight of soil directly above the pipe) shown in Equation 6-1 to give the total load of soil on the pipe. Unlike the previous design method used for the Marston and Spangler beddings, there is no need to assume a projection or settlement ratio.

Prism Load

$$PL = w\left[H + \frac{D_0(4-\pi)}{8}\right]D_0 \qquad \text{(6-1a)}$$

$$VAF = \frac{W_E}{PL} \tag{6-1b}$$

$$HAF = \frac{W_h}{PL} \tag{6-1c}$$

where,

w = soil unit weight, lbs/ft^3

H = Height of fill, ft

D = inside diameter, ft

Vertical Arching Factor (VAF)

VAF=1.40 for TREATED

VAF=1.45 for UNTREATED

Horizontal Arching Factor (HAF)

HAF=0.37 for TREATED

HAF=0.30 for UNTREATED

W_E =Total vertical earth load, lb/ft

W_H = Total horizontal earth load, lb/ft

2) Negative Projection Embankment Soil Load

The fill load on a pipe installed in a negative projecting embankment condition is computed by the equation:

$$W_d = C_n w B_d^2 \tag{6-2}$$

where,

C_n = Load Coefficient

B_d=Width of trench, ft

The load coefficient C_n is further defined as:

$$C_n = \frac{e^{-2K\mu' \frac{H}{B_d}} - 1}{-2K\mu'} \quad \text{when } H \le H_e \tag{6-3}$$

$$C_n = \frac{e^{-2K\mu'\frac{H_e}{B_d}} - 1}{-2K\mu'} + \left(\frac{H}{B_d} - \frac{H_e}{B_d}\right)e^{-2K\mu'\frac{H_e}{B_d}} \text{ when } H>H_e \tag{6-4}$$

where,

B_d=width of trench, ft

K=ratio of active lateral unit pressure to vertical unit pressure

$\mu' = tan\,\phi'$, coefficient of friction between fill material and sides of trench

The settlements, which influence the loads on negative projecting embankment installations, are shown in Figure 6.1. It is necessary to define the settlement ratio for these installations. Equating the deflection of the pipe and the total settlement of the prism of fill above the pipe to the settlement of the adjacent soil:

$$r_{sd} = \frac{S_g - \left(S_d + S_f + d_c\right)}{S_d} \tag{6-5}$$

Recommended settlement ratio design values are listed in Table 6.1.

3) Trench Soil Load

The backfill load on pipe installed in a trench condition is computed by the equation:

$$W_d = C_d w B_d^2 + \frac{D_o^2\left(4-\pi\right)}{8} w \tag{6-6}$$

C_d is further defined as:

$$C_d = \frac{1 - e^{-2K\mu'\frac{H}{B_d}}}{2K\mu'} \tag{6-7}$$

4) Jacked or Tunneled Soil Load

This type of installation is used where surface conditions make it difficult to install the pipe by conventional open excavation and backfill methods, or where it is necessary to install the pipe under an existing embankment. The earth load on a pipe installed by these methods is computed by the equation:

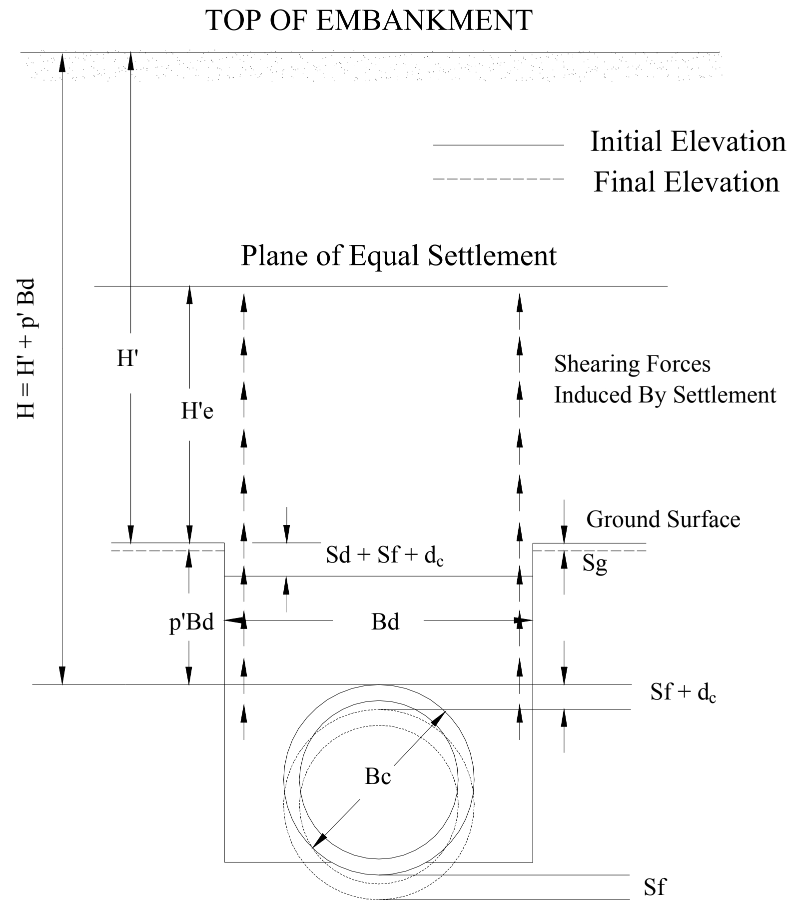

Figure 6.1 Settlements Which Influence Loads on Negative Projection Embankment Installation

$$W_t = C_t w B_t^2 - 2cC_t B_t \tag{6-8}$$

where,

B_t=width of tunnel bore, ft

The load coefficient C is defined as

$$C = \frac{1 - e^{-2K\mu' \frac{H}{B_t}}}{-2K\mu'} \tag{6-9}$$

Table 6.1 Design Values for the Settlement Ratio

Installation and Foundation Condition	Settlement Ratio r_{sd}	
	Usual Range	Design Value
Positive Projecting	0.0 to +1.0	
Rock or Unyielding Soil	+1.0	+1.0
Ordinary Soil	+0.5 to +0.8	+0.7
Yielding Soil	0.0 to +0.5	+0.3
Zero Projecting		0.0
Negative Projecting	-1.0 to 0.0	
$P' = 0.5$		-0.1
$P' = 1.0$		-0.3
$P' = 1.5$		-0.5
$P' = 2.0$		-1.0

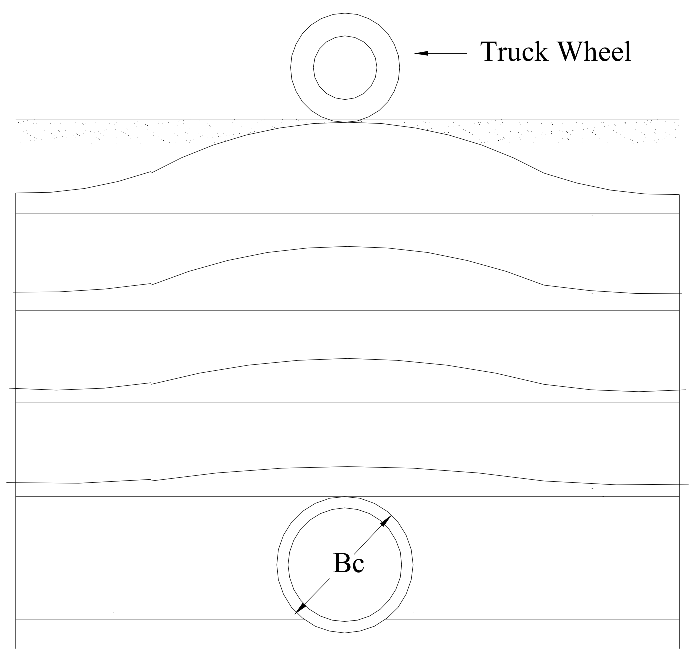

Figure 6.2 Live Load Distribution

6.2.2.2 Determination of Live Load

In the selection of pipe, it is necessary to evaluate the effect of live loads. Live load considerations are necessary in the design of pipe installed with shallow cover under railroads, airports and unsurfaced highways. The distribution of a live load at the surface on any horizontal plane in the subsoil is shown in Figure 6.2. In typical concrete pipe design, the governing moments and shears in the pipe are at the invert. However, in extremely shallow installations, the governing moments and shears may occur in the crown of the pipe as a result of the concentrated live load.

1) Highways

In the case of flexible pavements designed for light duty traffic but subjected to heavy truck traffic, the live load transmitted to the pipe must be considered. In analyses, the most critical AASHTO loadings shown in Figure 6.3 are used in either the single mode or passing mode.

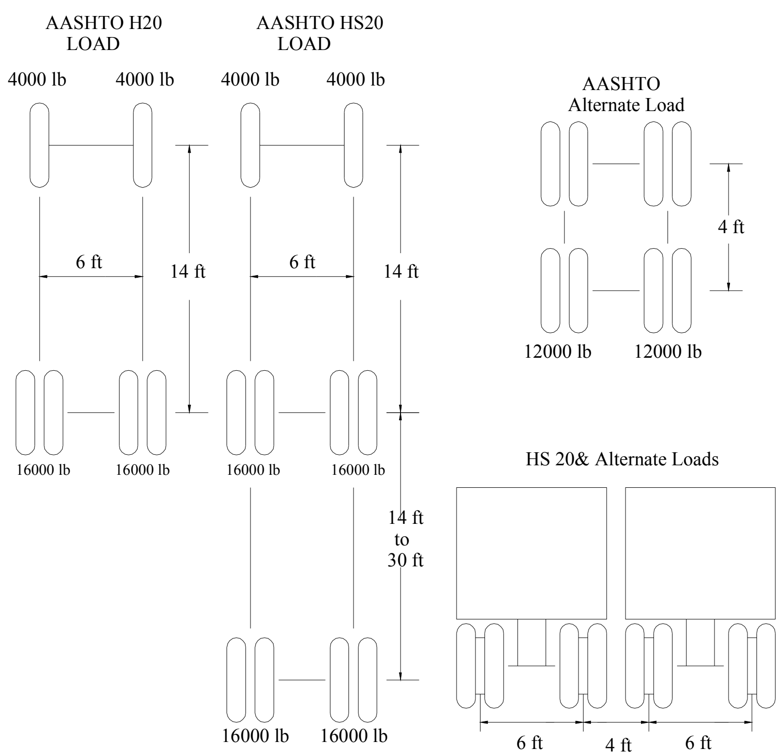

Figure 6.3 AASHTO Live Load

Each of these loadings is assumed to be applied through dual wheel assemblies uniformly distributed over a surface area of 10 inches by 20 inches, as shown in Figure 6.4.

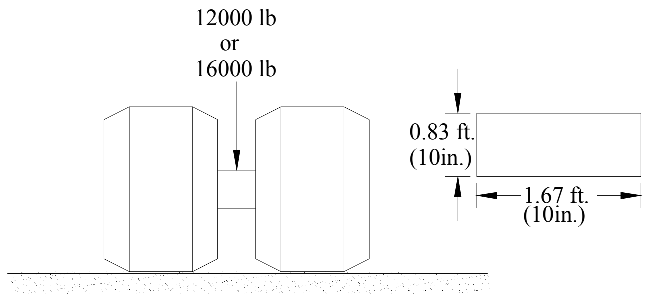

Figure 6.4 Wheel Load Surface Contact Area

The total wheel load is then assumed to be transmitted and uniformly distributed over a rectangular area on a horizontal plane at a depth H, as shown in Figure 6.5.

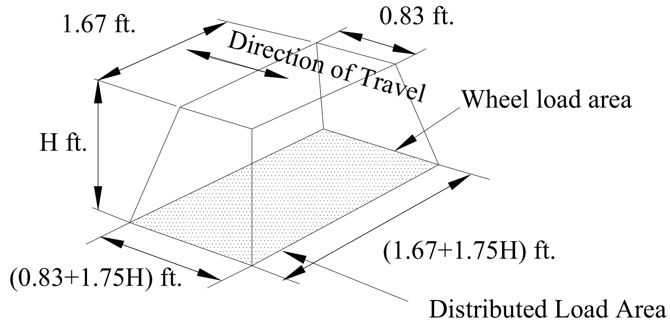

Figure 6.5 Distributed Load Area, Single Dual Wheel

The average pressure intensity on the subsoil plane at the outside top of the pipe at depth H, is determined by the equation:

$$W_L = \frac{P(1+I_f)}{A_{LL}} \qquad (6\text{-}10)$$

where,

W_L=average pressure intensity, in pounds per square foot

P=total applied surface wheel loads, in pounds

A_{LL}=distributed live load area, in square feet

I_f=impact factor (Table 6.2)

Table 6.2 Impact Factors for Highway Truck Loads

H, HEIGHT OF COVER	I_f, IMPACT FACTOR
0’-0” to 1’-0”	1.3
1’-1” to 2’-0”	1.2
2’-1” to 2’-11”	1.1
3’-0” and greater	1.0

Since the exact geometric relationship of individual or combinations of surface wheel loads cannot be anticipated, the most critical loading configurations and the outside dimensions of the distributed load areas within the indicated cover depths are summarized in Table 6.3.

Table 6.3 Critical Loading Configurations

H, feet	P, pounds	A_{LL}, Distributed Load Area
H<1.33	16,000	(0.83+1.75H)(1.67+1.75H)
$1.33 \leq H < 4.10$	32,000	(0.83+1.75H)(5.67+1.75H)
$4.10 \leq H$	48,000	(4.83+1.75H)(5.67+1.75H)

The total live load acting on the pipe is determined by the following formula

$$W_T = W_L L S_L \tag{6-11}$$

where,

W_T=total live load, in pounds

L=Length of A_{LL} parallel to longitudinal axis of pipe, in feet

S_L=outside horizontal span of pipe or width of A_{LL} transverse to longitudinal axis of pipe, whichever is less, in feet.

The live load acting on the pipe in pounds per linear foot is determined by the following equation:

$$W_L = \frac{W_T}{L_e} \tag{6-12}$$

Where,

W_L=live load on pipe, in pounds per linear foot

L_e=effective supporting length of pipe, in feet

Since the buried concrete pipe is similar to a beam on continuous supports, the effective supporting length of the pipe is assumed as in Figure 6.6 and determined by the following equation:

$$L_e = L + 1.75\left(\frac{3D_o}{4}\right) \tag{6-13}$$

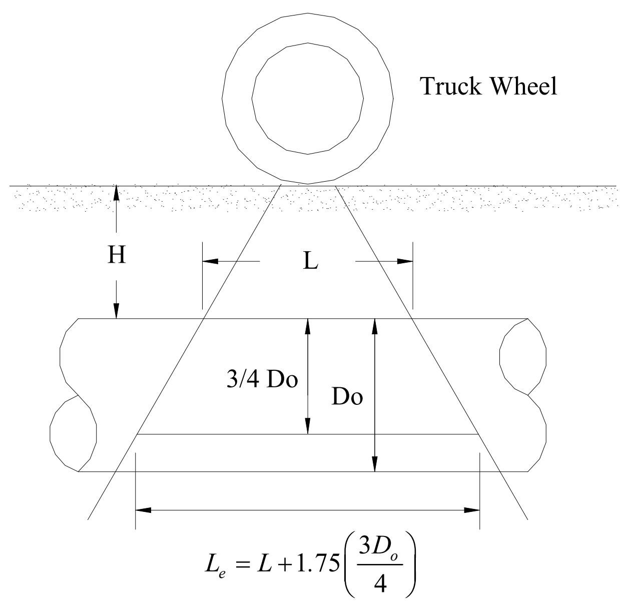

Figure 6.6 Effective Supporting Length of Pipe

Tables 42 through 45 of ACPA Concrete Pipe Design Manual [38] present the maximum highway live loads in pounds per linear foot imposed on circular, horizontal elliptical, vertical elliptical and arch pipe with impact included.

2) Airports

The distribution of aircraft wheel loads on any horizontal plane in the soil mass is dependent on the magnitude and characteristics of the aircraft loads, the aircraft's landing gear configuration, the type of pavement structure and the subsoil conditions. The distributions of wheel loads through rigid and flexible pavements are shown in Figures 6.7 and 6.8.

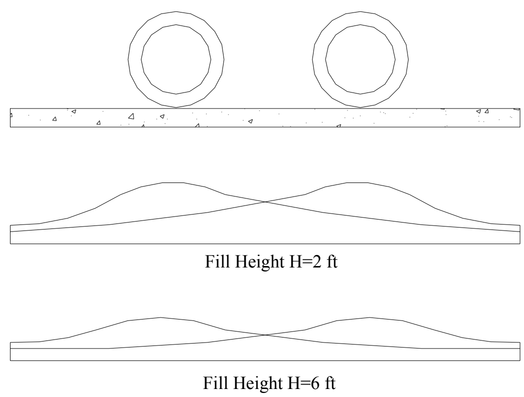

Figure 6.7 Aircraft Pressure Distribution for Rigid Pavement

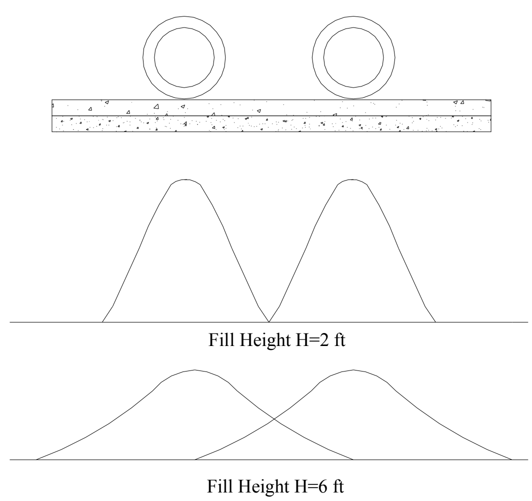

Figure 6.8 Aircraft Pressure Distribution for Flexible Pavement

Rigid Pavement

The pressure intensity is computed by the equation:

$$p(H,X)=\frac{CP}{R_s^2} \tag{6-13}$$

R_s is further defined as:

$$R_s=\sqrt[4]{\frac{E_h^{\,3}}{12(1-u^2)k}} \tag{6-14}$$

where,

C=Pressure Coefficient

P=wheel load, in pounds

R_s=radius of stiffness of rigid pavement slab, in feet

E_h=4,000,000psi

u=0.15

k=modulus of subgrade reaction

Tables 46 through 50 of ACPA Concrete Pipe Design Manual [38] present values for the pressure coefficients

Flexible Pavement

The pressure intensity is computed by the equation:

$$P(H,X)=Cp_o \tag{6-15}$$

Where,

p_o=tire pressure, in psf

The pressure coefficient, C, is dependent on the horizontal distance (X), the vertical distance (H) between the pipe and the surface load, and the radius of the circle of pressure at the surface (r). r is further defined as:

$$r=\sqrt{\frac{P}{p_o\pi}} \tag{6-16}$$

Pressure coefficients in terms of the radius of the circle of pressure at the surface(r) are presented in Table 51 of the ACPA Concrete Pipe Design Manual [38]. For rigid and flexible pavements, Tables 53 through 55 of the ACPA Concrete Pipe Design Manual [38] present aircraft loads in pounds per linear foot for circular, horizontal elliptical and arch pipe. These Tables are based on Equations 6-14 and 6-15.

3) Railroads.

In determining the live load transmitted to a pipe installed under railroad tracks, the weight on the locomotive driver axles plus the weight of the track structure including ballast is considered to be uniformly distributed over an area equal to the length occupied by the drivers multiplied by the length of the ties. The American Railway Engineering and Maintenance of Way Association (AREMA) recommends a Cooper E80 loading, with axle loads and axle spacing as shown in Figure 6.9. Based on a uniform load distribution at the bottom of the ties and through the soil mass, the live load transmitted to a pipe underground is computed by the equation:

$$W_L = Cp_o B_c I_f \tag{6-17}$$

Tables 56 through 58 of the ACPA Concrete Pipe Design Manual [38] present live loads in pounds per linear foot based on Equation 6-16 with a Cooper E80 design loading, a track structure weighing 200 pounds per linear foot, and the locomotive load uniformly distributed over an area 8 feet×20 feet, yielding a uniform live load of 2025 pounds per square foot.

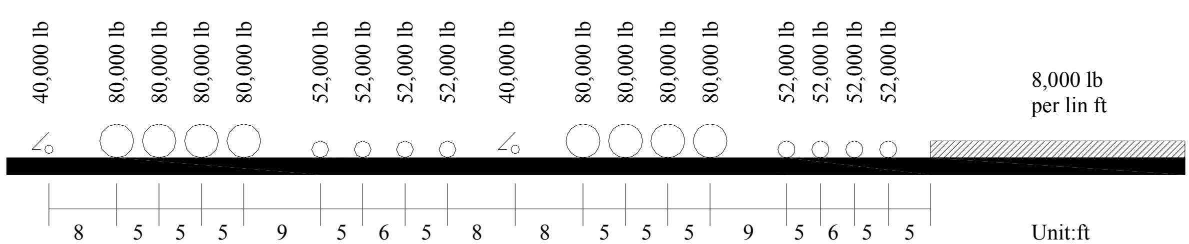

Figure 6.9 Cooper E80 Design Load

4) Construction Loads

During grading operations it may be necessary for heavy construction equipment to travel over an installed pipe. Unless adequate protection is provided, the pipe may be subjected to load concentrations in excess of the design loads. Before heavy construction equipment is permitted to cross over a pipe, a temporary earth fill should be constructed to an elevation at least 3 feet over the top of the pipe. The fill should be of sufficient width to prevent possible lateral displacement of the pipe.

6.2.2.3 Bedding Factors

Under installed conditions, the vertical load on a pipe is distributed over its width and the reaction is distributed in accordance with the type of bedding. When the pipe strength used in the design has been determined by plant testing, bedding factors must be developed to relate the in-place supporting strength to the more severe plant test strength. The bedding factor is the ratio of the strength of the pipe under the installed condition of loading and bedding to the strength of the pipe in the plant test.

The development of bedding factors for Proposed Installations follows the concepts of reinforced concrete design theories. The basic definition of bedding factor is that it is the ratio of maximum moment in the three-edge bearing test to the maximum moment in the buried condition, when the vertical loads under each condition are equal:

$$B_f = \frac{M_{TEST}}{M_{FIELD}} \tag{6-18}$$

Where,

B_f = bedding factor

M_{TEST} = maximum moment in pipe wall under three-edge bearing test load, in inch-pounds

M_{TEST} = maximum moment in pipe wall under field loads, in inch-pounds

Consequently, in order to evaluate the proper bedding factor relationship, the vertical load on the pipe for each condition must be equal, which occurs when the springline axial thrusts for both conditions are equal. In accordance with the laws of statics and equilibrium,

$$M_{TEST} = [0.318 N_{FS}] \times [D + t] \tag{6-19}$$

$$M_{FIELD} = [M_{FI}] - [0.38 t N_{FI}] - [0.125 N_{FI} \times C] \tag{6-20}$$

where,

N_{FS}= axial thrust at the springline under a three-edge bearing test load, in pound per foot

D = internal pipe diameter, in inches

t = pipe wall thickness, in inches

M_{FI} = moment at the invert under field loading, in inch-pound/ft

N_{FI} = axial thrust at the invert under field loads, in pounds per foot

C = thickness of concrete cover over the inner reinforcement, in inches

Substituting Equations 6-19 and 6-20 into Equation 6-18,

$$B_f = \frac{[0.318N_{FS}]\times[D+t]}{[M_{FI}]-[0.318tN_{FI}]-[0.125N_{FI}\times C]} \tag{6-21}$$

The resulting bedding factors are presented in Table 6.4. These calculations were based on a one inch cover concrete over the reinforcement, a moment arm of 0.875d between the resultant tensile and compressive forces, and a reinforcement diameter of 0.075t.

Table 6.4 Bedding Factors, Embankment Condition, B_{fe}

Pipe Inside Diameter (in.)	Installation Type	
	TREATED	UNTREATED
12	2.5	1.7
24	2.4	1.7
36	2.3	1.7
72	2.2	1.7
144	2.2	1.7

For trench installations, as discussed previously, experience indicates that active lateral pressure increases as trench width increases to the transition width, provided the sidefill is compacted. A parameter study of Proposed Installations indicates the bedding factors are constant for all pipe diameters under conditions of zero lateral pressure on the pipe. These bedding factors are called minimum bedding factors, B_{fo}, to differentiate them from the fixed bedding factors developed by Spangler. Table 6.5 presents the trench minimum bedding factors.

Table 6.5 Trench Minimum Bedding Factors, B_{fo}

Installation Type	Minimum Bedding Factor, B_{fo}
TREATED	1.7
UNTREATED	1.5

Table 6.6 Bedding Factors, B_{fLL}, for HS20 Live Loadings

Fill Heights (ft.)	Pipe Diameter, inches				
	12	24	36	72	144
1	2.2	2.2	1.7	1.3	1.1
2	2.2	2.2	2.2	1.5	1.3
4	2.2	2.2	2.2	2.2	1.5
6	2.2	2.2	2.2	2.2	2.0

A conservative linear variation is assumed between the minimum bedding factor and the bedding factor for the embankment condition, which begins at the transition width. The equation for the variable trench bedding factor is:

$$B_{fv} = \frac{\left[B_{fe} - B_{fo}\right]\left[B_d - B_c\right]}{\left[B_{dt} - B_c\right]} + B_{fo} \quad (6\text{-}22)$$

where,

B_c=out-to-out horizontal span of pipe, in feet

B_d=trench width at top of pipe, in feet

B_{dt}=transition width at top of pipe, in feet

B_{fe}=bedding factor, embankment

B_{fo}=minimum bedding factor for the trench

B_{fv}=variable bedding factor for the trench

Transition width values, B_{dt} are provided in Tables 13 through 39 of the ACPA Concrete Pipe Design Manual [38]. For pipe installed with 6.5 ft or less of overfill and subjected to truck loads, the controlling maximum moment may be at the crown rather than the invert. Consequently, the use of an earth load bedding factor may produce unconservative designs. When HS20 or other live loadings are encountered to a significant value, the live load bedding factors, B_{LL}, presented in Table 6.6 are satisfactory for a Type 4 AASHTO Standard Installation and become increasingly conservative for Types 3, 2, and 1.

6.2.2.4 Application of Factor of Safety

The indirect design method for concrete pipe is similar to the common working stress method of steel design, which employs a factor of safety between yield stress and the desired working stress. In the indirect method, the factor of safety is defined as the relationship between the ultimate strength D-load and the 0.01inch crack D-load. This relationship is specified in the ASTM Standards C76 and C655 for concrete pipe. A factor of safety of 1.0 should be applied if the 0.01 inch crack strength is used as the design criterion rather than the ultimate strength.

6.2.2.5 Selection of Pipe Strength

The ASTM Standard C76 for reinforced concrete culvert, storm drain and sewer pipe specifies strength classes based on the D-load at 0.01inch crack and/or ultimate load. The 0.01inch crack D-load ($D_{0.01}$) is the maximum three-edge-bearing test load supported by a concrete pipe before a crack occurs having a width of 0.01inch measured at close intervals, throughout a length of at least 1 foot. Since numerous reinforced concrete pipe sizes are available, three-edge bearing test strengths are classified by D-loads. The D-load concept provides a strength classification for pipe independent of the pipe diameter. For reinforced circular pipe, the three-edge-bearing test load in pounds per linear foot equals the D-load × the inside diameter in feet.

$$T.E.B = \left[\frac{W_E}{B_{fe}} + \frac{W_L}{B_{fLL}} \right] \times F.S. \qquad (6\text{-}23)$$

The required three-edge-bearing strength for circular reinforced concrete pipe is expressed in terms of the D-load and is computed by the equation:

$$D - load = \left[\frac{W_E}{B_{fe}} + \frac{W_L}{B_{fLL}} \right] \times \frac{F.S.}{D_{in}} \qquad (6\text{-}24)$$

When an HS20 truck live loading is applied to the pipe, the live load bedding factor, B_{fLL}, is used as indicated in Equations 6-26, 6-27, and 6-28, unless the earth load bedding factor, B_{fe}, is of lesser value, in which case, the lower B_{fe} value is used in place of B_{fLL}

6.2.3 Indirect Design for Box Culverts

6.2.3.1 Design Method

The effects of soil structure interactions must be taken into account and are based on the design earth cover, sidefill compaction, and bedding characteristics. These parameters may be determined by a soil-structure interaction analysis of the system. The bedding is assumed to provide some slightly yielding, and the design earth covers and reinforcement areas are based on the weight of a column of earth over the width of the box section. The total earth load, W_E on the box section is determined from Equations 6-16, 6-17, 6-18, and Table 6.7, which provides soil-structure interaction factors for different conditions of foundation and sidefill compaction. The earth cover loads for designs given in Tables 1, 2, and 3 of AASHTO M 259 [68] are the weight of a standard weight of earth fill with a unit weight of 120 lb/ft^3, a width equal to the outside width dimensions of the box section, and a height equal to the depth of the earth cover over the top of the section. For some installations, the design engineer may determine that for a given height of cover, the weight of earth to be supported by the box section is more or less than the "standard weight of earth fill" used to develop the designs given in the tables.

For example, the Marston-Spangler theory for loads on buried structures indicates that the weight of earth that must be supported by positive projecting conduits is greater than the weight of the column of earth directly over the conduit, while the weight of earth that must be supported by “trench-type” conduits. “negative-projecting” conduits, and “induced-trench” conduit is less than the weight of earth over the conduits. Also, the designer may wish to use a unit weight of earth that is more or less than the 120 lb/ft^3 used in the “standard weight”, or may wish to include a particular uniformly distributed surface surcharge loading. For any weight of earth or surface surcharge, or both, a designer can use Tables 1, 2, 3, and 5 of AASHTO M259 [68] to determine the required area of reinforcing steel for various heights of earth cover, or the maximum height of earth cover that does not required special shear reinforcing, for any of the standard box section sizes shown in these tables. The design procedure for the selection of box section is as follows:

STEP 1. Determination of Standard Earth Load

$$W = wB_cH \tag{6-25}$$

where:

- W = standard weight of the column of earth on culvert, in lb/linear ft;
- H = height of earth cover, ft;
- B_c = out-to-out horizontal span of the conduit, ft;
- w = unit weight of earth, $\mathrm{lb/ft^3}$;

STEP 2. Selection of Sidefill Compaction

Select TREATED or UNTREATED sidefill compaction.

STEP 3. Determination of Soil-Structure Interaction Factor

Soil-structure interaction factor, F_e, accounts for the type and conditions of installation and may be determined by AASHTO or proposed equations as follows.

Embankment Installations

AASHTO

$$F_{el} = 1 + 0.20\frac{H}{B_c} \tag{6-26}$$

where,

F_{e1} = soil-structure interaction factor for embankment installations

H = height of fill above top of box culverts, in feet

B_c = out-to-out horizontal span of the conduits, in feet

F_{e1} need not be greater than 1.15 for installations with compacted fill (TREATED) at the sides of the box section, and need not be greater than 1.4 for installations with uncompacted fill (UNTREATED) at the sides of the box section.

Table 6.7 Proposed Equations for Soil-Structure Interaction Factor

Sidefill	F_{e1} for top earth pressure	F_{e1} for Bottom pressure
TREATED	$F_{e1} = -0.009(H/B_c)^2 + 0.109(H/B_c) + 1.131$	$F_{e1} = 1.823(H/B_c)^{-0.136}$
UNTREATED	$F_{e1} = -0.007(H/B_c)^2 + 0.077(H/B_c) + 1.357$	$F_{e1} = 2.803(H/B_c)^{-0.169}$

Trench Installations

$$F_{e2} = \frac{C_d B_d^2}{HB_c} \tag{6-27}$$

where,

F_{e2} = soil-structure interaction factor for trench installations

C_d = load coefficient for trench installation

B_d = horizontal width of trench at top of box culverts, in feet

H = height of fill above top of box culverts, in feet

B_c = out-to-out horizontal span of the conduit, in feet

Values of C_d can be obtained from Figure 2.5 for normally encountered soils. The maximum value of F_{e2} need not exceed F_{e1}.

STEP 4. Determination of Total Earth Load

$$W_E = F_e w B_c H \tag{6-28}$$

where:

W_E = total weight of the column of earth on culvert, in lb/linear ft;

H = height of earth cover, ft;

B_c = out-to-out horizontal span of the conduit, ft;

w = unit weight of earth, $\mathrm{lb/ft^3}$;

F_e = soil-structure interaction factor, from Table 6.7.

STEP 5. Determination of Box Section

Select standard box section from Table 3 of AASHTO M 259.

For special designs, reinforcing steel areas are determined based on the elastic method of structural analysis and the ultimate strength method of reinforced concrete design given in ACI Building Code (ACI 318-02) [69]. Steel areas are governed by the ultimate flexural strength.

6.2.4 Imperfect Trench Design

For the case of an imperfect trench installation, STEP 0 must be added in order to convert the required fill height (H_{req}) to the equivalent fill height (H_{equiv}). The objective of STEP 0 is to take into account the reduction in the earth load. STEP 0 reduces the wall thickness and reinforcing steel areas of box culverts accordingly.

STEP 0

$$H_{equiv} = H_{req}\,(1-R) \tag{6-29}$$

Where;

H_{equiv} = Equivalent fill height (or maximum fill height), in feet

H_{req} = Required fill height (or maximum fill height with imperfect trench installation), in feet

R = Reduction Rate (0< R <1)

Table 6.8 Proposed Equations for the Reduction Rate of Soft Materials – Concrete Pipe

Soft Zone	Foundation	Sidefill Compaction	Equations for Reduction Rate
Geometry I	Yielding Or Unyielding	TREATED	$R = 0.445 - 6\times10^{-4} E_s$
		UNTREATED	$R = 0.345 - 4\times10^{-4} E_s$
Geometry II	Yielding Or Unyielding	TREATED	$R = 0.824 e^{-0.0027 E_s}$
		UNTREATED	$R = 0.821 e^{-0.0037 E_s}$

Table 6.9 Proposed Equations for the Reduction Rate of Soft Materials – Box Culverts

Soft Zone	Foundation	Sidefill Compaction	Equations for Reduction Rate
Geometry I	Yielding	TREATED	R= -0.0003 E_s + 0.275
		UNTREATED	R= -0.0004 E_s + 0.355
	Unyielding	TREATED	R= -0.0004 E_s + 0.374
		UNTREATED	R= -0.0005 E_s + 0.407
Geometry II	Yielding	TREATED	R= 0.7334 $e^{-0.0041 E_s}$
		UNTREATED	R= 0.6488 $e^{-0.0032 E_s}$
	Unyielding	TREATED	R= 0.8212 $e^{-0.0037 E_s}$
		UNTREATED	R= 0.7193 $e^{-0.0030 E_s}$

The procedures that follow STEP 0 are the same as those used in the standard design, described previously.

6.3 Design Examples

The following design examples illustrate how the abovementioned procedures may be used to determine a suitable design for a conduit section to support an earth load that is greater than the standard weight of earth used to develop Tables 1, 2, and 3 in AASHTO M 259.

6.3.1 Concrete Roadway Pipe

6.3.1.1 Positive Projection Embankment Installation

Example 1

Given: A 72 in. circular concrete pipe with a Wall B is to be installed in a positive projecting embankment condition using a TREATED installation. The pipe will be covered with 32ft. of 120 $\mathrm{lb/ft^3}$ overfill.

Find: 1) The required pipe strength in terms of 0.01 inch D-load.

2) Appropriate Pipe Class from Tables 1,2,3,4, and 5 of AASHTO M 170

STEP 1. Determination of Earth Load

From Equation 6-1, determine the soil prism load and multiply it by the appropriate Vertical Arching Factor.

Prism Load

$$D_0 = \frac{D_i + 2(t)}{12} = \frac{72 + 2(7)}{12} = 7.167\ ft$$

$$PL = w\left[H + \frac{D_0(4-\pi)}{8}\right]D_0 = 120\left[32 + \frac{7.17(4-\pi)}{8}\right](7.17) = 28{,}194\ lb/ft$$

$$W_E = VAF \times PL = 1.4 \times 28{,}194 = 39{,}472\ lb/ft$$

Where,

t = wall thickness, in

w = soil unit weight, lbs/ft^3

H = Height of fill, ft

D_{in} = inside diameter, in

D_0 = outside diameter, ft

VAF=1.40 for TREATED

STEP 2. Determination of Live Load

The live load is negligible at depths of over 10 feet.

STEP 3. Determination of Bedding Factor

A TREATED installation will be used for this example.

The embankment bedding factor for a TREATED installation may be interpolated from Figure 6.10.

B_{fe72}=2.2

STEP 4. Application of Factor of Safety

A factor of safety of 1.0 should be applied if the 0.01 inch crack strength is to be used as the design criterion rather than the ultimate strength.

STEP 5. Determination of the required D-Load strength

This D-Load is the service load condition and is given by Equation 6-24.

$$D_{0.01} = \left[\frac{W_E}{B_{fe}} + \frac{W_L}{B_{fLL}}\right] \times \frac{F.S.}{D_{in}} = \left[\frac{39,472}{2.2}\right] \times \frac{1}{6} = 2,990\ lb/ft/ft$$

A pipe which would withstand a minimum three-edge bearing test for the 0.01 inch crack of 2,990 pounds per linear foot per foot of inside diameter would be required.

STEP 6. Selection of Pipe Class

Select a pipe that is in Class V from Table 5 of AASHTO M 170.

Example 2

Given: A 120 in. circular concrete pipe with a Wall B is to be installed in a positive projecting embankment condition using UNTREATED installation. The pipe will be covered with 21ft. of 120 $\mathrm{lb/ft^3}$ overfill.

Find: 1) The required pipe strength in terms of 0.01 inch D-load.

2) Appropriate Pipe Class from Tables 1,2,3,4, and 5 of AASHTO M 170

STEP 1. Determination of Earth Load

From Equation 6-1, determine the soil prism load and multiply it by the appropriate Vertical Arching Factor.

Prism Load

$$D_0 = \frac{D_i + 2(t)}{12} = \frac{120 + 2(10)}{12} = 11.67\ ft$$

$$PL = w\left[H + \frac{D_0(4-\pi)}{8}\right]D_0 = 120\left[21 + \frac{11.67(4-\pi)}{8}\right](11.67) = 31{,}161\ lb/ft$$

$$W_E = VAF \times PL = 1.45 \times 31{,}161 = 45{,}183\ \ lb/ft$$

where,

t = wall thickness, in

w = soil unit weight, $\mathrm{lbs/ft^3}$

H = Height of fill, ft

D_{in} = inside diameter, in

D_0 = outside diameter, ft

VAF=1.45 for UNTREATED

STEP 2. Determination of Live Load

The live load is negligible at depths of over 10 feet.

STEP 3. Determination of Bedding Factor

UNTREATED installation will be used for this example.

The embankment bedding factor for UNTREATED installation may be interpolated from Figure 6.10.

B_{fe120}=1.7

STEP 4. Application of Factor of Safety

A factor of safety of 1.0 should be applied if the 0.01 inch crack strength is to be used as the design criterion rather than the ultimate strength.

STEP 5. Determination of the required D-Load strength

This D-Load is the service load condition and is given by Equation 6-24:

$$D-load = \left[\frac{W_E}{B_{fe}} + \frac{W_L}{B_{fLL}}\right] \times \frac{F.S.}{D_{in}} = \left[\frac{45,183}{1.7}\right] \times \frac{1}{10} = 2,657\ lb/ft/ft$$

A pipe which would withstand a minimum three-edge bearing test for the 0.01 inch crack of 2,657 pounds per linear foot per foot of inside diameter would be required.

STEP 6. Selection of Pipe Class

Select a pipe that is in Class V from Table 5 of AASHTO M 170, as the D-load (lb/ft/ft) is between 2,000 ~ 3,000. For modified or special designs, see Section F.2.1.3.

Example 3

Given: A 144 in. circular concrete pipe with a Wall B is to be installed in a positive projecting embankment condition using a TREATED installation. The pipe will be covered with 90ft. of 120 $\mathrm{lb/ft^3}$ overfill.

Find: 1) The required pipe strength in terms of 0.01 inch D-load.

2) Appropriate Pipe Class from Tables 1,2,3,4, and 5 of AASHTO M 170

STEP 1. Determination of Earth Load

From Equation 6-1, determine the soil prism load and multiply it by the appropriate Vertical Arching Factor.

Prism Load

$$D_0 = \frac{D_i + 2(t)}{12} = \frac{144 + 2(12)}{12} = 14\ ft$$

$$PL = w\left[H + \frac{D_0(4-\pi)}{8}\right]D_0 = 120\left[90 + \frac{14(4-\pi)}{8}\right](14) = 153,723\ lb/ft$$

$$W_E = VAF \times PL = 1.40 \times 153,723 = 215,212\ \ lb/ft$$

where,

t = wall thickness, in

w = soil unit weight, lbs/ft^3

H = Height of fill, ft

D_{in} = inside diameter, in

D_0 = outside diameter, ft

VAF=1.40 for TREATED

STEP 2. Determination of Live Load

The live load is negligible at depths of over 10 feet.

STEP 3. Determination of Bedding Factor

A TREATED installation will be used for this example.

The embankment bedding factor for a TREATED installation may be interpolated from Table 6.4.

B_{fe144}=2.2

STEP 4. Application of Factor of Safety

A factor of safety of 1.0 should be applied if the 0.01 inch crack strength is to be used as the design criterion rather than the ultimate strength.

STEP 5. Determination of the required D-Load strength

This D-Load is the service load condition. The D-load is given by Equation 6-24.

$$D-load = \left[\frac{W_E}{B_{fe}} + \frac{W_L}{B_{fLL}}\right] \times \frac{F.S.}{D_{in}} = \left[\frac{215,212}{2.2}\right] \times \frac{1}{12} = 8,151\ lb/ft/ft$$

A pipe which would withstand a minimum three-edge bearing test for the 0.01 inch crack of 8,151 pounds per linear foot per foot of inside diameter would be required.

STEP 6. Selection of Pipe Class

A special designs is needed, as the D-load (lb/ft/ft) exceeds 3,000.

For modified or special designs, see Section 5.2.1.2.

6.3.1.2 Negative Projection Embankment Installation (Figure 2.3(c))

Given: A 72 in. circular concrete pipe with a Wall B is to be installed in a negative projecting embankment condition in ordinary soil. The pipe will be covered with 35ft. of 120 $\mathrm{lb/ft^3}$ overfill. A 10 ft trench width will be constructed with a 5 ft depth from the top of the pipe to the natural ground surface.

Find: 1) The required pipe strength in terms of 0.01 inch D-load.

2) Appropriate Pipe Class from Tables 1,2,3,4, and 5 of AASHTO M 170

STEP 1. Determination of Earth Load

A settlement ratio must be assumed. The negative projection ratio of this installation is the height of the soil from the top of the pipe to the top of the natural ground (5ft) divided by the trench width (10ft). The negative projection ratio of this installation is therefore p'=0.5, and a typical value of the settlement ratio for negative projecting embankments used in practice is -1, as given in Table 6.1.

From Figure 2.7(a), for H/B_d=35/10=3.5, C_n=2.75. Determine the earth load from Equation 2-3b.

$$W = C_n \gamma B_d^2 = (2.75)(120)(10)^2 = 33,000\ lb/ft$$

STEP 2. Determination of Live Load

The live load is negligible at depth of over 35 feet.

STEP 3. Determination of Bedding Factor

No specific bedding was given. A TREATED installation will be used for this example. The variable bedding factor will be determined using Equation 6-22.

$B_c = \frac{72+2\times7}{12} = 7.17\,ft$, outside diameter of pipe

$B_d = 10\ ft$, trench width

$B_{dt} = 13.7\ ft$, transition width for a Type 3 installation with $K\mu' = 0.130$ interpolated from Table 2.7. of ACPA concrete design manual [38].

$B_{fe} = 2.2\ ft$, embankment bedding factor from Table 6.4.

$B_{fo} = 1.7\ ft$, minimum bedding factor from Table 6.5.

$$B_{fv} = \frac{\left[B_{fe} - B_{fo}\right]\left[B_d - B_c\right]}{\left[B_{dt} - B_c\right]} + B_{fo} = \frac{(2.2-1.7)(10-7.17)}{13.7-7.17} + 1.7 = 1.9$$

STEP 4. Application of Factor of Safety

A factor of safety of 1.0 should be applied if the 0.01 inch crack strength is to be used as the design criterion rather than the ultimate strength.

STEP 5. Determination of the required D-Load strength

This D-Load is the service load condition. The D-load is given by Equation 6-24.

$$D-load = \left[\frac{W_E}{B_{fe}} + \frac{W_L}{B_{fLL}}\right] \times \frac{F.S.}{D_{in}} = \left[\frac{33,000}{1.9}\right] \times \frac{1}{6} = 2,894\ lb/ft/ft$$

A pipe which would withstand a minimum three-edge bearing test for the 0.01 inch crack of 2,894 pounds per linear foot per foot of inside diameter would be required.

STEP 6. Selection of Pipe Class

A pipe that is in Class V should be selected from Table 5 of AASHTO M 170.

6.3.1.3 Trench Installation (Figure 2.3(a))

Given: A 48 in. circular concrete pipe with a Wall B is to be installed in a 7 ft wide trench with 10ft of cover over the top of the pipe. The pipe will be backfilled with sand of 120 $\mathrm{lb/ft^3}$ overfill. Assume UNTREATED sidefill.

Find: 1) The required pipe strength in terms of 0.01 inch D-load.

2) Appropriate Pipe Class from Tables 1,2,3,4, and 5 of AASHTO M 170

STEP 1. Determination of Earth Load

To determine the earth load, we must first determine if the installation is behaving as a trench installation or an embankment installation. Since we are not told what the existing in-situ material is, assume a $K\mu'$ value between the existing soil and backfill of 0.150. From Table 27 of the ACPA concrete design manual [38], the transition width with $K\mu' = 0.130$ under 10 feet of fill is:

$$B_{dt} = 8.5\ ft$$

The transition width is thus greater than the actual trench width, and therefore the installation will act as a trench.

$w = 120\,lb/ft^2$, unit weight of soil

$$H = 10\,ft$$

$$B_d = 7\,ft$$

$$K\mu' = 0.150$$

$$D_o = \frac{48 + 2\times(5)}{12} = 4.83\ ft$$

Calculate the earth load using Equations 6-6 and 6-7

$$C_d = \frac{1 - e^{-2K\mu' \frac{H}{B_d}}}{2K\mu'} = \frac{1 - e^{-2 \times (0.150) \times \left(\frac{10}{7}\right)}}{(2) \times (0.150)} = 1.16$$

$$W_d = C_d w B_d^2 + \frac{D_o^2 (4 - \pi)}{8} w = 1.16(120)(7)^2 + \frac{(4.83)^2 (4 - \pi)}{8} \times 120 = 7,121\ lb/ft$$

STEP 2. Determination of Live Load

The live load is negligible at depths of over 10 feet.

STEP 3. Determination of Bedding Factor

Because of the narrow trench, good compaction of the soil on the sides of the pipe would be difficult. Therefore, UNTREATED sidefill is assumed. The variable bedding factor will be determined using Equation 6-22.

$B_c = D_o = 4.83\ ft$, outside diameter of pipe

$B_d = 7\ ft$, trench width

$B_{dt} = 8.5\ ft$, transition width for a Type 4 installation with $K\mu' = 0.150$ interpolated from Table 27 of ACPA concrete design manual [38].

$B_{fe} = 1.7\ ft$, embankment bedding factor from Table 6.4.

$B_{fo} = 1.5\ ft$, minimum bedding factor from Table 6.5.

$$B_{fv} = \frac{[B_{fe} - B_{fo}][B_d - B_c]}{[B_{dt} - B_c]} + B_{fo} = \frac{(1.7 - 1.5)(7 - 4.83)}{8.5 - 4.83} + 1.5 = 1.62$$

STEP 4. Application of Factor of Safety

A factor of safety of 1.0 should be applied if the 0.01 inch crack strength is to be used as the design criterion rather than the ultimate strength.

STEP 5. Determination of the required D-Load strength

This D-Load is the service load condition. The D-load is given by Equation 6-24.

$$D-load=\left[\frac{W_E}{B_{fe}}+\frac{W_L}{B_{fLL}}\right]\times\frac{F.S.}{D_{in}}=\left[\frac{7,121}{1.62}\right]\times\frac{1}{4}=1,098\ lb/ft/ft$$

A pipe which would withstand a minimum three-edge bearing test for the 0.01 inch crack of 1,098 pounds per linear foot per foot of inside diameter would be required.

STEP 6. Selection of Pipe Class

A pipe that is in Class III should be selected from Table 3 of AASHTO M 170.

6.3.1.4 Imperfect Trench Installation

Apply an imperfect trench installation to Example 3 in Section 6.3.1.1.

Example 3-1

Given: A 144 in. circular concrete pipe with a Wall B is to be installed in an imperfect trench installation using a TREATED installation. The pipe will be covered with 90ft. of 120 $\mathrm{lb/ft^3}$ overfill.

Find: 1) The required pipe strength in terms of 0.01 inch D-load.

2) Appropriate Pipe Class from Tables 1,2,3,4, and 5 of AASHTO M 170

STEP 0

Determination of Soft zone and material

Soft zone: Geometry II or I

Material: EPS Geofoam (E_s = 50-100psi)

Determination of Equivalent Fill Height

In the case of Geometry I for soft zone, From Equation 6-25 and Table 6.8, we get

$$R = 0.445 - 6\times10^{-4}E_s = 0.445 - 6\times10^{-4}\left(100\right) = 0.385$$

$$H_{equiv} = H_{req}\,(1-R) = 90(1-0.385) = 55.35 \cong 56\,ft$$

In the case of Geometry II for the soft zone,

From Equation 6-25 and Table 6.8, we get

$$R = 0.824\,e^{-0.0027E_s} = 0.824\,e^{-0.0027(100)} = 0.629$$

H_{equiv} = H_{req} (1-R)=90(1-0.629)=33.39 ≅ 34 ft

We can design the pipe for an earth fill of 56ft or 34ft instead of 90ft.

Choose 34ft and Geometry II for this example.

The following procedures are same as those used in the standard design

STEP 1. Determination of Earth Load

From Equation 6-1, determine the soil prism load and multiply it by the appropriate Vertical Arching Factor.

Prism Load

$$D_0 = \frac{D_i + 2\left(t\right)}{12} = \frac{144 + 2\left(12\right)}{12} = 14\,ft$$

$$PL = w\left[H + \frac{D_0\left(4-\pi\right)}{8}\right]D_0 = 120\left[34 + \frac{14\left(4-\pi\right)}{8}\right]\left(14\right) = 59,643\ lb/ft$$

$$W_E = VAF \times PL = 1.40 \times 59,643 = 83,500\ lb/ft$$

where,

t = wall thickness, in

w = soil unit weight, lbs/ft^3

H = Height of fill, ft

D_{in} = inside diameter, in

D_0 = outside diameter, ft

VAF=1.40 for TREATED

STEP 2. Determination of Live Load

The live load is negligible at depths of over 10 feet.

STEP 3. Determination of Bedding Factor

A TREATED installation will be used for this example.

The embankment bedding factor for a TREATED installation may be interpolated from Figure 6.10.

B_{fe144}=2.2

STEP 4. Application of Factor of Safety

A factor of safety of 1.0 should be applied if the 0.01 inch crack strength is to be used as the design criterion rather than the ultimate strength.

STEP 5. Determination of the required D-Load strength

This D-Load is the service load condition and is given by Equation 6-24.

$$D-load=\left[\frac{W_E}{B_{fe}}+\frac{W_L}{B_{fLL}}\right]\times\frac{F.S.}{D_{in}}=\left[\frac{83,500}{2.2}\right]\times\frac{1}{12}=3,163\ lb/ft/ft$$

A pipe which would withstand a minimum three-edge bearing test for the 0.01 inch crack of 3,163 pounds per linear foot per foot of inside diameter would be required.

STEP 6. Selection of Pipe Class

In case of example 3 using only embankment installation, a specially designed pipe was needed, as the D-load (lb/ft/ft) exceeded 3,000. Using the Imperfect trench installation, however, standard pipe class V from design table 5 of AASHTO M 170 was suitable.

6.3.2 Box Culverts

6.3.2.1 Positive Projection Embankment Installation

1) Standard design

Example 1

Given: A 12ft by 12ft by 12in. precast concrete box section is to be installed under 14ft of cover with 115 $\mathrm{lb/ft^3}$ earth instead of the standard 120 $\mathrm{lb/ft^3}$ earth in a positive projecting embankment condition using TREATED installation with a yielding foundation.

Find: The required A_{s1}, A_{s2}, A_{s3} circumferential reinforcement areas. In all cases A_{s4} is governed by the minimum steel areas, as described in A1.4.2 of AASHTO M 259 and is not changed by increased vertical loads.

STEP 1. Determination of Standard Earth Load

Determine the standard weight of earth fill equivalent to the weight of a column of earth with a unit on the culvert in lbs force/linear ft

$$W = wB_cH$$

where:

W = standard weight of the column of earth on culvert, lb/linear ft;

H = height of earth cover, ft;

B_c = out-to-out horizontal span of the conduit, ft;

w = unit weight of earth, $\mathrm{lb/ft^3}$;

$$\therefore W_{115} = wB_cH = (115)(12+2\times12/2)(14) = 22,540\,lb/ft$$

STEP 2. Selection of Sidefill Compaction

Select TREATED sidefill compaction and a yielding foundation.

STEP 3. Determination of the Soil-Structure Interaction Factor

F_e = Soil-structure interaction factor

AASHTO

$F_{el} = 1 + 0.20\frac{H}{B_c}$, Not greater than 1.15 with TREATED sidefill and 1.4 with UNTREATED sidefill

$$F_{el} = 1 + 0.20\left(\frac{14}{12 + 2*12/12}\right) = 1.2 > 1.15$$

$$\therefore F_e = 1.15$$

Proposed equation for bottom pressure

$F_e = 1.823(H/B_c)^{-0.136}$, for TREATED sidefill

$F_e = 2.803(H/B_c)^{-0.169}$, for UNTREATED sidefill

$\therefore F_e = 1.823(14/14)^{-0.136} = 1.823$, controls

Proposed equation for top earth pressure

$F_e = -0.009(H/B_c)^2 + 0.109(H/B_c) + 1.131$, for TREATED sidefill

$F_e = -0.007(H/B_c)^2 + 0.077(H/B_c) + 1.357$, for UNTREATED sidefill

$$\therefore F_e = -0.009(14/14)^2 + 0.109(14/14) + 1.131 = 1.231$$

STEP 4. Determination of Total Earth Load

$$W_E = F_e w B_c H$$

where:

W_E = total weight of the column of earth on culvert, lb/linear ft;

H = height of earth cover, ft;

B_c = out-to-out horizontal span of the conduit, ft;

w = unit weight of earth, $\mathrm{lb/ft^3}$;

F_e = Soil-structure interaction factor, from Equation 6-27 and Table 6.7.

$$W_E = F_e w B_c H = (1.823)(115)(12+2\times 12/12)(14) = 41,090\, lb/ft$$

STEP 5. Determination of Box Section

Determine change in total weight of earth on culvert in lbs/ft

$$W_s = w B_c H = (120)(12+2\times 12/12)(14) = 23,520\, lb/ft$$

$$W_E = F_e w B_c H = (1.823)(115)(12+2\times 12/12)(14) = 41,090\, lb/ft$$

$$W = W_E - W_s = 41,090 - 23,520 = 17,570\, lb/ft$$

Determine the change in circumferential reinforcement areas.

From Table 3 of AASHTO M 259, for a 12ft by 12ft by 12in. section under 14ft of cover A_{s1}=0.17, A_{s2}=0.17, and A_{s3}=0.17in.^{2}/ft. From Table 5 of AASHTO M 259, for a 12ft by 12ft by 12in. section, the changes in reinforcing areas are A_{s1}=0.013, A_{s2} and A_{s3}=0.030in.^{2}/ft for each 1000 lbf/ft of load change. Therefore:

$$\Delta A_{s1} = 17.57\times 0.013 = 0.228$$

$$\Delta A_{s2} = 17.57\times 0.030 = 0.527$$

$$\Delta A_{s3} = 17.57\times 0.030 = 0.527$$

Therefore, the correct reinforcement areas are as follows:

$$A_{s1} = 0.42 + 0.228 = 0.648\, in.^2/ft$$

$$A_{s2} = 0.68 + 0.527 = 1.207\, in.^2/ft$$

$$A_{s3} = 0.75 + 0.527 = 1.277\, in.^2/ft$$

2) Special design

Example 2

The design tables of AASHTO M 259 are not applicable to deeply buried box culverts. Therefore, we need special designs for sizes and loads other than those shown in Tables 1, 2, and 3 of AASHTO M 259. The following is an example of this type of special design.

Given: A 14ft by 14ft by 14in. precast concrete box section is to be installed under 60ft of cover with 110 $\mathrm{lb/ft^3}$ earth instead of the standard 120 $\mathrm{lb/ft^3}$ earth in a positive projecting embankment condition using UNTREATED installation and an unyielding foundation.

Find: the total earth load in lbs force/linear ft

STEP 1. Determination of Standard Earth Load

Determine the standard weight of earth fill equal to the weight of a column of earth with a unit weight of 110 lb/ft^3 on the culvert in lbs force/linear ft

$$W = wB_cH$$

where:

W = standard weight of the column of earth on culvert, lb/linear ft;

H = height of earth cover, ft;

B_c = out-to-out horizontal span of the conduits, ft;

w = unit weight of earth, $\mathrm{lb/ft^3}$;

$$\therefore W_{110} = wB_cH = (110)(14 + 2\times 14/12)(60) = 107,800\, lb/ft$$

STEP 2. Selection of Sidefill Compaction

Select UNTREATED sidefill compaction with an unyielding foundation.

STEP 3. Determination of Soil-Structure Interaction Factor

F_e = Soil-structure interaction factor, from Equation 6-27 and Table 6.7.

AASHTO

$F_{el} = 1 + 0.20\frac{H}{B_c}$, not greater than 1.15 with TREATED sidefill and 1.4 with UNTREATED sidefill

$$F_{el} = 1 + 0.20\left(\frac{60}{14 + 2*14/12}\right) = 1.73 > 1.40$$

$$\therefore F_e = 1.40$$

Proposed equation for bottom pressure

$F_e = 1.823(H/B_c)^{-0.136}$, for TREATED sidefill

$F_e = 2.803(H/B_c)^{-0.169}$, for UNTREATED sidefill

$$\therefore F_e = 2.803(60/16.33)^{-0.169} = 2.249$$

Proposed equation for top earth pressure

$F_e = -0.009(H/B_c)^2 + 0.109(H/B_c) + 1.131$, for TREATED sidefill

$F_e = -0.007(H/B_c)^2 + 0.077(H/B_c) + 1.357$, for UNTREATED sidefill

$$\therefore F_e = -0.007(60/16.33)^2 + 0.077(60/16.33) + 1.357 = 1.545$$

STEP 4. Determination of Total Earth Load

$$W_E = F_e w B_c H$$

where:

W_E = total weight of the column of earth on culvert, lb/linear ft;

H = height of earth cover, ft;

B_c = out-to-out horizontal span of the conduit, ft;

w = unit weight of earth, lb/ft^3;

F_e = Soil-structure interaction factor, from Equation 6-27 and Table 6.7.

$$\therefore W_E = F_e w B_c H = (2.249)(110)(14 + 2\times 14/12)(60) = 242,442\, lb/ft$$

STEP 5. Determination of Box Section

Reinforcing steel areas are determined based on the elastic method of structural analysis and the ultimate strength method of reinforced concrete design given in the ACI Building Code (ACI 318-02) [69]. Steel areas are governed by ultimate flexural strength.

Example 3

Given: A 12ft by 12ft by 12in. precast concrete box section is to be installed under 100ft of cover with 110 $\mathrm{lb/ft^3}$ earth instead of the standard 120 $\mathrm{lb/ft^3}$ earth in a positive projecting embankment condition using TREATED installation with a yielding foundation.

Find: the total earth load in lbs force/linear ft

STEP 1. Determination of Standard Earth Load

Determine standard weight of earth fill equal to the weight of a column of earth with a unit weight of 110 $\mathrm{lb/ft^3}$ on the culvert in lbs force/linear ft

$$W = wB_cH$$

where:

W = standard weight of the column of earth on culvert, lb/linear ft;

H = height of earth cover, ft;

B_c = out-to-out horizontal span of the conduit , ft;

w = unit weight of earth, $\mathrm{lb/ft^3}$;

$$\therefore W_{110} = wB_cH = (110)(12 + 2 \times 12/12)(100) = 154,000\,lb/ft$$

STEP 2. Selection of Sidefill Compaction

Select TREATED sidefill compaction and a yielding foundation.

STEP 3. Determination of Soil-Structure Interaction Factor

F_e = Soil-structure interaction factor, from Equation 6-17 and Table 6.7.

AASHTO

$F_{el} = 1 + 0.20\dfrac{H}{B_c}$, not greater than 1.15 with TREATED sidefill and 1.4 with UNTREATED sidefill

$$F_{el} = 1 + 0.20\left(\frac{100}{12 + 2*12/12}\right) = 2.43 > 1.15$$

$$\therefore F_e = 1.15$$

Proposed equation for bottom pressure

$F_e = 1.823(H/B_c)^{-0.136}$, for TREATED sidefill

$F_e = 2.803(H/B_c)^{-0.169}$, for UNTREATED sidefill

$$\therefore F_e = 1.823(100/14)^{-0.136} = 1.39$$

Proposed equation for top earth pressure

$F_e = -0.009(H/B_c)^2 + 0.109(H/B_c) + 1.131$, for TREATED sidefill

$F_e = -0.007(H/B_c)^2 + 0.077(H/B_c) + 1.357$, for UNTREATED sidefill

$\therefore \; F_e = -0.009(100/14)^2 + 0.109(100/14) + 1.131 = 1.45$, Controls

STEP 4. Determination of Total Earth Load

$$W_E = F_e w B_c H$$

where:

W_E = total weight of the column of earth on culvert, lb/linear ft;

H = height of earth cover, ft;

B_c = out-to-out horizontal span of the conduit, ft;

w = unit weight of earth, $\mathrm{lb/ft^3}$;

F_e = Soil-structure interaction factor, from Equation 6-17 and Table 6.7.

$$W_E = F_e w B_c H = (1.45)(110)(12 + 2\times 12/12)(100) = 223{,}300\, lb/ft$$

STEP 5. Determination of Box Section

Reinforcing steel areas are determined based on the elastic method of structural analysis and the ultimate strength method of reinforced concrete design given in the ACI Building Code (ACI 318-02) [69]. Steel areas are governed by the ultimate flexural strength.

6.3.2.2 Trench Installation

Example 1

Given: A 8ft by 8ft by 8in. precast concrete box section is to be installed under 120ft of cover with 120 $\mathrm{lb/ft^3}$ in a trench condition with a yielding foundation. The trench wall is assumed to be vertical.

Find: the total earth load in lbs force/linear ft

STEP 1. Determination of Standard Earth Load

Determine the standard weight of earth fill equal to the weight of a column of earth with a unit weight of 110 lb/ft^3 on the culvert in lbs force/linear ft

$$W = wB_cH$$

where:

- W = standard weight of the column of earth on culvert, lb/linear ft;
- H = height of earth cover, ft;
- B_c = out-to-out horizontal span of the conduit, ft;
- w = unit weight of earth, $\mathrm{lb/ft^3}$;

$$\therefore W_{120} = wB_cH = (120)(8+2\times 8/12)(120) = 134,400\, lb/ft$$

STEP 2. Selection of Sidefill Compaction

There are no requirements of sidefill compaction for the trench installation.

STEP 3. Determination of Soil-Structure Interaction Factor

F_e = Soil-structure interaction factor, Equation 6-27 and Table 2.7.

<u>AASHTO</u>

From Figure 2.7, $C_d = 2.75$ is taken for $K\mu^{'} = 0.165$ (maximum for sand and gravel), and from Equation 6-27, the soil-structure interaction factor and total earth load for trench installation are computed as follows:

$$F_{e2} = \frac{C_d B_d^2}{HB_c} = \frac{(2.75)\left[2\times(8+2\times 8/12)\right]^2}{(120)(8+2*8/12)} = 0.85$$

STEP 4. Determination of Total Earth Load

$$W_E = F_e w B_c H$$

where:

W_E = total weight of the column of earth on culvert, lb/linear ft;

H = height of earth cover, ft;

B_c = out-to-out horizontal span of the conduit, ft;

w = unit weight of earth, lb/ft^3;

F_{e2} = Soil-structure interaction factor, Equation 6-27 and Table 2.7.

$$W_E = F_e w B_c H = (0.85)(120)(8+2\times 8/12)(120) = 114,240\, lb/ft$$

STEP 5. Determination of Box Section

Reinforcing steel areas are determined based on the elastic method of structural analysis and the ultimate strength method of reinforced concrete design given in the 1971 ACI Building Code (ACI 318-02). Steel areas are governed by the ultimate flexural strength.

6.3.2.3 Imperfect Trench Installation

Example 3-1

Given: A 12ft by 12ft by 12in. precast concrete box section is to be installed under 100ft of cover with 110 $\mathrm{lb/ft^3}$ earth instead of the standard 120 $\mathrm{lb/ft^3}$ earth in an imperfect trench condition using TREATED installation with a yielding foundation.

Find: the total earth load in lbs force/linear ft

STEP 0

Determination of Soft zone and material

Soft zone: Geometry II or I

Material: EPS Geofoam (E_s = 50-100psi)

Determination of Equivalent Fill Height

From Equation 6-29 and Table 6.9, for Geometry I, we get

R=-0.0003(100)+0.275=0.245

$H_{equiv} = H_{req}\,(1\text{-}R)$=100(1-0.245)=75.5 $\cong$ 76 ft

In the case of Geometry II for the soft zone,

From Equation 6-29 and Table 6.9, we get

$$R = 0.7334\,e^{-0.0041\,E_s} = 0.7334\,e^{-0.0041\,(100)} = 0.487$$

$H_{equiv} = H_{req}\,(1\text{-}R)$=100(1-0.487)=51.4 $\cong$ 52 ft

We can thus design the box culvert for an earth fill of either 76ft or 52 ft instead of 100ft.

Choose 52 ft of Geometry II for this example,

The following procedures are the same as those used in the standard design

STEP 1. Determination of Standard Earth Load

Determine the standard weight of earth fill equal to the weight of a column of earth with a unit weight of 110 $\mathrm{lb/ft^3}$ on the culvert in lbs force/linear ft

$$W = wB_cH$$

where:

- W = standard weight of the column of earth on culvert, lb/linear ft;
- H = height of earth cover, ft;
- B_c = out-to-out horizontal span of the conduit, ft;
- w = unit weight of earth, $\mathrm{lb/ft^3}$;

$$W_{110} = wB_cH = (110)(12 + 2\times 12/12)(52) = 80,080\,lb/ft$$

STEP 2. Selection of Sidefill Compaction

Select TREATED sidefill compaction and a yielding foundation.

STEP 3. Determination of Soil-Structure Interaction Factor

F_e = Soil-structure interaction factor, from Equation 6-27 and Table 6.7.

<u>AASHTO</u>

$F_{el} = 1 + 0.20\dfrac{H}{B_c}$, not greater than 1.15 with TREATED sidefill and 1.4 with UNTREATED sidefill

$$F_{el} = 1 + 0.20\left(\frac{52}{12 + 2*12/12}\right) = 1.74 > 1.15$$

$$\therefore F_e = 1.15$$

<u>Proposed equation</u> for bottom pressure

$F_e = 1.823(H/B_c)^{-0.136}$, for TREATED sidefill

$F_e = 2.803(H/B_c)^{-0.169}$, for UNTREATED sidefill

$\therefore F_e = 1.823(100/14)^{-0.136} = 1.40$, controls

Proposed equation for top earth pressure

$$F_e = -0.009(H/B_c)^2 + 0.109(H/B_c) + 1.131$$, for TREATED sidefill

$$F_e = -0.007(H/B_c)^2 + 0.077(H/B_c) + 1.357$$, for UNTREATED sidefill

$$\therefore \quad F_e = -0.009(100/14)^2 + 0.109(100/14) + 1.131 = 1.45$$

STEP 4. Determination of Total Earth Load

$$W_E = F_e w B_c H$$

where:

W_E = total weight of the column of earth on culvert, lb/linear ft;

H = height of earth cover, ft;

B_c = out-to-out horizontal span of the conduit, ft;

w = unit weight of earth, $\mathrm{lb/ft^3}$;

F_e = Soil-structure interaction factor, from Equation 6-27 and Table 6.7.

$$W_E = F_e w B_c H = (1.40)(110)(12 + 2\times 12/12)(52) = 112,112\, lb/ft$$

Comments:

For Example 3 in Section 6.3.2.1 under only embankment installation, the total earth load was as follows:

$$W_E = F_e w B_c H = (1.45)(110)(12 + 2\times 12/12)(100) = 223,300\, lb/ft$$

Therefore, using an imperfect trench installation achieves reduction in the total earth load of 49.8%.

$$\frac{(223,300 - 112,112)}{223,300}\times 100 = 49.8\%$$

STEP 5. Determination of Box Section

Reinforcing steel areas are determined based on the elastic method of structural analysis and the ultimate strength method of reinforced concrete design given in the 1971 ACI Building Code (ACI 318-02) [69]. Steel areas are governed by the ultimate flexural strength.

6.4 Special Highway Drawings

As part of deliverables of the project, Special Highway Drawings which will replace RPC-530 (Special and Standard Highway Drawings, Alabama Department of Transportation, 2001 Index No. 447-448) have been prepared. Portions of these drawings are illustrated in the early part of Chapter 4. AutoCAD electronic files are given in Appendix G.

6.5 Maximum Fill Heights

Maximum Fill Height (MFH) without imperfect trench installation or with imperfect trench installation refers to the highest earth fill height that roadway conduits can withstand before a 001-inch crack develops. The design of roadway conduits by using MFH tables is one of the most convenient indirect design methods.

MFH tables for roadway conduits are based on a soil weight of 120 lbs/ft^3 and embankment installation, which is the worst case vertical load conditions for rigid conduits, and which provide conservative results for other embankment and trench conditions. MFH with imperfect trench installation can be easily calculated from MFH without imperfect trench installation, Equation 6-25 and Table 6.4. The development and verification of MFH are based on Finite Element Analyses.

6.5.1 Concrete Roadway Pipe

MFH tables for concrete roadway pipe are presented in the Appendix C. The values given in the MFH tables show good agreement with those derived in the indirect design process described in Section 6.2 and the MFH published by ACPA [38].

6.5.2 Box Culverts

Generally, MFH tables are not available for box culverts. Earth loads determined from design method described in Section 6.2 can be used to determine MFH. MFH with imperfect trench installation of roadway conduits can then be easily calculated from MFH, Equation 6-25, and Table 6.5.

6.6 Recommended Finite Element Analysis Programs

6.6.1 SPIDA

The finite element computer program SPIDA (Soil-Pipe Interaction Design and Analysis) was developed for the analysis and design of buried reinforced precast concrete pipe through a long-range research project funded by ACPA in the early 1980s [50]. Using the results of numerous SPIDA parameter studies, a set of AASHTO Standard Installations were developed. The SPIDA studies were conducted for positive projection embankment conditions, which are the worst case vertical load conditions for pipe. The design method used in SPIDA is in accordance with the design procedures given in the AASHTO Specifications [6].

These results provide the basis for the "Direct Design Method" based on a soil-structure interaction analysis of the pipe-soil system and the reinforced concrete design procedures given under the Direct Design Method based on Heger pressure distribution given in the AASHTO Specifications [6]. The results obtained using SPIDA show good agreement with those found using the indirect design process given in Section 6.2 and the MFH values published by ACPA. Typical SPIDA input and output files are shown in the Appendix A.

6.6.2 CANDE-89

CANDE, an acronym derived from Culvert ANalysis and Design, is a computer program that is used for the structural analysis, design and evaluation of buried culverts and other soil-structure systems.[28] For example, buried structures made of corrugated metal, reinforced concrete, or structural plastic may be analyzed and designed to withstand incremental soil loading, temporary construction loads, and surface loads due to vehicular traffic. Since its introduction in 1976, the CANDE program has been widely distributed by the Federal Highway Administration (FHWA). CANDE is a well-established and well-accepted tool for the design and analysis of all types and sizes of culverts used in highway construction. Typical CANDE input and output files are shown in the Appendix B.

CHAPTER 7
Construction Procedures

7.1 General

Construction is the final step in a process that includes research, investigations, design, specification preparation, pipe manufacturing and material testing. This chapter presents construction procedures that conform to the requirements of the Special Provisions and design guides in Chapters 5 and 6. The soil-structure makes it difficult to separate construction practice from design practice. The soil-structure used in the design process assumes that certain minimum conditions of installation will be met. In this chapter, Acceptance criteria and considerations are presented to assure that the workmanship and material quality provided during construction practice meet the design requirements. There are many important steps that must be taken to achieve a quality buried pipe installation. These procedures are detailed below:

7.2 Construction Procedures

7.2.1 Planning

Adequate planning can identify and eliminate many of the potential problems that may produce unnecessary delays and extra costs. Pre-construction planning assists in the development of rapport among engineers, inspectors, material suppliers and construction personnel. Pre-construction planning should be preceded by a review of all the construction contract documents, including plans, project specifications, soil information, standard drawings and special provisions. During the field check, any questions concerning the plans and specifications can be resolved. Planning also requires all involved personnel to be familiar with the administrative requirements of the construction contract, such as wage rates, insurance requirements, change order procedures, safety regulations, etc.

7.2.2 Site Preparation

Site preparation can significantly influence progress on the project. The amount and type of work involved in site preparation varies with the location of the project, topography, surface conditions and existing utilities. Factor that should be considered include:

- Top soil stripping
- Clearing and grubbing
- Pavement and sidewalk removal
- Rough grading
- Relocation of existing natural drainage
- Removal of unsuitable soil material
- Access roads
- Detours
- Protection of existing structures and utilities
- Environmental issues

7.2.3 Excavation

7.2.3.1 General

Excavation must conform to Section 210 of the ALDOT Specifications, 2002 [67] except as provided otherwise in this section. Excavation requires the removal of all the material that is not to be left in place that is encountered within the limits of the work. When ledge rock, compacted rocky, or other unyielding foundation material is encountered, it must be removed at least to the requirements shown in Figures 6.2 and 6.3 and Tables 6.1 and 6.2. For sewers and culverts, excavation can include trenching, tunneling, backfilling, embankment construction, soil stabilization and control of groundwater and surface drainage. Adequate knowledge of the subsurface conditions is essential for any type of excavation. Over-excavated areas shall be backfilled with approved materials and compacted to at least the Standard Proctor density specified for the bedding. Where surface water or groundwater conditions exist, the site and trench shall be dewatered.

7.2.3.2 Equipment

Several types of excavating equipment are available. Selection of the most efficient piece of equipment for a specific excavation operation is important, since all excavating equipment has practical and economic limitations. Considerations include the type and amount of material to be excavated, depth and width of excavation, dimensional limitations established in the plans, pipe size, operating space and spoil placement. The basic equipment often used as follows:

- Backhoe
- Bulldozer
- Clamshell
- Dragline
- Crawler-Mounted
- Front-End Loaders
- Wheel-Mounted
- Hydraulic Excavator
- Scraper
- Trencher

7.2.3.3 Line and Grade

For trench installation, line and grade are usually established by control points consisting of stakes, spikes, plugs or shiners set at the ground surface and offset from the proposed centerline of the pipe, and control points set in the trench. The basic procedures include:

1) Stakes, spikes, plugs or shiners are driven flush with the ground surface, at 25 to 50 foot intervals for straight alignment and at shorter intervals for curved alignment.
2) The control point is offset 10 feet, or some other convenient distance, on the opposite side of the trench from which excavated material will be placed.

3) Control point elevations are determined, and the depth from the control point to the trench bottom or pipe invert is indicated on a guard stake next to the control point.

4) A cut sheet is prepared listing reference points, stationing, offset distance and vertical distance from the control points to the trench bottom or pipe invert.

7.2.3.4 Excavation Limits

Excavation, pipe installation and backfill operations should succeed each other as rapidly as possible. The most important excavation limitations are trench width and depth. As excavation progresses, for example in a sewer line, trench grades are continuously checked to obtain the elevations established for the sewer profile. Incorrect trench depths may adversely affect the hydraulic capacity of the sewer and require correction or additional maintenance after the line is completed.

The backfill load ultimately transmitted to the pipe is a function of the trench width. The designer assumes a certain trench width in determining the backfill load, and selects a pipe strength capable of withstanding that load. If the actual trench width exceeds the width assumed in the design, the load on the pipe will be greater than estimated and structural distress may result. Therefore, trench widths should be as narrow as established in the plans or standard drawings. If an excessively wide trench is excavated or the sides sloped back, the pipe can be installed in a narrow subtrench excavated at the bottom of the wider trench to avoid any increase in the backfill load. The recommended depth of the subtrench is the vertical height of the pipe plus one foot.

For culverts installed under embankments, it may be possible to simulate a narrow subtrench by installing the pipe in an existing stream bed. When culverts are installed in a negative projection condition or using the induced trench method of construction, the same excavation limits apply as for trench conditions. For jacked or tunneled installations, the excavation should coincide as closely as possible to the outside dimensions and shape of the pipe. The usual procedure for jacking pipe is to equip the leading edge with a cutter, or shoe, to protect the lead pipe. As the pipe is jacked forward, soil is excavated and removed through the pipe. Materials should be trimmed approximately one or two inches larger than the outside diameter of the pipe and excavation should not precede pipe advancement more than necessary. This procedure results in minimum disturbance of the earth adjacent to the pipe.

7.2.3.5 Spoil Placement

The placement and storage of excavated material is an important consideration in sewer and culvert construction and influences the selection of excavating equipment, the need for providing sheathing and shoring, and backfilling operations. For trench installations, the excavated material is usually used for backfill, and the material stockpiled along the trench in such a manner as to reduce unnecessary handling during backfill operations, Figure 7.1.

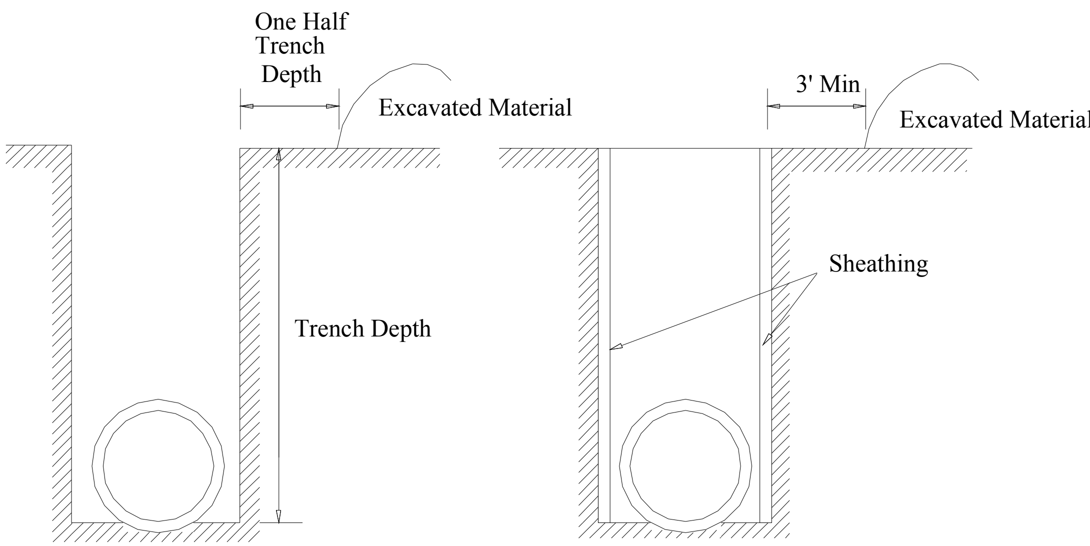

Figure 7.1 Spoil Placement

7.2.3.6 Sheathing and Shoring

Trench stabilization is usually accomplished through the use of sheathing and shoring. The structural requirements for sheathing and shoring depend on:

- Depth and width of excavation
- Characteristics of the soil
- Water content of the soil
- Water table
- Weather conditions
- Proximity of other structures

- Vibration from construction equipment and traffic
- Spoil placement or other surcharge loads
- Code requirements

7.2.3.7 Dewatering

Groundwater conditions should be investigated prior to excavation. Test borings may be required to determine the depth, quantity and direction of flow of the water table. Groundwater is usually controlled by one or a combination of the following:

- Tight sheathing
- Drains
- Pumping
- Wellpoints

7.2.4 Foundation

The foundation shall be moderately firm to hard in situ soil, stabilized soil, or compacted fill material. A stable and uniform foundation is necessary for satisfactory performance of any pipe. The foundation must have sufficient load bearing capacity to maintain the pipe in proper alignment and sustain the loads imposed. When unsuitable or unstable material is encountered, the foundation shall be stabilized by ballasting or soil modification. Ballasting requires the removal of undesirable foundation material and replacement with selected materials, such as sand, gravel, crushed rock, slag, or suitable earth backfill.

Soil modification involves the addition of select material to the native soil. Crushed rock, gravel, sand, slag or other durable inert materials with a maximum size of three inches is worked into the subsoil to accomplish the required stabilization. Soil modification can also be accomplished by the addition of lime, cement or chemicals to the soil. Where groundwater and soil characteristics may contribute to the migration of soil fines into or out of foundation, bedding, sidefill, and backfill materials, methods to prevent migration shall be provided. Pipe installed over an unyielding foundation shall be cushioned so as to prevent blasting shock if blasting is anticipated in the area of some time in the future.

7.2.5 Bedding

An important function of the bedding is to assure uniform support along the barrel of each pipe section. The bedding distributes the load reaction around the lower periphery of the pipe. The bedding shall be constructed as required by Proposed Installations in Figures 4.2, 4.3, 13, 4.25, 4.26, and 4.34 uniformly over the full length of the pipe barrel, to distribute the load-bearing reaction uniformly on the pipe barrel over its full length, and to maintain the required pipe grade. The bedding layers for TREATED installation shall be placed to be as uniform as possible, but shall be loosely placed uncompacted material under the middle third of the pipe prior to placement of the pipe. The maximum aggregate size for beddings shall not be greater than 1 in. (25mm) except if the bedding has a thickness of 6 in. (150mm) or greater, the maximum aggregate size shall not be greater than 1-1/2 in. (38mm).

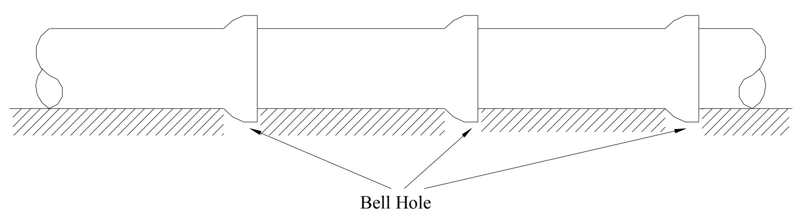

Figure 7.2 Uniform Pipe Support

Bell holes shall be excavated in the bedding and/or foundation when installing pipe with expanded bells so that the pipe is supported by the barrel in Figure 7.2 and not by the bells. Any outer bedding under the lower side areas shall be compacted to at least the same requirements as for the lower side areas.

7.2.6 Pipe Placement and Joining

7.2.6.1 General

Pipe shall be installed to the line and grade shown on the project plans and specifications. Joining shall be in accordance with the pipe manufacturer's recommendations. Where practical, work should be started at the lowest end of the pipeline and the pipe laid ith the bell end upgrade. The top of the pip section being laid shall be positioned to the correct orientation (if required) and then joined. Adjustments in grade by exerting force on the barrel of the pipe with excavating equipment or by lifting and dropping the pipe shall be prohibited. If the installed pipe section is not on grade, the pipe section shall be completely unjoined, the grade corrected, and the pipe then rejoined.

7.2.6.2 Joint Gaskets and Sealants

Gaskets and sealants vary in cost and inherent performance characteristics, but field performance is always dependent upon installation procedures. The most common are:

- Rubber, attached or separate
- Mastic, bulk or preformed
- Cement, paste or mortar
- External bans, cement mortar or rubber

7.2.6.3 Jointing Procedures

When jointing pipe sizes up to 24 inches in diameter, the axis of the pipe section to be installed should be aligned as closely as possible the axis of the last installed pipe section, and the tongue or spigot end inserted slightly into the bell or groove. A bar is then driven into the bedding and wedged against the bottom bell or groove end of the pipe section being installed. A wood block is placed horizontally across the end of the pipe to act as a fulcrum point and to protect the joint end during assembly. By pushing the top of the vertical bar forward, lever action pushes the pipe into a home position.

When jointing larger diameter pipe, mechanical pipe pullers are required. Several types of pipe pullers or come along devices have been developed. Large diameter pipe can be jointed by placing a dead man blocking inside the installed pipe, several sections back from the last installed section, which is connected by means of a chain or cable to a strong back placed across the end of the pipe section being installed. When jointing small diameter pipe, a chain or cable is wrapped around the barrel of the pipe a few feet behind the tongue or spigot and fastened with a grab hook or other suitable connecting device. A lever assembly is anchored to the installed pipe, several sections back from the last installed section, and connected by means of a chain or cable to the grab hook on the pipe to be installed. By pulling the lever back, the tongue or spigot of the pipe being jointed is pulled into the bell or groove of the last installed pipe section. To maintain close control over the alignment of the pipe, a laying sling can be used to lift the pipe section slightly off the bedding foundation.

7.2.6.4 Service Connections

When the pipe is connected to a rigid structure, such as a building, manhole or junction chamber, the bedding and foundation for the pipe must be highly compacted to minimize differential settlement.

7.2.7 Haunch

The haunch shall be constructed using the specified soil type and the minimum compaction level required for Proposed Installations in Figures 4.2, 4.3, 4.13, 4.25, 4.26, and 4.34. It shall be placed and compacted uniformly for the full length of the pipe barrel so as to distribute the load-bearing reaction uniformly to the bedding over the full length of the pipe barrel.

The maximum aggregate size for the haunch shall not be greater than 1 in. (25 mm) except if the bedding has a thickness of 6 in. (150 mm) or greater, the maximum aggregate size shall not be greater than 1-1/2 in. (38 mm).

7.2.8 Lower Side

The lower side zone shall be as specified in Proposedd Installations in Figures 4.2, 4.3, 4.13, 4.25, 4.26, and 4.34. The soil, if not in situ, shall be approved material containing no debris, organic matter, frozen material, or large stones with a diameter greater than one half the thickness of the compacted layers being placed. Any placed soil shall be deposited uniformly on each side of the pipe to prevent lateral displacement and compacted to the specified density.

7.2.9 Backfilling

7.2.9.1 General

The backfill consists of two zones with separate material and compaction criteria. The first zone extends from the bedding to a plane approximately 12 inches above the top of the pipe. The second zone includes all of the remaining fill.

7.2.9.2 Backfilling around Pipe

The load carrying capacity of an installed pipe is largely dependent on the initial backfilling around the pipe. Because of the importance of obtaining proper compaction of backfill material immediately around the pipe, material and density criteria are often included as part of the bedding requirements. For trench installations, where space is limited, tamping by pneumatic or mechanical impact tampers is usually the most effective means of compaction. Impact tampers which compact by static weight and kneading action are primarily useful for clay soils, while granular soils are most effectively consolidated by vibration. Where impact type tampers are used, caution should be exercised to prevent direct blows on the pipe. Backfill material should be compacted and brought up in even layers on both sides of the pipe.

7.2.9.3 Final backfilling

Once the backfill material is placed around the pipe and properly compacted, the remainder of the fill is placed and compacted to prevent settlement at the surface as specified. The soil shall be approved material containing no debris, organic matter, frozen material, or large stones with a diameter greater than one half the thickness of the compacted layers being placed. When impact or vibratory equipment is used for compaction, care shall be taken to avoid damaging the pipe. Several types of compaction equipment are available and certain types are best for particular soils. The steel wheeled roller is best suited for compacting coarse aggregate such as slag, coarse gravel and graded rock. The sheepsfoot roller is best suited for cohesive clays or silts, and is not suitable for use on granular soils. Rubber tired rollers, which provide static weight and kneading action, are effective for many soils. Vibratory rollers are effective for granular materials.

7.2.9.4 Imperfect Trench Installation

The imperfect trench installation shall be constructed as Figures 6.5 and 6.6 of Section 6.3 for concrete roadway pipe and Figure 6.9 and 6.10 of Section 6.3 for box culverts.

7.2.9.4.1 Soft Zone on the Top of Conduit (Geometry I)

The roadway conduits shall be placed and backfilled as specified in Sections 6.3.1 and 6.3.2. The fill shall be constructed to a height of a half the out-to-out vertical rise of pipe or box above the top of the conduit. The minimum distance of the fill on each side shall be constructed to at least 5 times the out-to-out horizontal span of the pipe or box in each direction from the outside edge of the conduit.

Next, a trench equal in width to the out-to-out horizontal span (plus 2 times the wall thickness in the case of box culverts) of the conduit shall be dug in the fill directly down to the top of the conduit. Care shall be exercised to keep the sides of this trench as nearly vertical as possible. The trenches shall then be refilled with soft materials as shown in Table 6.3 of Section 6.3.1.3. After this loose backfill is completed, the remainder of the fill up to subgrade elevation shall be constructed as specified in Section 6.4.

7.2.9.4.2 Soft Zone on the Top and Side of Conduit (Geometry II)

The roadway conduits shall be placed and backfilled as specified in Sections 6.3.1 and 6.3.2. The fill shall be constructed to a quarter of the out-to-out vertical rise of pipe or box above the top of the conduit. The minimum distance of the fill on each side shall be constructed to at least 5 times the out-to-out horizontal span of the pipe or box in each direction from the outside edge of the conduit.

Next, a trench equal in width to the out-to-out horizontal span plus the wall thickness (2 times the wall thickness in the case of box culverts) of the conduit shall be dug in the fill directly down to the springline for a pipe and to the bottom for box culverts. Care shall be exercised to keep the sides of this trench as nearly vertical as possible. The trenches shall then be refilled with soft materials as specified in Table 6.3 of Section 6.3.1.3. After this loose backfill is completed, the remainder of the fill up to subgrade elevation shall be constructed as specified in Section 6.4.

7.2.10 Sheathing Removal and Trench Shield Advancement

Unless the sheathing is to be left in place, it shall be pulled out in vertical increments to permit placement and compaction of fill material for the full width of the trench. When trench shields or boxes are moved, the previously placed pipe shall not be disturbed. It may be necessary to restrain the installed pipe by use of deadman anchors or other means. Voids in the sidefill that are created by the movement of a shield or box shall be filled and compacted.

7.2.11 Minimum Cover for Construction Loads

If the passage of construction equipment over an installed pipeline is necessary during project construction, compacted overfill in the form of a ramp shall be constructed to a minimum elevation of 3 ft (0.9m) over the top of the pipe or to a height such that the equipment loads on the pipe do not exceed the pipe design strength. In an embankment installation, the overfill shall extend a minimum of one pipe diameter width or 3ft (0.9m), whichever is greater, beyond each side of the pipe to prevent possible lateral displacement of the pipe. If a large volume of construction traffic must cross an installed pipe, the point of crossing shall be changed occasionally to minimize the possibility of lateral displacement.

7.3 Summary

The construction procedures are as follows:

Planning

⇩

Site Preparation

⇩

Excavation

⇩

Foundation

⇩

Bedding

⇩

Pipe Placement

⇩

Haunch

⇩

Lower Side

⇩

Backfilling

CHAPTER 8

Conclusions and Recommendations for Future Study

8.1 Conclusions

This report presents the results of advanced Finite Element Analyses for the soil-structure interactions that take place during the process of excavating a trench, preparing the bedding, installing the conduit, and then placing and compacting the backfill for concrete roadway pipe and box culverts. The materials and procedures used will significantly affect the conduit performance. The imposed loading is greatly affected by the relative settlement of the soil prism directly above the conduit. An improved understanding of these fundamentals will be essential to develop technically better and more economical specifications for both designers and contractors. The findings of this research are as follows:

1) The behavior of concrete roadway pipe and box culverts is more significantly affected by installation practices for the bedding and sidefill (haunch and lower side) than by foundation characteristics such as yielding or unyielding.

2) The highest earth load reduction due to imperfect trench installation occurs when the soft zone is placed immediately on top of the conduits. The earth load reduction rates have variations with a diminishing return characteristic as the ratio of the height of the soft zone to out-to-out horizontal span (or out-to-out vertical span), H_s/B_c (or H_s/H_c) and the ratio of the width of the soft zone to out-to-out horizontal span, W/B_c increase. The increase in the earth load reduction rate slows down after H_s/B_c (or H_s/H_c) =0.5 and W/B_c =1.5.

3) Optimum geometries for the soft zones of an imperfect trench installation were developed by numerous parameter studies for the height, the distance from the top of the conduit, and the width of the soft zone. Optimum geometries of the soft zone were proposed as Geometry I and Geometry II for concrete pipe and box culverts. The upper half of the pipe is surrounded by soft material in Geometry II for pipes and in Geometry II for box culverts, the whole sidewall is

surrounded by soft material. Geometry II is more effective than Geometry I for earth load reduction. Therefore, only Geometry II was used in the Special Provisions and Special Highway Drawings of ALDOT Specifications, 2002.

<u>Concrete roadway pipe</u>

Geometry I : $H'/B_c=0$, $Hs/B_c=0.50$, $W/B_c=1.0$

Geometry II (Proposed Geometry of Soft Zone): $H'/B_c=0$, $Hs/B_c=0.25$, $W_s=B_c+t$

<u>Box culverts</u>

Geometry I : $H'/H_c=0$, $Hs/H_c=0.50$, $W=B_c+2.0t$

Geometry II (Proposed Geometry of Soft Zone) : $H'/H_c=0$, $Hs/H_c=0.25$, $W_s=B_c+2.0t$

Where H'=distance between the top of the conduit and the bottom of the soft zone (ft); B_c= out-to-out horizontal span of the conduit (ft); Hs:= height of soft zone (ft); W:=width of soft zone in Geometry I (ft); W_s= width of soft zone in Geometry II (ft); t:= wall thickness (ft); H_c:= out-to-out vertical span of conduit (ft).

4) The mechanical properties of soft materials based on the test results were considered in this study. The modulus of elasticity of soft materials ranged from 50 psi to 400 psi. The Poisson's ratio for the soft materials was close to zero.

5) Reduction rates are highly affected by the modulus of elasticity (E_s) of the soft materials, which means the reduction rate is a function of E_s. Based on the results of numerous parameter studies with Finite Element programs, proposed equations for the reduction rates were derived for both concrete roadway pipe and box culverts by means of a linear regression method.

6) The total vertical earth loads or bottom loads that act on box culverts are composed of the top earth load, dead load, and shear force on the sidewall. Therefore, the design loading of box

culverts should be based on the bottom pressure. AASHTO provisions for the design loading of box culverts are unconservative, as AASHTO provisions do not consider the shear force on the sidewall.

7) TREATED sidefill for the box culverts is an effective way to reduce the shear force occurring on the sidewall of box culverts. AASHTO has no provisions for the requirements and geometry of sidefill for box culverts. For the convenience of installation and design, installations for box culverts were proposed that are similar to those for pipes in this study.

8) The use of imperfect trench installations for box culverts increases the shear force on the sidewall due to the reverse arching effect of imperfect trench installation as well as the reduction of top earth pressure and total vertical earth load. A method to prevent this increase of shear force was developed in order to maximize the effect of imperfect trench installation. The increased shear force can be reduced by installing soft material between the sidewall and soil. Using Geometry II for the soft zone of box culverts is highly effective in relieving the increased effect of downward shear force on the sidewall.

8.2 Recommendations for Future Study

Further studies are recommended in the following areas:

1) Whether the pipe is classified to be flexible or rigid is dependent upon the pipe stiffness relative to the stiffness of the surrounding soil [62]. If the pipe is stiffer than the soil, then the pipe is considered rigid. If the soil is stiffer than the pipe, then the pipe is considered flexible. With recent improvements in material and production technology, the range of available flexible plastic pipe such as plastic pipes and corrugated metallic pipe, has increased considerably, and a more thorough means evaluating the structural aspects of buried flexible pipeline thus demands more attention. Since a flexible pipe has very little inherent strength, the backfill plays a vital role in the overall response of the pipe-soil system. Research on flexible pipes should aim to investigate the overall interaction of pipe wall stress and strain, pipe deflections, and the buckling capacity of

flexible pipes. In the next phase of this research program, a concentrated effort will be made to characterize the behavior of these flexible pipes.

2) Experimental field studies of buried rigid conduits would be desirable in order to validate the results of the Finite Element Analyses presented in this study. In addition, experiments on selected samples of soft material may be necessary to augment the properties obtained from the literature.

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APPENDIX A

Typical Input for SPIDA

Example 1

Conduit	Installation type	Sidefill
Concrete Round Pipe	Embankment	TREATED

Input filename: spidain_ex1, Output filename: spidaout_ex1 (Electric files are given in Appendix G)

```
*****************************************************************************************************************************

TITLE   EMBANKMENT
TITLE   PIPE 72 INCH  WALL B
TITLE    FILL HEIGHT 32 FT COVER
TITLE    TREATED SIDEFILL
TITLE   CIRCULAR PLUS ELLIPTICAL CAGE
PIPE          72.0    7.0
CAGEC       0.48   0.37    1.0    1.0    2.0    2.0    3
CAGEE       0.51    1.0     1.0    2.0      3   12.5
MATERIAL  65.0    6.0
INSTALL       0   32.0          12.0   12.0
MESH       0.15   0.17   0.17   0.83
ZONES         1     11      1
ZONES        18     25      0
ZONES        19     25      0
ZONES        20     25      0
ZONES        21     25      0
ZONES        22     25      0
ZONES        23     25      0
ZONES
PROPERTY     31      3      2
```

```
PROPERTY     21      25       2

PROPERTY     33      25       5

PROPERTY     23      25       5

PROPERTY     41      25       7

PROPERTY     51      25       3

PROPERTY     57      25       2

PROPERTY     62      25       2

PROPERTY     67      25       2

PROPERTY     72      25       2

PROPERTY     28      25       2

PROPERTY     38      25       2

PROPERTY     48      25       2

PROPERTY     54      25       2

PROPERTY     59      25       2

PROPERTY     64      25       2

PROPERTY     69      25       2

PROPERTY     74      25       2

PROPERTY    125      25       6

PROPERTY    135      25       6

PROPERTY    145      25       6

PROPERTY    155      25       6

PROPERTY    165      25       6

SOIL          4 0.06940   200.0    0.26    0.89    18.4    0.10     3.5    32.0

            0.0     0.9

SOIL         25 0.06940   120.0    0.45    1.00    21.1    0.13     9.0    15.0

            4.0     0.8

SOIL         22 0.0694    450.0    0.35    0.80    12.7    0.08     0.0    38.0

            2.0     0.9
```

```
SOIL         1 0.0694   640.0   0.43   0.75   40.9   0.05    0.0   42.0
           4.0    1.1
SOIL        21 0.0694   950.0   0.60   0.70   74.8   0.02    0.0   48.0
           8.0    1.3
SOIL        23 0.0694   440.0   0.40   0.95   48.3   0.06    4.0   34.0
           0.0    1.2
SOIL        26 0.0694    50.0   0.60   0.90   13.0   0.15    6.0   18.0
           8.0    0.5
SOIL        29 0.0057    35.0   0.55   0.70   1.00   0.50    0.0   44.0
           1.0    0.7
SOIL        28 0.0009     5.0   0.08   0.70   0.12   1.0     0.0   44.0
           1.0    0.7
FACTORS     1.0    1.0    1.0   1.00   1.00   1.00    1.0    1.0    1.0
PRINTB        1   32.0
PRINT        1      1
END
```

Example 2

Conduit	Installation type	Sidefill
Concrete Round Pipe	Imperfect Trench, Geometry I	TREATED

Input filename: spidain_ex2, Output filename: spidaout_ex2 (Electric files are given in Appendix G)

****

```
TITLE   IMPERFECT TRENCH
TITLE   72 INCH WALL B
TITLE    FILL HEIGHT 32 FT COVER
TITLE    UNTREATED sidefill
TITLE   CIRCULAR PLUS ELLIPTICAL CAGE
PIPE       72.0     7.0
CAGEC      0.48    0.37     1.0     1.0     2.0     2.0     3
CAGEE      0.51     1.0     1.0     2.0       3    12.5
MATERIAL    65.0     6.0
INSTALL       0    51.0           12.0    12.0
MESH       0.25    0.17    0.17    0.83
ZONES        1      11       1
ZONES       18      25       0
ZONES       19      25       0
ZONES       20      25       0
ZONES       21      25       0
ZONES       22      25       0
ZONES       23      25       0
ZONES
PROPERTY     31       3       2
PROPERTY     21      25       2
PROPERTY     33      25       5
```

```
PROPERTY    23    25    5
PROPERTY    41    25    7
PROPERTY    51    25    3
PROPERTY    57    25    2
PROPERTY    62    25    2
PROPERTY    67    25    2
PROPERTY    72    25    2
PROPERTY    28    25    2
PROPERTY    38    25    2
PROPERTY    48    25    2
PROPERTY    54    25    2
PROPERTY    59    25    2
PROPERTY    64    25    2
PROPERTY    69    25    2
PROPERTY    74    25    2
PROPERTY   105    29    6
PROPERTY   115    29    6
PROPERTY   125    29    6
PROPERTY   135    25    6
PROPERTY   145    25    6
PROPERTY   155    25    6
PROPERTY   165    25    6
SOIL        4 0.06940   200.0   0.26   0.89   18.4   0.10    3.5   32.0
        0.0   0.9
SOIL       25 0.06940   120.0   0.45   1.00   21.1   0.13    9.0   15.0
        4.0   0.8
SOIL       22 0.0694    450.0   0.35   0.80   12.7   0.08    0.0   38.0
        2.0   0.9
```

```
SOIL        1 0.0694    640.0    0.43    0.75    40.9    0.05     0.0    42.0
         4.0     1.1
SOIL       21 0.0694    950.0    0.60    0.70    74.8    0.02     0.0    48.0
         8.0     1.3
SOIL       23 0.0694    440.0    0.40    0.95    48.3    0.06     4.0    34.0
         0.0     1.2
SOIL       26 0.0694     50.0    0.60    0.90    13.0    0.15     6.0    18.0
         8.0     0.5
SOIL       29 0.000520   35.0    0.55    0.70    1.00    0.50     0.0    44.0
         1.0     0.7
SOIL       28 0.0009      5.0    0.08    0.70    0.12    1.0      0.0    44.0
         1.0     0.7
FACTORS     1.0     1.0     1.0    1.00    1.00    1.00     1.0     1.0     1.0
PRINTB        1    51.0
PRINT        1      1
END
```

Example 3

Conduit	Installation type	Sidefill
Concrete Round Pipe	Embankment	UNTREATED

Input filename: spidain_ex3, Output filename: spidaout_ex3 (Electric files are given in Appendix G)

****

```
TITLE   EMBANKMENT
TITLE   72 INCH WALL B
TITLE    21 FT COVER
TITLE    UNTREATED sidefill
TITLE   CIRCULAR PLUS ELLIPTICAL CAGE
PIPE       72.0     7.0
CAGEC      0.48    0.37     1.0     1.0     2.0     2.0     3
CAGEE      0.51     1.0     1.0     2.0       3    12.5
MATERIAL    65.0     6.0
INSTALL       0    21.0           12.0    12.0
MESH       0.17    0.17    0.17    0.83
ZONES         1      11       1
ZONES        18      25       0
ZONES        19      25       0
ZONES        20      25       0
ZONES        21      25       0
ZONES        22      25       0
ZONES        23      25       0
ZONES
PROPERTY     31      11       2
PROPERTY     21      11       2
PROPERTY     33      11       8
```

```
PROPERTY    23    11    8
PROPERTY    41    11    1
PROPERTY    42    26    6
PROPERTY    51    26    3
PROPERTY    57    26    2
PROPERTY    62    26    2
PROPERTY    67    26    2
PROPERTY    72    26    2
PROPERTY    28    26    2
PROPERTY    38    26    2
PROPERTY    48    26    2
PROPERTY    54    26    2
PROPERTY    59    26    2
PROPERTY    64    26    2
PROPERTY    69    26    2
PROPERTY    74    26    2
PROPERTY   125    25    6
PROPERTY   135    25    6
PROPERTY   145    25    6
PROPERTY   155    25    6
PROPERTY   165    25    6
SOIL        29 0.07233   170.0   0.37   1.07   32.5   0.09   11.0   12.0
            0.0   1.0
SOIL         4 0.06940   200.0   0.26   0.89   18.4   0.10    3.5   32.0
            0.0   0.9
SOIL        25 0.06940   120.0   0.45   1.00   21.1   0.13    9.0   15.0
            4.0   0.8
SOIL        22 0.0694    450.0   0.35   0.80   12.7   0.08    0.0   38.0
```

```
           2.0     0.9
SOIL          1 0.0694    640.0    0.43    0.75    40.9    0.05     0.0    42.0
           4.0     1.1
SOIL         21 0.0694    950.0    0.60    0.70    74.8    0.02     0.0    48.0
           8.0     1.3
SOIL         23 0.0694    440.0    0.40    0.95    48.3    0.06     4.0    34.0
           0.0     1.2
SOIL         26 0.0694     50.0    0.60    0.90    13.0    0.15     6.0    18.0
           8.0     0.5
FACTORS      1.0     1.0     1.0    1.00    1.00    1.00     1.0     1.0     1.0
PRINTB        1    21.0
PRINT        1       1
END
```

Example 4

Conduit	Installation type	Sidefill
Concrete Round Pipe	Imperfect Trench, Geometry Ii	UNTREATED

Input filename: spidain_ex4, Output filename: spidaout_ex4 (Electric files are given in Appendix G)

```
************************************************************************************************************************

TITLE   IMPERFECT TRENCH, GEOMETRY II
TITLE   72 INCH WALL B
TITLE    21 FT COVER
TITLE    UNTREATED sidefill
TITLE   CIRCULAR PLUS ELLIPTICAL CAGE
PIPE       72.0    7.0
CAGEC      0.48    0.37    1.0    1.0    2.0    2.0    3
CAGEE      0.51    1.0    1.0    2.0    3    12.5
MATERIAL    65.0    6.0
INSTALL       0    21.0          12.0    12.0
MESH       0.25   0.042   0.083    1.50
ZONES        1      11      1
ZONES       18      25      0
ZONES       19      25      0
ZONES       20      25      0
ZONES       21      25      0
ZONES       22      25      0
ZONES       23      25      0
ZONES
PROPERTY    31      11      2
PROPERTY    21      11      2
PROPERTY    33      11      8
```

```
PROPERTY    23    11    8
PROPERTY    41    26    7
PROPERTY    51    26    3
PROPERTY    57    26    2
PROPERTY    62    26    2
PROPERTY    67    26    2
PROPERTY    72    26    2
PROPERTY    28    26    2
PROPERTY    38    26    2
PROPERTY    48    26    2
PROPERTY    54    26    2
PROPERTY    59    26    2
PROPERTY    64    26    2
PROPERTY    69    26    2
PROPERTY    77    29    2
PROPERTY    82    29    2
PROPERTY    87    29    2
PROPERTY    92    29    3
PROPERTY    98    29    4
PROPERTY    105    29    7
PROPERTY    115    29    7
PROPERTY    125    25    6
PROPERTY    135    25    6
PROPERTY    145    25    6
PROPERTY    155    25    6
PROPERTY    165    25    6
SOIL        29 0.07233   170.0    0.37    1.07    32.5    0.09    11.0    12.0
        0.0    1.0
```

```
SOIL      4 0.06940  200.0   0.26   0.89   18.4   0.10    3.5   32.0
        0.0    0.9
SOIL     25 0.06940  120.0   0.45   1.00   21.1   0.13    9.0   15.0
        4.0    0.8
SOIL     22 0.0694   450.0   0.35   0.80   12.7   0.08    0.0   38.0
        2.0    0.9
SOIL      1 0.0694   640.0   0.43   0.75   40.9   0.05    0.0   42.0
        4.0    1.1
SOIL     21 0.0694   950.0   0.60   0.70   74.8   0.02    0.0   48.0
        8.0    1.3
SOIL     23 0.0694   440.0   0.40   0.95   48.3   0.06    4.0   34.0
        0.0    1.2
SOIL     26 0.0694    50.0   0.60   0.90   13.0   0.15    6.0   18.0
        8.0    0.5
FACTORS    1.0    1.0    1.0   1.00   1.00   1.00    1.0    1.0    1.0
PRINTB      1   21.0
PRINT      1     1
END
```

APPENDIX B

Typical Input for CANDE-89

Example 1

Conduit	Installation type	Foundation	Sidefill
Box Culverts	Embankment	Yielding	TREATED

Input filename: candein_ex1, Output filename: candeout_ex1 (Electric files are given in Appendix G)

```
*******************************************************************************************************************

ANALYS 2 CONCRE BOX CULVERTS 7FT TIMES 7FT 8IN                          2

-1.0     8.0      STD  3    0.0001

5000.0    3600000.0 0.25      145.0

8.0       8.0       8.0       8.0       8.0

0.018     0.034     0.035     0.015     0.565     1.0

EMBA     EMBANKMENT

2    2    3    10   46.0      46.0      18.0      120.0              8.0

 1   3    130.0     INSITU (NO 11)

    1.0       1

100.0     50.0      0.01      350.0     0.01      0.01

453.0     0.014

 2   3    120.0     BEDDING (NO 25)

    1.0       1

9.0       15.0      4.0       120.0     0.45      1.0

53.0      0.092

L3   3    120.0     FILL (NO 25)

    0.5       1

9.0       15.0      4.0       120.0     0.45      1.0

21.0      0.13

STOP
```

Example 2

Conduit	Installation type	Foundation	Sidefill
Box Culverts	Imperfect Trench, Geometry I	Yielding	TREATED

Input filename: candein_ex2, Output filename: candeout_ex2 (Electric files are given in Appendix G)

****

```
ANALYS 2 CONCRE BOX CULVERTS 7FT TIMES 7FT 8IN                    2
-1.0    8.0     STD  3    0.0001
5000.0   3600000.0 0.25     145.0
8.0     8.0     8.0     8.0     8.0
0.018    0.034    0.035    0.015    0.565    1.0
EMBA    IMPERFECT TRENCH                                     .            MOD
2   2   3   10   46.0     46.0     18.0     120.0          8.0
0   8   0
111              4
112              4
113              4
114              4
121              4
122              4
123              4
124              4
 1   3    130.0    INSITU (NO 11)
   1.0      1
100.0    50.0     0.01     350.0    0.01     0.01
453.0    0.014
 2   3    120.0    BEDDING (NO 25)
   1.0      1
```

```
9.0      15.0     4.0      120.0     0.45      1.0
53.0      0.092
 3   3    120.0     FILL (NO 25)
    0.5       1
9.0      15.0     4.0      120.0     0.45      1.0
21.0      0.13
L4   3    10.0      SOFT (NO 29)
    0.5       1
0.0      44.0     1.0      35.0      0.1       1.0
1.0      0.5
STOP
```

APPENDIX C

Maximum Fill Height Tables

Table C.1 Maximum Fill Heights without Imperfect Trench Installation for Concrete Round Pipes

Unit:ft

Wall B	TREATED SIDEFILL				UNTREATED SIDEFILLL			
	CLASS OF PIPE				CLASS OF PIPE			
Pipe Inside Diameter (IN.)	II	III	IV	V	II	III	IV	V
12	11	15	22	33	5	7	12	20
24	11	15	22	33	6	8	13	21
36	10	14	21	33	6	8	13	21
60	10	14	21	32	6	8	13	21
72	9	13	20	32	6	8	13	21
84	9	13	20	32	6	8	13	21
108	9	13	20	32	6	8	13	21
132	9	13	20	32	6	8	13	21
144	9	13	20	32	6	8	13	21

Note: II, III, IV, and V are classes of pipe. The corresponding strength requirements are presented in AASHTO M170

Table C.2 Maximum Fill Heights with Imperfect Trench Installation for Concrete Round Pipes

Unit:ft

Wall B	TREATED SIDEFILL				UNTREATED SIDEFILL			
	CLASS OF PIPE				CLASS OF PIPE			
Pipe Inside Diameter (IN.)	II	III	IV	V	II	III	IV	V
12	30	40	59	89	12	16	28	46
24	27	38	57	89	14	18	30	49
36	27	38	57	89	14	18	30	49
60	27	38	57	89	14	18	30	49
72	24	35	54	86	14	18	30	49
84	24	35	54	86	14	18	30	49
108	24	35	54	86	14	18	30	49
132	24	35	54	86	14	18	30	49
144	24	35	54	86	14	18	30	49

Note: EPS-Geofoam is used as the soft material (Es = 100psi)

APPENDIX D

Survey Results

Neighboring States' Procedures for Construction Practice on Buried Concrete Pipes

A. Questionnaires

Symbols

E: Positive Projection Embankment Installation

N: Negative Projection Embankment Installation

T: Trench Installation

I: Imperfect Trench Installation

J: Jacked or Tunneled Soil Load

1. Do your state's design specifications for buried culverts include the following installation method? If yes, how many standard bedding and backfill types (as classified in AASHTO Standard Specifications for Highway Bridges: Division II, Sec.27) are permitted for each installation type?

	N	Y	Number of Types
Positive Projection Embankment Installation (E)	☐	☐	( )
Negative Projection Embankment Installation (N)	☐	☐	( )
Trench Installation (T)	☐	☐	( )
Imperfect Trench Installation (I)	☐	☐	( )
Jacked or Tunneled Soil Load (J)	☐	☐	( )
Any others:			

2. Which design criteria are adopted or referred to for the design of each installation type stipulated in your state's specifications for buried concrete culvert systems?

	E	N	T	I	J
AASHTO Direct Design Methods (SIDD)	☐	☐	☐	☐	☐
AASHTO Indirect Design Methods	☐	☐	☐	☐	☐
Marston/Spangler design procedures	☐	☐	☐	☐	☐
State's own procedures (in-house)	☐	☐	☐	☐	☐
Others:					

3. If the Direct Design Method is used, what computer software or approximate methods are used for soil-structure interaction analyses?

4. Which procedure is used to determine the earth load and live load transmitted to culvert structures in each installation type?

	E	N	T	I	J
Heger Pressure Distribution	☐	☐	☐	☐	☐
ACPA Concrete Pipe Design Manual	☐	☐	☐	☐	☐
Marston/Spangler design criteria	☐	☐	☐	☐	☐
State's own procedures (in-house)	☐	☐	☐	☐	☐
Others					

5. Which tests are required by your state's specifications to ensure an acceptable level of quality control in workmanship and materials during construction?

Soil Density

Line and Grade

Visual Inspection

Infiltration

Exfiltration

Air Testing

Vacuum Testing

Joint Testing Air

6. If the Imperfect Trench Installation method is used, do your state′s specifications include provisions for the size and location of the “soft” material zone relative to the concrete pipe?

7. What are your state’s specifications for the “soft” material used in the Imperfect Trench Installation?

B. Survey Results

5 states' results are shown in the following table. Most states allow not only their own design criteria (in-house) but also old Marston and Spangler's theory in 1930's.

Item	Arkansas	Georgia	Tennessee	North Carolina	Mississippi
Installation (No. of Types)	E(3), T(3)	T(1), I(1), J(1)	E(1), N(1), T(1), J(1)	T(2)	T(?),I(?),J(?)
Design Criteria	State's Own	Marston and Spangler	Marston and Spangler	State's Own	AASHTO Indirect Design
Imperfect Trench	N/A	State's Own	N/A	N/A	State's Own

APPENDIX E

Properties of Soils and Soft Materials

Table E.1 Soil Properties for Constructed Soil (Placed Backfill) [39]

Soil No.	Soil Type	Proctor Compaction Std. T 99 %	Proctor Compaction Mod. T 180 %	γ_m pcf	K	n	R_f	B_i/P_a	e_u	C psi	ϕ deg	$\Delta\phi$ deg	K_o
27	SW	100	95	148	1300	0.90	0.65	108.8	0.01	0	54	15	1.5
21		95	90	141	950	0.60	0.70	74.8	0.02	0	48	8	1.3
1		90	85	134	640	0.43	0.75	40.8	0.05	0	42	4	1.1
22		85	80	126	450	0.35	0.80	12.7	0.08	0	38	2	0.9
2		80	75	119	320	0.35	0.83	6.1	0.11	0	36	1	0.8
3		80	60	91	54	0.85	0.90	1.7	0.23	0	29	0	0.5
28	ML	100	95	134	800	0.54	1.02	79.0	0.03	5.5	36	0	1.5
23		95	90	127	440	0.40	0.95	48.3	0.06	4	34	0	1.2
4		90	85	120	200	0.26	0.89	18.4	0.10	3.5	32	0	0.9
24		85	80	114	110	0.25	0.85	9.5	0.14	3	30	0	0.8
5		80	75	107	75	0.25	0.80	5.1	0.19	2.5	28	0	0.7
6		50	45	66	16	0.95	0.55	1.3	0.43	0	23	0	0.5
29	CL	100	90	125	170	0.37	1.07	32.5	0.09	11	12	0	1.0
25		95	85	119	120	0.45	1.00	21.1	0.13	9	15	4	0.8
7		90	80	112	75	0.54	0.94	10.2	0.17	7	17	7	0.6
26		85	75	106	50	0.60	0.90	5.2	0.21	6	18	8	0.5
8		80	70	100	35	0.66	0.87	3.5	0.25	5	19	8.5	0.4
9		50	40	56	16	0.95	0.75	0.7	0.55	0	23	11	0.3

Table E.2 Soil Properties for Pre-Existing (In-Situ) Soil and Special Materials [39]

Soil No.	Soil Type	State		γ_m pcf	K	n	R_f	B_i/P_a	e_u	C psi	ϕ deg	$\Delta\phi$ deg	K_o
10	1	A	Dense	145	680	0	0	22E3	0	100	50	0	1.0
11		B	Medium	130	408	0	0	45.3	0	100	50	0	1.0
12		C	Loose	115	136	0	0	76	0	100	50	0	1.0
13	2	D	Very Stiff	125	408	0	0	340	0	100	50	0	1.0
14		E	Firm to Stiff	117	238	0	0	393	0	100	50	0	1.0
15		F	Soft	110	68	0	0	2200	0	100	50	0	1.0
16		Concrete		150	21E4	0	0	11E4	0	300	0	0	1.0
17		Asphalt-warm		140	2E4	0	0	34E3	0	50	0	0	1.0
18		Asphalt-cold		140	10E4	0	0	17E4	0	150	0	0	1.0
19		Rock-weak		145	68E2	0	0	38E3	0	100	50	0	1.0
20		Rock-competent		160	34E4	0	0	28.3E4	0	1000	0	0	1.0

Table E.3 Properties of Soft Materials [49]

Material	Density (pcf)	Modulus of Elasticity, E (psi)
EPS, Geofoam	0.9	34
Polystyrene Beads	0.6	36
Straw bales	9.3	35
Wood waste	66.9	161
Sawdust	15.6	155
Woodchips	9.1	270
Tire chips	39.4	263
Hay	2.1	400

Note: A low-density expanded polystyrene (EPS) material is generally used as the compressible material.

APPENDIX F

AASHTO and ASCE Standard Specifications

F.1 Material

F.1.1 Concrete Roadway Pipe

F.1.1.1 General

Concrete pipe shall comply with the requirements of the following specifications for the classes and sizes.

Circular Pipe	AASHTO M 170 or AASHTO M 242
	(ASTM C76 or C 655)
Arch Pipe	AASHTO M 206 (ASTM C 506)
Elliptical Pipe	AASHTO M 207 (ASTM C 507)

For definitions of terms relating to concrete pipe, see AASHTO M262.

F.1.1.2 Classification

Pipe manufactured in accordance with this specification shall be of five classed identified as Class I, Class II, Class III, Class IV, and Class V. The corresponding strength requirements are prescribed in Table 1 to 5 of AASHTO M 170.

F.1.1.3 Materials

1) Reinforced Concrete

The reinforced concrete shall consist of cementitious materials, mineral aggregates, and water in which steel has been embedded in such a manner that the steel and concrete act together. Aggregate, cement, steel reinforcement, and water shall meet the requirements of AASHTO M 170.

2) Aggregate (ALDOT Section 801, 802)

Aggregate shall be sized, graded, pro-portioned, and mixed with such pro-portions of Portland cement, blended hydraulic cement, or Portland cement and supplementary cementing materials, or admixtures, or a combination thereof, and water to produce a homogeneous concrete mixture of such quality that the pipe will conform to the test and design requirements of the specification.

Aggregates shall conform to AASHTO M 6 and M 80.

3) Cement (ALDOT Section 815)

Cement shall conform to the requirements for Portland cement of M 85, or shall be Portland blast-furnace slag cement or Portland-pozzolan cement conforming to the requirements of M 240, except that the pozzolan constituent in the Type IP Portland-pozzolan cement shall be fly ash.

4) Steel Reinforcement (ALDOT Section 835)

Reinforcement shall conform to AASHTO M 170. Reinforcement shall consist of wire conforming to M 32M/M 32 or M 225M/M 225, or of wire fabric conforming to M 55M/M55 or M 221 M/M 221, or of bars of minimum Grade 40 steel conforming to M 31M/M 31.

F.1.1.4 Acceptance

Acceptance on the Basis of Material Tests and Inspection of Manufactured Pipe for Defects and Imperfections – Acceptability of the pipe in all diameters and classed produced in accordance with requirements specified in AASHTO M 170 shall be determined by the results of such material tests as are required in Section F.1.1.3; by crushing tests on concrete cores or cured concrete cylinders; by absorption tests on selected samples from the wall of the pipe; and by inspection of the finished pipe including amount and placement of reinforcement to determine its conformance with the accepted design and its freedom from defects.

F.1.1.5 Repairs

Pipe may be repaired, if necessary, because of imperfections in manufacturing or damage during handling and will be acceptable if, in the opinion of the owner, the repaired pipe conforms to the requirements of this specification.

F.1.1.6 Inspection

The quality of materials, the process of manufacture, and the finished pipe shall be subject to inspection and approval by the owner.

F.1.1.7 Rejection

Pipe shall be subject to rejection on account of failure to conform to any of the specification requirements. Individual sections of pipe may be rejected because of any of the followings:

1) Fractures of cracks passing through the wall, except for a single end crack that does not exceed the depth of the joint.
2) Defects that indicate proportioning, mixing, and molding not in compliance with Section 10 of AASHTO M 170 or surface defects indicating honey-combed or open texture that would adversely affect the function of the pipe.
3) The ends of the pipe are not normal to the walls and centerline of the pipe.
4) Any continuous crack having a surface width of 0.01 in. or more and extending for a length of 12 in. or more, regardless of position in the wall of the pipe.

F.1.1.8 Handling and Storage

Pipe shall be handled, transported, delivered, and stored in a manner that will not injure or damage the pipe. Pipe shall not be shipped before it has been inspected and approved. Pipe that is damaged during shipment or handling will be rejected even though satisfactory before shipment. Pipe dropped from platforms or vehicles will be rejected.

F.1.2 Box Culverts

F.1.2.1 General

Box culverts shall comply with the requirements of AASHTO M 259. For definitions of terms relating to concrete pipe, see AASHTO M 262.

F.1.2.2 Types

Precast reinforced concrete box sections manufactured in accordance with this specification shall be of three types identified in Tables 1, 2, and 3 of AASHTO M 259, and shall be designated by type, span, and design earth cover.

F.1.2.3 Acceptance

Acceptability of the box sections produced shall be determined by the results of the concrete compressive strength tests described in AASHTO T 280, by the material requirements described in Section F.1.2.4, and by inspection of the finished box sections.

F.1.2.4 Materials

Aggregate, cement, steel reinforcement, and water shall meet the requirements of AASHTO M 259.

1) Aggregate (ALDOT Section 801, 802)

Aggregates shall conform to AASHTO M 6 and M 80, except that the requirements for gradation shall not apply.

2) Cement (ALDOT Section 815)

Cement shall conform to the requirements for Portland cement of M 85, or shall be Portland blast-furnace slag cement or Portland-pozzolan cement conforming to the requirements of M 240, except that the pozzolan constituent in the Type IP Portland-pozzolan cement shall be fly ash and shall not exceed 25 percent by weight.

3) Steel Reinforcement (ALDOT Section 835)

Reinforcement shall consist of welded wire fabric conforming to M 55M/M55 or M 221 M/M 221.

F.1.2.5 Manufacture

Manufacture shall conform to the requirements in Section 9 of AASHTO M 259.

F.1.2.6 Repairs

Box sections may be repaired, if necessary, because of imperfections in manufacture of handling damage and will be acceptable if, in the opinion of the owner, the repaired box section conforms to the requirements of this specification.

F.1.2.7 Inspection

The quality of materials, the process of manufacture, and the finished box sections shall be subject to inspection and approval by the owner.

F.1.2.8 Rejection

Box sections shall be subject to rejection on account of failure to conform to any of the specification requirements. Individual box sections may be rejected because of any of the followings:

1) Fractures of cracks passing through the wall, except for a single end crack that does not exceed the depth of the joint.

2) Defects that indicate proportioning, mixing, and molding not in compliance with Section F.1.2.5 or surface defects indicating honey-combed or open texture that would adversely affect the function of the box sections.

3) The ends of the box sections are not normal to the walls and centerline of the box section, except where beveled ends are specified.

4) Damaged ends, where such damage would prevent making a satisfactory joint.

F.1.2.9 Handling and Storage

Box Culverts shall be handled, trans-ported, delivered, and stored in a manner that will not injure or damage the Box Culverts. Box Culverts shall not be shipped before it has been inspected and approved. Box Culverts that are damaged during shipment or handling will be rejected even though satisfactory before shipment. Box Culverts dropped from platforms or vehicles will be rejected.

F.2 Design

F.2.1 Concrete Roadway Pipe

F.2.1.1 General

This Specification covers the design for precast reinforced concrete circular pipe, elliptical Pipe, and arch pipe. Standard dimensions are shown in Table 1 to 5 of AASHTO Material Specifications M 170, M206, M207, and M242. Design wall thicknesses other than the standard wall dimensions may be used, provided the design complies with all applicable requirements of this Section. Design shall conform to applicable sections of these specifications except as provided otherwise in this Section.

F.2.1.2 Design Tables

The diameter, wall thickness, compressive strength of the concrete, and the area of the circumferential reinforcement shall be as prescribed for Class I to V in Tables 1 to 5 of AASHTO M 175, except as provided in Section 2.1.5.2 of AASHTO M 175.

F.2.1.3 Modified and Special Design

When so permitted by the plans or in the proposal, pipe of designs than those shown in the standard plans may be permitted. Such pipe must meet all of the test and performance requirements specified by the owner.

F.2.1.4 AASHTO Standard Installations

AASHTO Standard Embankment Installations are presented in Figure F.1 and AASHTO Standard Trench Installations are presented in Figure F.2; these figures define soil areas and critical dimensions. Generic soil types, minimum compaction requirements, and minimum bedding thicknesses are listed in Table F.1 for four AASHTO Standard Embankment Installation Types and in Table F.2 for four AASHTO Standard Trench Installation Types. Embankment beddings of miscellaneous shapes are presented in Figure F.3 and trench beddings of miscellaneous shapes are presented in Figure F.4.

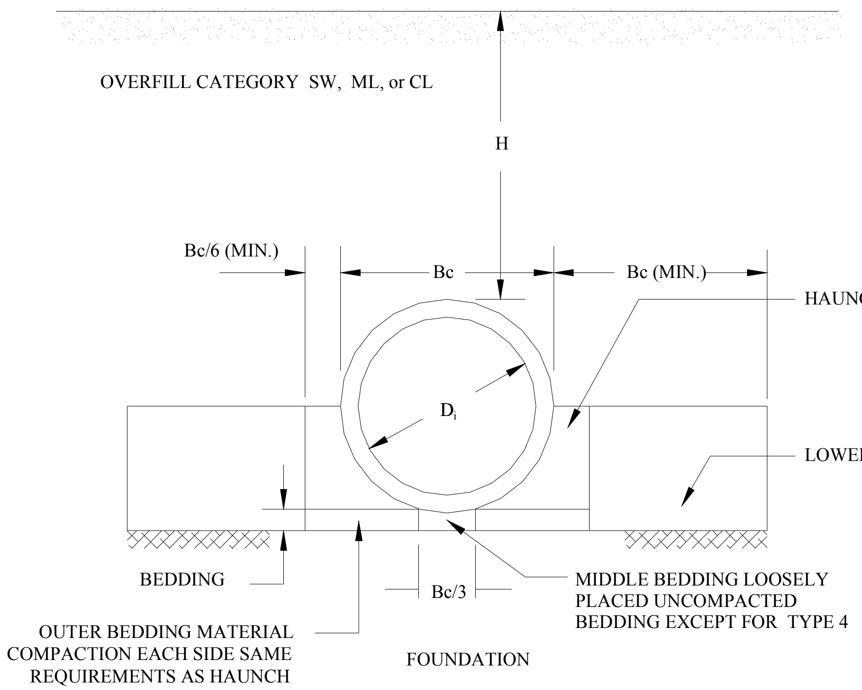


Figure F.1 AASHTO Standard Embankment Installations

Table F.1 AASHTO Standard Embankment Installation Soils and Minimum Compaction Requirements

Installation Type	Bedding Thickness	Haunch and Outer Bedding	Lower Side
Type 1	B_c /24" minimum, not less than 3". If rock foundation, use B_c /12" minimum, not less than 6".	95% SW	90% SW, 95% ML, 0r 100% CL
Type 2 (See Note 3.)	B_c /24" minimum, not less than 3". If rock foundation, use B_c /12" minimum, not less than 6".	90% SW or 95% ML	85% SW, 90% ML, 0r 95% CL
Type 3 (See Note 3.)	B_c /24" minimum, not less than 3". If rock foundation, use B_c /12" minimum, not less than 6".	85% SW, 90% ML, or 95% CL	85% SW, 90% ML, 0r 95% CL
Type 4	No bedding required, except if rock Foundation, use B_c /12" minimum, not less than 6".	No compaction required Except if CL, use 85% CL	No compaction required Except if CL, use 85% CL

NOTES:

1. Compaction and soil symbols –i.e. “95% SW” refer to SW soil material with a minimum standard proctor compaction of 95%. See Table F.3 for equivalent modified proctor values.

2. Soil in the outer bedding, haunch, and lower side zones, except within B_c /3 from the pipe springline, shall be compacted to at least the same compaction as the majority of soil in the overfill zone.

3. Only Type 2 and 3 installations are available for horizontal elliptical, vertical elliptical and arch pipe..

4. Subtrenches

4.1 A subtrench is defined as a trench with its top below finished grade by more than 0.1H or, for roadways, its top is at elevation lower than 1’ (0.3 m) below the bottom of the pavement base material.

4.2 The minimum width of a subtrench shall be 1.33 B_c, or wider if required for adequate space to attain the specified compaction in the haunch and bedding zones.

4.3 For subtrenches with walls of natural soil, any portion of the lower side zone in the subtrench wall shall be at least as firm as an equivalent soil placed to the compaction requirements specified for the lower side zone and as firm as the majority of soil in the overfill zone, or shall be removed and replaced with soil compacted to the specified level.

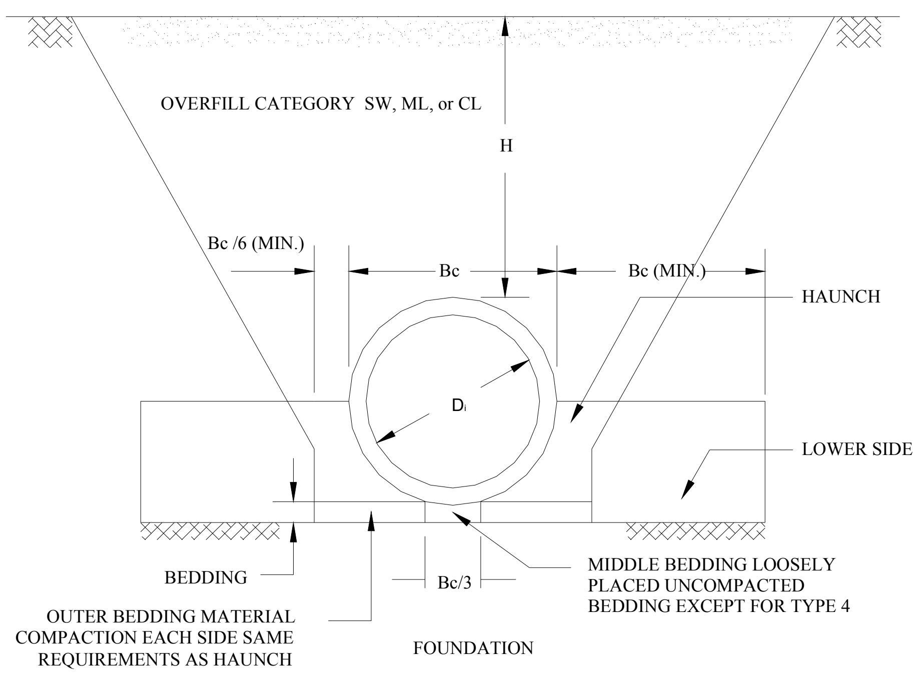


Figure F.2 AASHTO Standard Trench Installations

Table F.2 AASHTO Standard Trench Installation Soils and Minimum Compaction Requirements

Installation Type	Bedding Thickness	Haunch and Outer Bedding	Lower Side
Type 1	B_c /24" minimum, not less than 3". If rock foundation, use B_c /12" minimum, not less than 6".	95% SW	90% SW, 95% ML, 100% CL, or natural soils of equal firmness
Type 2 (See Note 3.)	B_c /24" minimum, not less than 3". If rock foundation, use B_c /12" minimum, not less than 6".	90% SW or 95% ML	85% SW, 90% ML, 95% CL, or natural soils of equal firmness
Type 3 (See Note 3.)	B_c /24" minimum, not less than 3". If rock foundation, use B_c /12" minimum, not less than 6".	85% SW, 90% ML, or 95% CL	85% SW, 90% ML, 95% CL, or natural soils of equal firmness
Type 4	No bedding required, except if rock Foundation, use B_c /12" minimum, not less than 6".	No compaction required Except if CL, use 85% CL	85% SW, 90% ML, 95% CL, or natural soils of equal firmness

NOTES:

1. Compaction and soil symbols –i.e. "95% SW" refer to SW soil material with a minimum standard proctor compaction of 95%. See Table F.3 for equivalent modified proctor values.

2. The trench top elevation shall be no lower than 0.1H below finished grade or, for roadways, its top shall be no lower than an elevation of 1' below the bottom of the pavement base material.

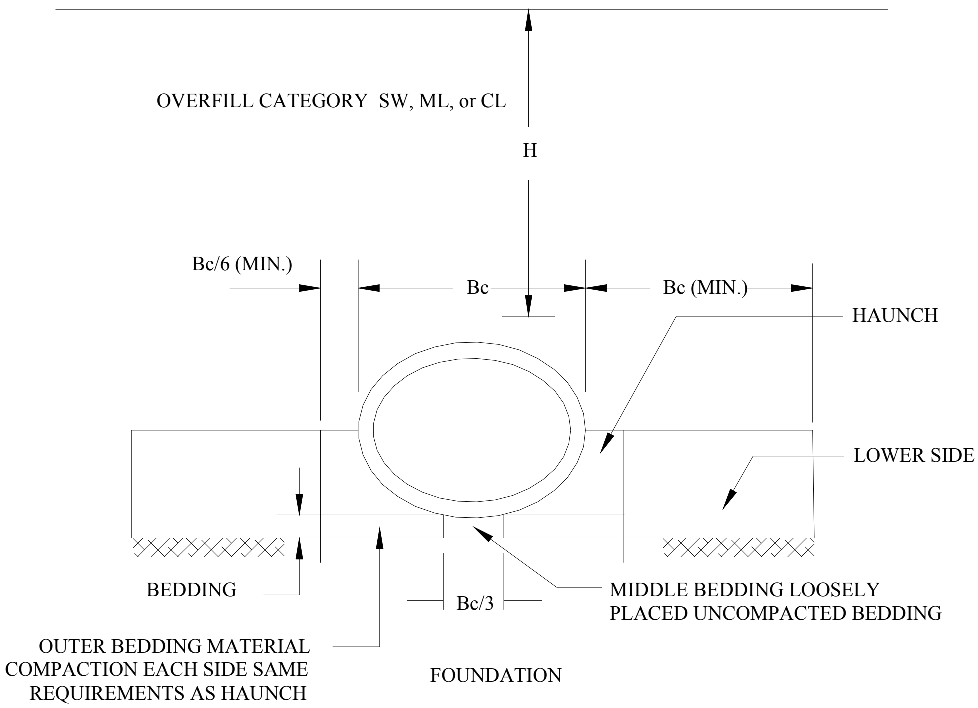
3. Only Type 2 and 3 installations are available for horizontal elliptical, vertical elliptical and arch pipe..

4. Soil in bedding and haunch zones shall be compacted to at least the same compaction as specified for the majority of soil in the backfill zone.

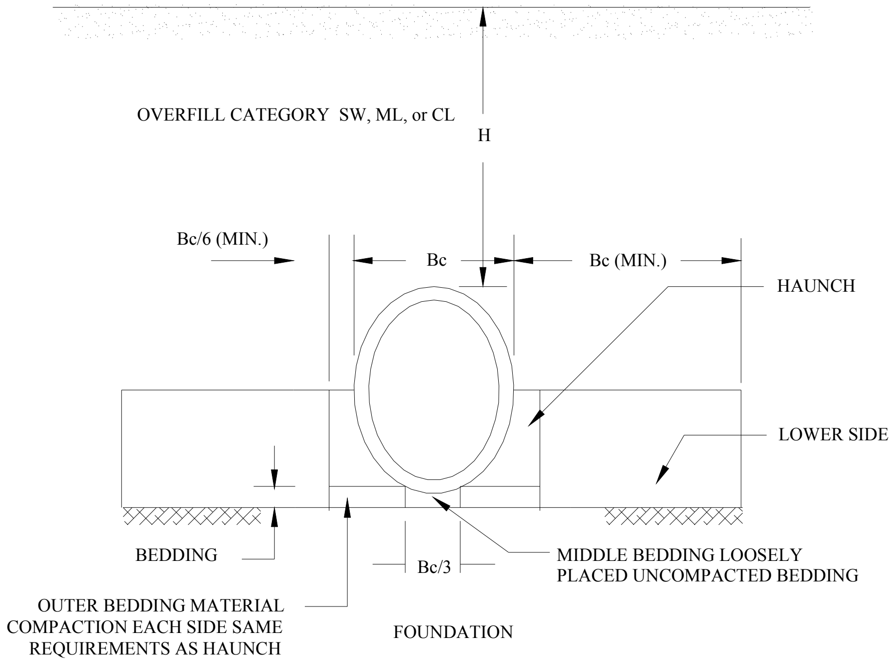
5. The trench width shall be wider shown if required for adequate space to attain the specified compaction in the haunch and bedding zones.

6. For trench walls that are within 10 degrees of vertical, the compaction or firmness of the soil in the trench walls and lower side zone need not be considered.

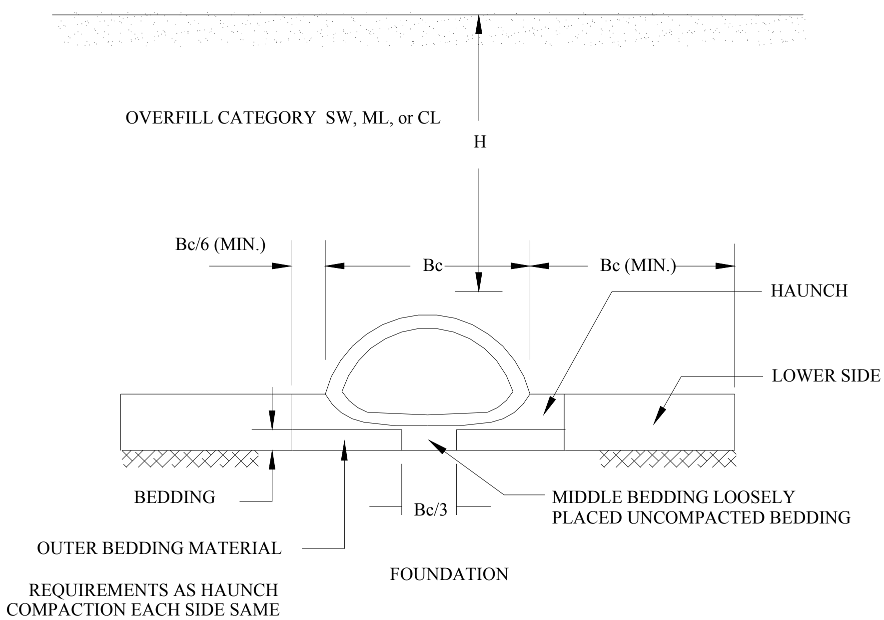
7. For trench walls with greater than 10-degree slopes that consist of embankment, the lower side shall be compacted to al least the same compaction as specified for the soil in the backfill zone.



Horizontal Elliptical Pipe

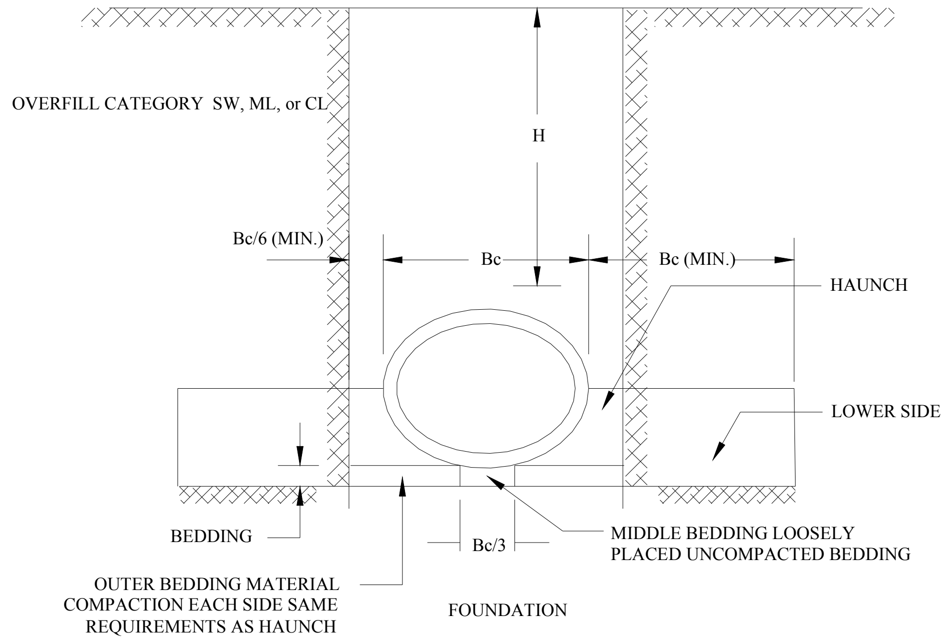


Vertical Elliptical Pipe

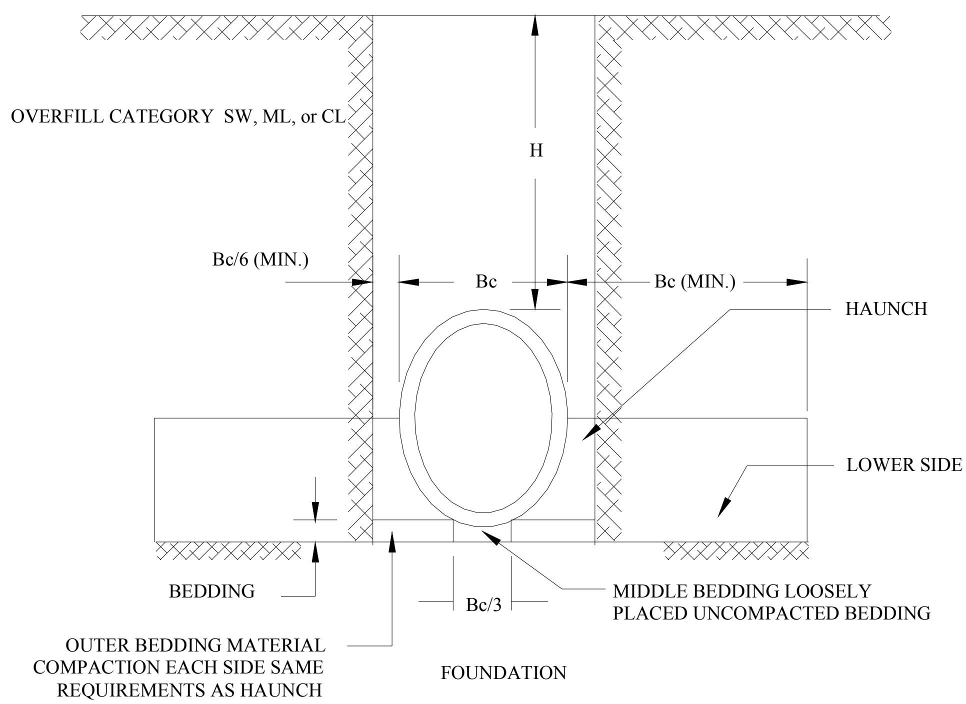


Arch Pipe

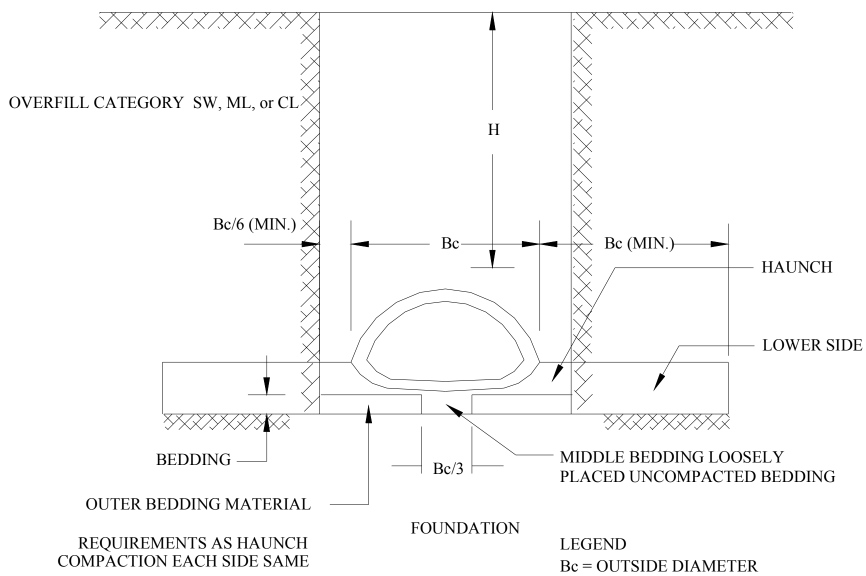
Figure F.3 Embankment Beddings, Miscellaneous Shapes



Horizontal Elliptical Pipe



Vertical Elliptical Pipe



Arch Pipe

Figure F.4 Trench Beddings, Miscellaneous Shapes

F.2.1.6 Soils

The AASHTO Soil Classifications and the USCS Soil Classifications equivalent to the generic soil types in the AASHTO Standard Installations are presented in Table F.3.

F.2.1.7 Dead Loads

1) Earth Loads and Pressure Distribution; The effects of soil-structure interaction shall be taken into account and shall be based on the design earth cover, sidefill compaction, and bedding characteristics of the pipe-soil installations.

2) The dead load of the pipe weight shall be considered in the design and based on a reinforced concrete density of 150 lbs/cu ft (24 kN/cu m), unless otherwise specified.

3) The earth load from the fill over the pipe shall be based on the design soil unit weight (mass) specified by the owner , but not less than 110lbs/cu ft (17.6 kN/cu m), unless otherwise specified.

Table F.3 Equivalent USCS and AASHTO Soil Classifications for SIDD Soil Designations

SIDD Soil	Representative Soil Types		Percent Compaction	
	USCS	AASHTO	Standard Proctor	Modified Proctor
Gravelly Sand	SW, SP	A1, A3	100	95
(SW)			95	90
			90	85
			85	80
			80	75
			61	59
Sandy Silt	GM, SM, ML	A2, A4	100	95
(ML)	Also GC,SC		95	90
	With less than		90	85
	20% passing		85	80
	No.200 sieve		80	75
			49	46
Silty Clay	GL, MH, GC,	A5, A6	100	90
(CL)	SC		95	85
			90	80
			85	75
			80	70
			45	40
	CH	A7	100	90
			95	85
			90	80
			45	40

4) For unpaved and flexible pavement areas, the minimum fill, including flexible pavement thickness, over the top outside of the pipe shall be 1 ft. (300mm), or 1/8 of the inside diameter, whichever is greater.

Under rigid pavements, the distance between the top of the pipe and the bottom of the pavement slab shall be a minimum of 6 in. (150 mm) of compacted granular fill.

5) The dead load of fluid in the pipe shall be based on a unit weight of 62.4 lbs/cu ft (10 kN/cu m), unless otherwise specified.

6) AASHTO Standard Installations

For the AASHTO Standard Installations given in Section F.2.1.2, the earth load, W_E may be determined by multiplying the prism load (weight of the column of earth) over the pipes outside diameter by the soil-structure interaction factor, F_e, for the specified installation type.

$$W_E = F_e w B_c H \tag{F-1}$$

AASHTO Standard Installations for both embankments and trenches shall be designed for positive projection, embankment loading conditions where F_e=VAF given, in Figure F.5 for each type of AASHTO Standard Installation.

For AASHTO Standard Installations, the earth pressure distribution shall be the Heger pressure distribution shown in Figure F.5 for each type of AASHTO Standard Installations. The unit weight of soil used to calculate earth load shall be the estimated unit weight for the soils specified for the pipe-soil installation and shall not be less than 110lbs/cu ft.

7) Nonstandard Installations

When nonstandard installations are used, the earth load and pressure distribution shall be determined by an appropriate soil-structure interaction analysis.

F.2.1.8 Live Loads

1) Truck loads shall be either the AASHTO HS-series or the AASHTO Interstate Design load. An impact factor need not be added to AASHTO live loads on pipe installed in accordance with F.2.1.7.

2) Railroad loads shall be the area designated Cooper E-series.

3) Aircraft or other live loads shall be is specified by the owner.

F.2.1.9 Minimum Fill

For unpaved areas and under flexible pavements, the minimum fill over precast reinforced concrete pipe shall be 1 foot or 1/8 of the diameter or rise, whichever is greater. Under rigid pavements, the distance between the top of the pipe and the bottom of the pavement slab shall be a minimum of 9 inches of compacted granular fill

F.2.1.10 Indirect Design Method

1) Loads

The design load-carrying capacity of a reinforced concrete pipe must equal the design load determined for the pipe as installed, or

$$D = \left[\frac{12}{S_1}\right]\left[\frac{W_E + W_F}{B_{fe}} + \frac{W_L}{B_{fLL}}\right] \tag{F-2}$$

Where,

D = D-load of the pipe (three edge-bearing test load expressed in pounds per linear foot per foot of diameter) to produce a 0.01-inch crack. For Type 1 installations, D-load as calculated above shall be modified by multiplying by an installation factor of 1.10;

S_i = internal diameter or horizontal span of the pipe in inches;

B_f = bedding factor;

B_{Fc} = internal diameter or horizontal span of the pipe in inches;

B_{FLL} = live load bedding factor;

W_T = $W_E + W_L$;

W_T = total load on the pipe ;

W_E = earth load on the pipe;

W_P = fluid load in the pipe;

W_L = live load on the pipe.

2) Ultimate D-load

The required D-load at which the pipe develops its ultimate strength in a three-edge-bearing test is the design D-load (at 0.01 – inch crack) multiplied by a strength factor that is specified in AASHTO materials specifications M 170 or M 242 (ASTM C 76 or C655) for circular pipe, M206 (AST C506) for arch pipe and M207 (AST C 507) for elliptical pipe.

3) Bedding Factor

The bedding Factor, B_f, is the ratio of the supporting strength of buried pipe to the strength of the pipe determined in the three-edge-bearing test. The supporting strength of buried pipe depends on the type of AASHTO Standard Installation. See Figures F.1 and F.2 for circular pipe and Figures F.3 and F.4 for other arch and elliptical shapes. The Tables F.1 and F.2 apply to circular, arch and elliptical shapes.

4) Earth Load Bedding Factor for Circular Pipe. Earth load bedding factors, B_{fe}, for circular pipe are presented in Table F.4.

5) Earth Load Bedding Factor for Arch Pipe and Elliptical Pipe. The bedding factor for installations of arch and elliptical pipe Figures F.3 and F.4, is

$$B_{fe} = \frac{C_A}{C_N - xq} \qquad \text{(F-3)}$$

Values for C_A and C_N are listed in Table 16.4D of AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002).

C_A = a constant corresponding to the shape of the pipe;

C_N = a parameter which is a function of the distribution of the vertical load and vertical reaction;

x = a parameter which is a function of the area of the vertical projection of the pipe over which lateral pressure is effective;

q = the ratio of the total lateral pressure to the total vertical fill load;

Table F.4 Bedding Factors for Circular Pipe

Pipe Inside Diameter (in.)	Type 1	Type 2	Type 3	Type 4
12	4.4	3.2	2.5	1.7
24	4.2	3.0	2.4	1.7
36	4.0	2.9	2.3	1.7
72	3.8	2.8	2.2	1.7
144	3.6	2.8	2.2	1.7

NOTE:

For pipe diameters other than listed, embankment condition bedding factors, B_{fc} can be obtained by interpolation.

Bedding factors are based on soils being placed with the minimum compaction specified in Tables F.1 and F.2 for each AASHTO Standard Installation.

6) Live Load Bedding Factor

The bedding factors for live load, W_L, for both circular pipe and arch and elliptical pipe are given in Table 16.5F of Standard Specifications for Highway Bridges (AASHTO, 2002). If B_{fe} is less than B_{FLL}, use B_{fe} instead of B_{FLL} for the live load bedding factor.

Design values for C_A, C_N, and x are found in Table 16.4D of Standard Specifications for Highway Bridges (AASHTO, 2002). The value of q is determined by the following equations:

Arch and Horizontal Elliptical Pipe

$$q = .23\frac{p}{F_e}\left(1 + .35p\frac{B_e}{H}\right) \tag{F-4}$$

Vertical Elliptical Pipe

$$q = .48\frac{p}{F_e}\left(1 + .73p\frac{B_e}{H}\right) \tag{F-5}$$

Where

p = projection ratio, ratio of the vertical distance between the outside top of the pipe and the ground or bedding surface to the outside vertical height of the pipe.

7) Intermediate Trench Widths

For intermediate trench widths, the bedding factor may be estimated by interpolation between the narrow trench and transition width bedding factors.

F.2.1.11 Direct Design Method

1) Application

This Specification is intended for use in direct design of precast reinforced concrete circular pipe, and is based on design of pipe wall for effects of loads and pressure distribution for installed conditions. Standard dimensions are shown in AASHTO M 170. Design wall thicknesses other than the standard wall dimension may be used provided the design complies with all applicable requirements of AASHTO M 170.

2) General

Design shall conform to applicable sections of these specifications, except as provided otherwise in this article. The total load on the pipe shall be determined according to Sections F.2.1.9 and F.2.1.10.

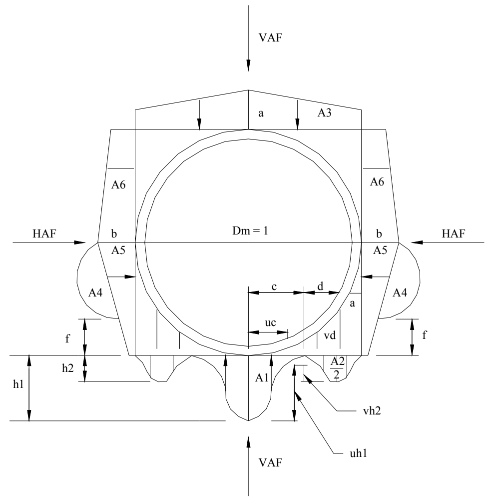
The pressure distribution on the pipe from applied loads and bedding reaction shall be determined from a soil-structure analysis or shall be a rational approximation. Acceptable pressure distribution diagrams are the Heger Pressure Distribution (see Figure F.5) for use with the AASHTO Standard InstallationS: the Olander/Modified Olander Radial Pressure Distribution (see Figure F.6); or the Paris/Manual Uniform Pressure Distribution (see Figure F.6). For use with the Heger Pressure Distribution, two Types of AASHTO Standard Embankment Installations, soil types, and compaction requirements are depicted in Figures F.1 and F.2 and Tables F.1 and F.2. Table F.3 relates the AASHTO Standard Installation designated soils to the AASHTO and Unified Soil Classification System categories. Other methods for determining total load and pressure distribution may be used, if based on successful design practice or tests that reflect the appropriate design condition.

3) Strength-Reduction Factors

Strength-reduction factors for load factor design of plant made reinforced concrete pipe may be taken as 1.0 for flexible and 0.9 for shear and radial tension.

4) Process and Material Factors

Process and material factors, Frp for radial tension and Fvp for shear strength for load factor design of plant made reinforced concrete pipe are conservatively taken as 1.0. Higher values may be used if substantiated by appropriate test data approved by the Engineer.



Type	VAF	HAF	A1	A2	A3	A4	A5	A6
1	1.35	0.45	0.62	0.73	1.35	0.19	0.08	0.18
2	1.40	0.40	0.85	0.55	1.40	0.15	0.08	0.17
3	1.40	0.37	1.05	0.35	1.40	0.10	0.10	0.17
4	1.45	0.30	1.45	0.00	1.45	0.00	0.11	0.19

Type	A	b	c	e	f	u	V
1	1.40	0.40	0.18	0.08	0.05	0.80	0.80
2	1.45	0.40	0.19	0.10	0.05	0.82	0.70
3	1.45	0.36	0.20	0.12	0.05	0.85	0.60
4	1.45	0.30	0.25	0.00	-	0.90	-

Notes:

VAF and HAF are vertical and horizontal arching factors. These coefficients represent nondimensional total vertical and horizontal loads on the pipe, respectively. The actual total vertical and horizontal loads are (VAF)×(PL) and (HAF) ×(PL), respectively, where PL is the prism load.

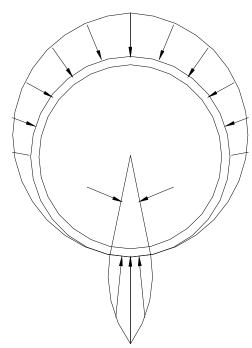
Coefficients A1 through A6 represent the integration of nondimensional vertical and horizontal components of soil pressure under the indicated portions of the component pressure diagram (i.e., the area under the component pressure diagram). The pressures are assumed to vary either parabolically or linearly, as shown, with the nondimensional magnitudes at governing points represented by h_1, uh_1,vh_1, a and b. Nondimensional horizontal and vertical dimensions of component pressure regions are defined by c, d, e, uc, vd, and f coefficients.

d is calculated as (0.5 c-e)

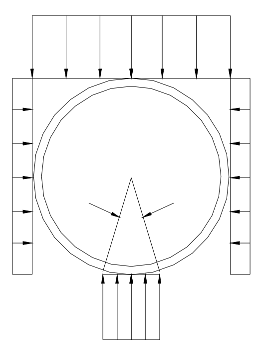
h_1 is calculated as (1.5A1)/(c) (l+u)

h_2 is calculated as (1.5 A2) / [(d)(1+v)+(2e)]

Figure F.5 Heger Pressure Distribution and Arching Factors



Olander/Modified Olander Radial Pressure Distribution



Paris/Manual Uniform Pressure Distribution

Figure F.6 Suggested Design Pressure Distribution

5) Orientation Angle

When quadrant mats, stirrups and/or elliptical cages are used, the pipe installation requires a specific orientation. Design shall be based on the possibility of a rotation misorientation during installation by an Orientation angle of 10° in either direction.

6) Reinforcement for Flexural Strength

$$A_s = \left(g\phi_f d - N_u - \sqrt{g[g(\phi_f d)^2 - N_u(2\phi_f d - h) - 2M_u]}\right)/(f_y) \quad \text{(F-6)}$$

Where g=0.85 bf_c'

b = 12in

7) Minimum Reinforcement

For inside face of pipe

$$A_{si} = \frac{b}{12}(S_i \div h)^2/(f_y) \quad \text{(F-7)}$$

Where b=12in

For outside face of pipe

$$A_{so} = 0.60\left(\frac{b}{12}\right)(S_f \div h)^2/(f_y) \quad \text{(F-8)}$$

Where b=12in

For elliptical reinforcement in circular pipe and for pipe 33-inch diameter and smaller with a single cage of reinforcement in the middle third of the pipe wall, reinforcement shall not be less than A, where:

$$A_s = 2(b/12)(S_i \div h)^2/(f_y) \quad \text{(F-9)}$$

Where b=12in

h=wall thickness in inches;

S_i = internal diameter or horizontal span of pipe in inches.

In no case shall the minimum reinforcement be less than 0.07 square inches per linear foot.

8) Maximum Flexural Reinforcement Without Stirrups

Limited by Radial Tension

$$A_{s\ max} = \left(\frac{b}{12}\right)\left(16 r_s F_{rp} \sqrt{f'_c}\left(\frac{\phi_r}{\phi_f}\right) F_{rt}\right) / (f_y) \tag{F-10}$$

Where

$A_{s\ max}$ = maximum flexural reinforcement area without stirrups in in.^{2}/ft

b =12in

F_{rt} = 1+0.00833 (72 – S_i)

For 12in.≤ S_i ≤ 72in

F_{rp} = 1.0 unless a higher value substantiated by test data is approved by the Engineer;

For 72in.< S_i ≤144in.

F_{rt} = 0.8 for S_i > 144in.

r_s = radius of the inside reinforcement in inches.

9) Limited by Concrete Compression

$$A_{s\ max} = \left(\left[\frac{5.5 \times 10^4 g' \phi_f d}{(87,000 + f_y)}\right] - 0.75 N_u\right) / (f_y) \tag{F-11}$$

Where

$$g' = bf'_c \left[0.85 - 0.05 \frac{(f'_c - 400)}{1,000}\right]$$

$g'_{max} = 0.85bf'_c$ and $g'_{min} = 0.65bf'_c$

10) Crack Width Control (Service Load Design)

$$F_{cr} = \frac{B_1}{30,000 \phi_f d A_s}\left[\frac{M_s + N_s\left(d - \frac{h}{2}\right)}{ij} - C_1 b h^2 \sqrt{f'_c}\right] \tag{F-12}$$

F_{cr} = crack control factor, see Note c;

M_s = bending moment, service load;

N_s = thrust (positive when compressive), service load.

Crack control is assumed to be 1 inch from the closest tension reinforcement, even if the cover over the reinforcement is greater or less than 1 in. The crack control factor F_{cr} in Equation F-12 indicates the probability that a crack of a specified maximum width will occur.

When F_{cr} =1.0, the reinforcement area, A_s, will produce an average crack maximum width of 0.01 inch. For F_{cr} values less than 1.0, the probability of a 0.01 inch crack is reduced. For F_{cr} values greater than 1.0, the probability of a crack greater than 0.01 inch is increased.

If the service load thrust, N_s is tensile rather than compressive (this may occur inn pipes subject to intermittent hydrostatic pressure), use the quantity (1.1 M_s – 0.6 N_s d) (with tensile N_s taken negative) in place of the quantity ([M_s + N_s (d-h/2)]/ji) in Equation F-12.

$j \cong 0.74 + 0.1\ e/d$;

j_{max} =0.9;

$$i = \frac{1}{1 - \frac{jd}{e}}$$

$$e = \frac{M_s}{N_s} + d - \frac{h}{2}, \text{in}$$

if e/d<1.15 crack control will not govern

t_b = clear cover over reinforcement in inches

h = wall thickness of pipe in inches;

$$B_1 = \sqrt[3]{t_b s_1 / 2n}$$

where

s_l =spacing of circumferential reinforcement, in

n = 1, when tension reinforcement is a single layer.

n = 2, when tension reinforcement is made of multiple layers.

C_1 =Crack Control Coefficient

Notes: Higher values for C_1 may be used if substantiated by test data and approved by the Engineer

11) Shear Strength

The area of reinforcement, A_s, determined in Section F.2.1.11 must be checked for shear strength adequacy, so that the basic shear strength, V_b, is greater than the factored shear force, V_{uc}, at the critical section located where $M_{nu}/V_u d$ = 3.0.

Type of Reinforcement	C_1
1. Smooth wire or plain bars	1.0
2. Welded smooth wire fabric 8in.(200mm) maximum spacing of longitudinal	1.5
3. Welded deformed wire fabric, deformed wire, deformed bars, or any reinforcement with stirrups anchored thereto	1.9

$$V_b = b\phi_v d F_{vp}\sqrt{f_c'}(1.1+63\rho)\left[\frac{F_d F_N}{F_C}\right] \qquad \text{(F-13)}$$

Where

V_b = shear strength of section where $M_{nu}/V_u d$ =3.0

F_{vp} = 1.0 unless a higher value substantiated by test data is approved by the Engineer;

$$\rho = \frac{A_s}{bd}$$

ρ_{max} = 0.02;

$f_c'{}_{max}$ = 7,000psi;

$$F_d = 0.8 - \frac{1.6}{d}$$

Max. F_d = 1.3 for pipe with two cages, or a single elliptical cage

Max. F_d = 1.4 for pipe through 36-inch diameter with a single circular cage

$$F_C = 1 \pm \frac{d}{2r}$$

(+) tension on the inside of the pipe

(-) tension on the outside of the pipe;

For compressive thrust(+N_u)

$$F_N = 1 + \frac{N_u}{2,000bh}$$

Where b=12in

For tensile thrust (- N_u)

$$F_N = 1 + \frac{N_u}{500bh}$$

Where b=12in

$$M_{nu} = M_u - N_u\left[\frac{(4h-d)}{8}\right]$$

If V_b is less than V_{uc}, radial stirrups must be provided.

12) Radial Tension Stirrups

$$A_{vr} = \frac{1.1s_v(M_u - 0.45N_u\phi_r d)}{f_v r_s \phi_r d} \tag{F-14}$$

Where

A_{vr} = required area of stirrup reinforcement for radial tension;

s_v = circumferential spacing of stirrups ($s_{v\,max}$ =0.75ϕ_r d);

f_v = maximum allowable strength of stirrup material (f_{max} =f_y or anchorage strength, whichever is less).

13) Shear Stirrups

$$A_{vs} = \frac{1.1s_v}{f_{vs}\phi_v d}\left[V_u F_c - V_c\right] + A_{vr} \tag{F-15}$$

Where

A_{vs}= required area of stirrups for shear reinforcement;

V_u = factored shear force at section;

$$V_c = \frac{4V_b}{\frac{M_{nu}}{V_u d} + 1}$$

$$V_{c\,max} = 2\phi_v bd\sqrt{f_c^{'}}$$

$$S_{v\,max} = 0.75\phi_v d$$

$f_{v\,max}$ = f_y or anchorage strength, whichever is less

13) Radial Tension Stirrup Anchorage

When stirrups are used to resist radial tension, they shall be anchored around each circumferential of the inside cage to develop the design strength of the stirrup, and they shall also be anchored around the outside cage, or embedded sufficiently in the compression side to develop the design strength of the stirrup.

14) Shear Stirrup Anchorage

When stirrups are not required for radial tension but required for shear, their longitudinal spacing shall be such that they are anchored around each or every other tension circumferential. Such spacing shall not exceed 6 inches (150mm).

15) Stirrup Embedment

Stirrups intended to resist forces in the invert and crown regions shall be anchored sufficiently in the opposite side of the pipe wall to develop the design strength of the stirrup.

F.2.2 Box Culverts

F.2.2.1 General

This specification covers the design of cast-in-place and precast reinforced concrete box culverts. Standard dimensions are shown in AASHTO materials specifications M259 and M 273.

Design shall conform to these specifications except as provided otherwise in Section F.2.

F.2.2.2 Design Tables

The box section dimensions, compressive strength of the concrete, and reinforcement details shall be as prescribed in Tables 1 ,2, or 3 and Figures 1, 2, and 3, subject to the provisions of Section 11 of AASHTO M259. Table 1 sections are designed for combined earth dead load and AASHTO HS20 live load conditions. Table 2 sections are designed for combined earth dead load and Interstate live load conditions when the Interstate live loading exceeds the HS20 live loading. Table 3 sections are designed for earth dead load conditions only. For modifications to the designs shown in Tables 1, 2, and 3 due to anticipated earth and surcharge loads different from those used to develop the tables, see Section 6.3.2 in Chapter 6.

F.2.2.3 Modified and Special Design

The manufacturer may request approval by the owner of modified designs which differ from the designs in Section F.2.2; or special designs for sizes and loads other than those shown in Tables 1, 2, and 3 of AASHTO M259.

F.2.2.4 Reinforced Concrete Box Installations

Embankment Installations are presented in Figure F.7 and Trench Installations are presented in Figure F.8; these figures define soil areas and critical dimensions.

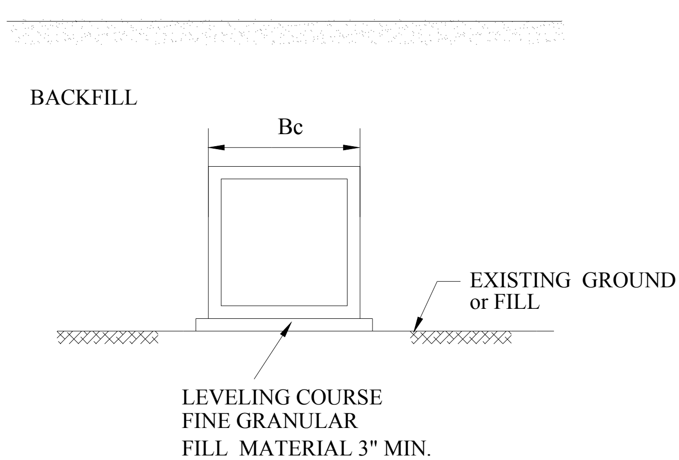


Figure F.7 Embankment Installations for Box Culverts

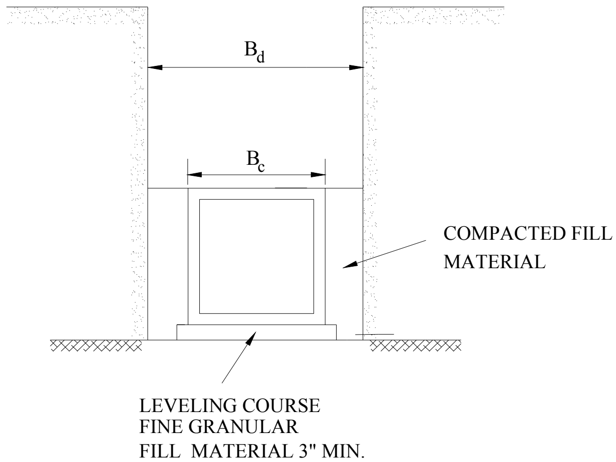


Figure F.8 Trench Installations for Box Culverts

F.2.2.6 Soils

The AASHTO Soil Classifications and the USCS Soil Classifications equivalent to the generic soil types in the AASHTO Standard Installations are presented in Table F.3.

F.2.2.7 Earth Loads

The effects of soil structure interaction shall be taken into account and shall be based on the design earth cover, sidefill compaction, and bedding characteristics. These parameters may be determined by a soil-structure interaction analysis of the system. The total earth load, W_E on the box section is

$$W_E = F_e w B_c H \tag{F-16}$$

F_e accounts for the type and conditions of installation as defined in Figures F.7 and F.8 and may be determined by the Marston-Spangler Theory of earth loads, as follows

Embankment Installations

$$F_{e1} = 1 + 0.20\frac{H}{B_c} \tag{F-17}$$

F_{e1} need not be greater than 1.15 for installations with compacted fill at the sides of the box section, and need not be greater than 1.4 for installations with uncompacted fill at the sides of the box section.

Trench Installations

$$F_{e2} = \frac{C_d B_d^2}{HB_c} \tag{F-18}$$

Values of C_d can be obtained from Figure 16.4B of Standard Specifications for Highway Bridges (AASHTO, 2002).for normally encountered soils. The maximum value of F_{e2} need not exceed F_{e1}.

The soil-structure interaction factor, F_e, is not applicable if the Service Load Design Method is used.

F.2.2.8 Distribution of Concentrated Load Effects to Bottom Slab

1) Reinforced concrete box, cast-in-place

The width of top slab strip used for distribution of concentrated wheel loads may be increased by twice the box height and use for the distribution of loads to the bottom slab.

2) Reinforced concrete box, precast

The width of the top slab strip used for distribution of concentrated wheel loads shall also be used for determination of bending moments, shears, and thrusts in the sides and bottoms.

F.2.2.9 Distribution of Concentrated Loads in Skewed Culverts

1) Reinforced concrete box, cast-in-place

Wheel loads on skewed culverts shall be distributed using the same provisions as given for culverts with main reinforcement parallel to traffic.

2) Reinforced concrete box, precast

Wheel loads on skewed culverts shall be distributed using the same provisions as given for culverts with main reinforcement parallel to traffic.

F.2.2.10 Span Length

When monolithic haunches included at 45° are considered in the design, negative moment reinforcement in walls and slabs may be proportioned based on the bending moment at the interaction of haunch and uniform depth member.

F.2.2.11 Strength-Reduction Factors

1) Reinforced concrete box, cast-in-place

Strength-reduction factors for load factor design may be taken at 0.9 for combined flexure and thrust and as 0.85 for shear.

2) Reinforced concrete box, precast

Strength-reduction factors for load factor design of machine-made boxes may be taken as 1.0 for moment and 0.9 for shear.

F.2.2.11 Crack Control

1) Reinforced concrete box, cast-in-place

The maximum service load stress in the reinforcing steel for crack control shall be

$$f_s = \frac{155}{\beta\sqrt[3]{d_c A}} \leq 0.6 fy \text{ ksi} \tag{F-19}$$

$$\beta = \left[1 + \frac{d_c}{0.7d}\right]$$

β = approximate ratio of distance from neutral axis to location of crack width at the concrete surface divided by distance from neutral axis to centroid of tensile reinforcing

d_c = distance measured from extreme tension fiber to center of the closet bar or wire in inches. For calculation purposes, the thickness of clear concrete cover used to compute d_c shall not be taken greater than 2 inches.

The service load stress should be computed considering the effects of both bending moment and thrust using:

$$f_s = \frac{M_s + N_s(d - h/2)}{(A_s jid)} \tag{F-20}$$

Where

f_s = stress in reinforcement under service load conditions, psi

$e = M_s/N_s + d - h/2$

e/d min =1.15

i=1/(1-(jd/e)

$j = 0.74 + 0.1(e/d) \leq 0.9$

2) Reinforced concrete box, precast

The maximum service load stress in the reinforcing steel for crack control shall be

$$f_s = \frac{98}{\sqrt[3]{d_c A}} \text{ksi} \tag{F-21}$$

The service load stress should be computed considering the effects of both bending moment and thrust using:

$$f_s = \frac{M_s + N_s(d - h/2)}{(A_s jid)} \tag{F-22}$$

Where

f_s = stress in reinforcement under service load conditions, psi

$e = M_s/N_s + d - h/2$

e/d min.=1.15

i=1/(1-(jd/e)

$j = 0.74 + 0.1(e/d) \leq 0.9$

F.2.2.12 Minimum Reinforcement

The primary flexural reinforcement in the direction of the span shall provide a ratio of reinforcement area to gross concrete area at least equal to 0.002. Such minimum reinforcement shall be provided at all cross sections subject to flexural tension, at the inside face of walls, and in each direction at the top of slabs of box sections with less than 2 feet of fill.

F.2.2.13 Concrete Cover for Reinforcement

The minimum concrete cover for reinforcement in boxes reinforced with wire fabric shall be three times the wire diameter but not less than 1 inch. For boxes covered by less than 2 feet of fill, the minimum cover for reinforcement in the top of the slab shall be 2 inches.

F.3 Construction

F.3.1 General

This specification covers construction practices of buried concrete culverts conforming to these specifications, the special provisions and the details shown on the plans. The detail construction procedures with regard to this specification are covered in Chapter 7. Precast reinforced concrete pipe shall be circular, arch or elliptical, as specified. Reinforced concrete box sections shall be of the dimensions specified or shown on the plans.

F.3.2 Materials

F.3.2.1 Reinforced Concrete Culverts

The materials for reinforced concrete culverts shall meet the requirements of the following specifications for the classes and sizes specified above.

Reinforced Concrete

Circular Pipe	AASHTO M170 or AASHTO M 242
	(ASTM C76 or C 655)
Arch Pipe	AASHTO M 206
	(ASTM C 506)
Elliptical Pipe	ASHTO M 207
	(ASTM C 506)
Box Sections	ASHTO M 259 and AASHTO M 273
	(ASTM C76 or C 655)

F.3.2.2 Joints and Sealants materials (ALDOT Section 846)

All joints are designed for ease of installation. Jointing procedures are provided in the Chapter 7.

1) Rubber gaskets

Rubber gaskets are either flat gaskets which may be cemented to the pipe tongue or spigot during manufacture, O-ring gaskets which are recessed in a groove on the pipe tongue or spigot and then confined by the bell or groove after the joint is completed, or roll-on gaskets which are placed around the tongue or spigot and then rolled into position as the tongue or spigot is inserted into the bell or groove.

2) Cement sealants

Cement sealants consist of Portland cement paste or mortar made with a mixture of Portland cement, sand and water. The joint surface is thoroughly cleaned and soaked with water immediately before the joint is made. A layer of paste or mortar is placed in the lower portion of the bell or groove end of the installed pipe and on the upper portion of the tongue or spigot end of the pipe section to be installed.

3) Mastic sealants

Mastic sealants consist of bitumen and inert mineral filler and are usually cold applied. The joint surfaces are thoroughly cleaned, dried and prepared in accordance with the manufacturer's recommendations.

4) Portland cement mortar bands

Portland cement mortar bands are specified around the exterior of the pipe joint. A slight depression is excavated in the bedding material to enable mortar to be placed underneath the pipe. The entire external joint surface is then cleaned and soaked with water.

5) Rubber-mastic bands

Rubber-mastic bands can be used around the exterior of the pipe joint. The bands are stretched tightly around the barrel of the pipe and held firmly in place by the weight of the backfill material.

6) Other Joint Sealant Materials

Other joint sealant materials shall be submitted for testing in advance of their use and shall not be used prior to receiving approval by the Engineer.

F.3.2.3 Bedding, Haunch, Lower Side and Backfill or Overfill Material

1) Reinforced Concrete Circular, Arch, and Elliptical Pipe

Bedding, haunch, lower side and overfill material shall conform to Figures F.1, F.2, F.3, and F.4, which define soil areas and critical dimensions, and Tables F.1 and F.2, which list generic soil types and minimum compaction requirements, and minimum bedding thicknesses for the two AASHTO Standard Installation Types. The AASHTO Soil Classifications and the USCS Soil Classifications equivalent to the generic soil types in the AASHTO Standard Installations are presented in Table F.3.

2) Reinforced Concrete Box Sections

For precast reinforced concrete box sections, bedding, sidefill and backfill material shall conform to Tables F.6 and F.7 with the following exceptions. Bedding material may be sand or select sandy soil all of which passes a U.S. Standard 3/8-inch sieve and not more than 10% of which passes a U.S. Standard No.200 sieve. Backfill may be select material and shall be free of organic material, stones larger than 3 inches in the greatest dimension frozen lumps, or moisture in excess of that permitting the specified compaction.

F.3.3 Assembly

F.3.3.1 General

Precast concrete units or elements shall be assembled in accordance with the manufacturer's instructions. All units or elements shall be handled with reasonable care and shall not be rolled or dragged over gravel or rock. Care shall be taken to prevent the units from striking rock or other hard objects during placement.

Cracks in an installed precast concrete culvert that exceed 0.01-inch width will be appraised by the Engineer considering the structural integrity, environmental conditions, and the design service life of the culvert. Generally in non-corrosive environments, cracks 0.10inch or less in width are considered acceptable; in corrosive environments, those cracks 0.01 inch or less in width are considered acceptable without repair. Cracks determined to be detrimental shall be sealed by a method approved by the Engineer.

F.3.3.2 Joints

Joints for reinforced concrete pipe and precast reinforced concrete box sections shall comply with the details shown on the plans, the approved working drawings, and the requirements of the special provisions. Each joint shall be sealed to prevent infiltration of soil fines or water as required by the contract documents. Joint sealant materials shall comply with the provisions of Section F.3.2.2.

F.3.4 Installation

F.3.4.1 General

Trenches shall be excavated to the dimensions and grade specified in the plans or ordered by the Engineer. The Contractor shall make such provisions as required to insure adequate drainage of the trench to protect the bedding during construction operations. Proper preparation of foundation, placement of foundation material where required, and placement of bedding material shall precede the installation of the culvert. This shall include necessary leveling of the native trench bottom or the top of foundation materials as well as placement and grading of required bedding material to a uniform grade so that the entire length of pipe will be supported on a uniform slightly yield bedding. The backfill material shall be placed around the culvert in a manner to meet the requirements specified.

F.3.4.2 Bedding

1) General

If rock strata or boulders are encountered under the culvert within the limits of the required bedding, the rock or boulders shall be removed and replaced with bedding material. Special care may be necessary with rock or other unyielding foundations to cushion pipe from shock when blasting can be anticipated in the area. Where, in the opinion of the Engineer, the natural foundation soil is such as to require stabilization, such material shall be replaced by al layer of bedding material. Where an unsuitable material (peat, muck, etc) is encountered at or below invert elevation during excavation, the necessary subsurface exploration and analysis shall be made and corrective treatment shall be as directed by the Engineer.

2) Reinforced Concrete Circular Arch and Elliptical Pipe

A bedding shall be provided for the type of installation specified conforming to Figures F.1, F.2, F.3, and F.4 which define soil areas and critical dimensions, and Tables F.1 and F.2, which list generic soil types and minimum compaction requirements, and minimum bedding thicknesses for the two AASHTO Standard Installation Types.

3) Reinforced Concrete Box Sections

A bedding shall be provided for the type of installation specified conforming to Figures F.8 and F.9 unless in the opinion of the Engineer, the natural soil provides a suitable bedding

F.3.4.3 Placing Culvert Sections

1) General

Unless otherwise authorized by the Engineer, the laying of culvert sections on the prepared foundation shall be started at the outlet and with the spigot or tongue end pointing downstream and shall proceed toward the inlet end with the abutting sections properly matched, true to the established lines and grades. Where pipe with bells is installed, bell holes shall be excavated in the bedding to such dimensions that the entire length of the barrel of the pipe will be supported by the bedding when properly installed. Proper facilities shall be provided for hoisting and lowering the sections of culvert into the trench without disturbing the prepared foundation and the sides of the trench. The ends of the section shall be carefully cleaned before the section is jointed. The section shall be fitted and matched so that when laid in the bed it shall form a smooth, uniform conduit. When elliptical pipe with circular reinforcing or circular pipe with elliptical reinforcing is used, the pipe shall be laid in the trench in such position that the markings “Top” or “Bottom” shall not be more than 5° from the vertical plane through the longitudinal axis of the pipe.

2) Multiple pipe culverts

Multiple installations of reinforced concrete culverts shall be laid with the center lines of individual barrels parallel at the spacing shown on the plans. Pipe and box sections used in parallel installations require positive lateral bearing between the sides of adjacent pipe or box sections. Compacted earth fill, granular backfill, or grouting between the units is considered means of providing positive bearing.

F.3.4.4 Haunch, Lower Side and Backfill or Overfill

1) Reinforced Concrete Circular Arch and Elliptical Pipe

Haunch, lower side and backfill or overfill materials shall be installed to the limits shown on Figures F.1, F.2, F.3, and F.4.

2) Reinforced Concrete Box Sections

Haunch, lower side and backfill or overfill materials shall be installed to the limits shown on Figure F.7 and F.9 for the embankment or trench condition. Trenches shall have vertical walls and no over-excavating or sloping sidewalls shall be permitted.

3) Placing of Haunch, Lower Side and Backfill or Overfill

Compaction of fill material to the required density is dependent on the thickness of the layer of fill being compacted, soil type, soil moisture content, type of compaction equipment, and amount of compactive force and the length of time the force is applied. Fill material shall be placed in layers with a maximum thickness of 8inches and compacted to obtain the required density. The fill material shall be placed and compacted with care under the haunches of the culvert and shall be brought up evenly and simultaneously on both sides of the culvert. For the lower haunch areas of Type 1, 2, and 3 AASHTO Standard Installations, soils requiring 90% or greater Standard Proctor densities shall be placed in layers with a maximum thickness of 4 inches and compacted to obtain the required density. The width of trench shall be kept to the minimum required for installation of the culvert. Ponding or jetting will be only by the permission of the Engineer.

4) Protection of Culverts

Culverts shall be protected by a minimum of 3 feet of cover to prevent damage before permitting heavy construction equipment to pass over them during construction.

F.3.5 Measurement

Culverts shall be measured in linear feet installed in place, completed, and accepted. The number of feet shall be the average of the top and bottom center line lengths for pipe and box sections.

F.3.6 Payment

The length determined as herein given shall be paid for at the contract unit prices per linear foot bid for culverts of the several sizes and shapes, as the case may be, which prices and payments shall constitute full compensation for furnishing, handling, and installing the culvert and for all materials, labor, equipment, tools, and incidentals necessary to complete this item.

APPENDIX G

Electronic Files for Special Highway Drawings, SPIDA and CANDE-89 Sources

Appendix G which is given in the CD-ROM in the final report, ALDOT 930-592, includes the following items:

1. Special Highway Drawings (AutoCAD files)
   - Round concrete roadway pipes
   - Box culverts

2. SPIDA
   - SPIDA source
   - Exe. file
   - Input and output files in the Appendix A of the final report , ALDOT 930-592

3. CANDE-89
   - CANDE-89 source
   - Exe. file
   - Input and output files in the Appendix B of the final report , ALDOT 930-592

4. SPIDA and CANDE-89 users guides